APPENDIX A

ENVIRONMENTAL PLANNING AND COMPLIANCE

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

APPENDIX A-1

PUBLIC SCOPING MATERIAL

(NOI & PUBLIC MEETINGS)

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

US ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Memorandum for Record

March 16, 2010

To: File

From: CESPK-PD-RP (Doug Edwards, Environmental Planner)

Subject: LSJRFS Scoping Meeting

Scoping Period

January 15 – February 15, 2010

Public Notices (Attachment 1)

Federal Register
State Clearing House
Newspapers
    Stockton Bulletin
    Manteca Bulletin
    Lodi News Sentinel
    Ripon Record
    Tracy Press
SAJAFCA Website
Email Distribution List

Scoping Meeting (Attachment 2)

University of Pacific, Regents Dining Room
Wednesday, January 27, 2010
6:00 – 7:00

Comment Letters (Attachment 3)

USPS/FAX

River Islands at Lathrop, Susan Dell'Osso, Project Director

City of Lathrop, Cary Keaten, City Manager

California Department of Transportation, Kathy Selsor

Neumiller & Beardslee, John W. Stovall, Attorney at Law

San Joaquin County, Misquito & Vector Control District, John R. Stroh, Manager

California Department of Fish and Game, North Central Region, Jeff Drongessen, Acting Conservation Program Manger

San Joaquin River Group Authority, Dennis W. Westcot, Project Administrator

U.S. EPA, Region IX, Kathleen M. Goforth

Email

Private Individual, Nicky Suard,

Insite Environmental, Inc., Charlie Simpson, Principal

Friant Water Authority, Bill Luce, P.E. Resources Manager

Private, Richard Riley

South Delta Water Agency, John Herrick Law, John Herrick, Esq.,

San Joaquin River Group Authority, Dennis W. Westcot, Project Administrator

U.S. EPA, Region IX, James Munson

Regional Water Quality Control Board, Central Valley (Region 5 Sacramento), Sanj Joaquin TMDL &NPS Unit, Christine Joab

California Department of Transportation, Kathy Selsor

U.S. Bureau of Reclamation, Policy and Administration, Theresa Taylor

ATTACHMENT 1

Federal Register Notice

Notice of Completion & Environmental Document Transmittal

Email Distribution List

Email Content

Webpage Confirmation Email

Newspaper List Email

The above rates are effective for services rendered on or after October 1, 2009.

Dated: December 18, 2009.

Patricia Toppings,

OSD Federal Register Liaison Officer, Department of Defense.

[FR Doc. 2010–598 Filed 1–14–10; 8:45 am]

BILLING CODE 5001–06–P

DEPARTMENT OF DEFENSE

Department of the Army; Army Corps of Engineers

Notice of Intent To Prepare a Joint Environmental Impact Statement and Environmental Impact Report for the Lower San Joaquin River Feasibility Study

AGENCY: Department of the Army, U.S. Army Corps of Engineers; DOD.

ACTION: Notice of intent.

SUMMARY: The action being taken is the preparation of a joint environmental impact statement/environmental impact report (EIS/EIR) for the Lower San Joaquin River Feasibility Study (LSJRFS). The EIS/EIR will be prepared in accordance with the National Environmental Policy Act (NEPA) and the California Environmental Quality Act (CEQA). The U.S. Army Corps of Engineers (USACE) will serve as lead agency for compliance with NEPA, and the San Joaquin Area Flood Control Agency (SJAFCA) will serve as lead agency for compliance with CEQA. The LSJRFS will evaluate alternatives, including a locally preferred plan, for providing flood damage reduction and ecosystem restoration along the lower (northern) portion of the San Joaquin River system in the Central Valley of California. The approximate area of the proposed action and analysis is identified in Figure 1.

DATES: Written comments regarding the scope of the environmental analysis should be received at (*see* ADDRESSES) by February 15, 2010.

ADDRESSES: Written comments concerning this study and requests to be included on the LSJRFS mailing list should be submitted to Mr. Doug Edwards, U.S. Army Corps of Engineers, Sacramento District, *Attn:* Planning Division (CESPK–PD–R), 1325 J Street, Sacramento, CA 95814.

FOR FURTHER INFORMATION CONTACT: Mr. Doug Edwards via telephone at (916) 557–7062, e-mail at *Douglas.M.Edwards@usace.army.mil*, or regular mail at (*see* ADDRESSES).

SUPPLEMENTARY INFORMATION:

1. *Proposed Action.* USACE is preparing an EIS/EIR to analyze the environmental impacts associated with a range of alternatives for providing flood damage reduction and ecosystem restoration along the lower (northern) portion of the San Joaquin River system (Figure 1).

2. *Alternatives.* The EIS/EIR will address an array of alternatives for providing flood risk management alternatives that are intended to reduce flood risk within the project area. Alternatives analyzed during the investigation may include, but are not limited to, a combination of one or more of the following flood damage reduction measures: adding, modifying, and/or re-regulating storage on major tributaries; new transitory storage within flood plains, increasing conveyance by raising levees; widening channels and floodway areas; dredging; and constructing or modifying weirs and bypasses; and various floodplain management measures. Ecosystem restoration measures may include, but are not limited to, restoring riparian, wetland, and floodplain habitats, and/or constructing setback levees for habitat restoration.

3. *Scoping Process.*

a. A public scoping meeting will be held to present an overview of the LSJRFS and the EIS/EIR process, and to afford all interested parties with an opportunity to provide comments regarding the scope of analysis and potential alternatives. The public scoping meeting will be held at the University of Pacific, Regent's Dining Room, 3601 Pacific Avenue, Stockton, CA on January 27, 2010, from 6–8 p.m.

b. Potentially significant issues to be analyzed in depth in the EIS/EIR include project specific and cumulative effects on hydraulics, wetlands and other waters of the U.S., vegetation and wildlife resources, special-status species, esthetics, cultural resources, recreation, land use, fisheries, water quality, air quality, and transportation.

c. USACE is consulting with the State Historic Preservation Officer to comply with the National Historic Preservation Act and with the U.S. Fish and Wildlife Service and National Marine Fisheries Service to comply with the Endangered Species Act. USACE is also coordinating with the U.S. Fish and Wildlife Service to comply with the Fish and Wildlife Coordination Act

d. A 45-day public review period will be provided for all interested parties individuals and agencies to review and comment on the draft EIS/EIR. All interested parties are encouraged to respond to this notice and provide a current address if they wish to be notified of the draft EIS/EIR circulation.

4. *Availability.* The draft EIS/EIR is currently scheduled to be available for public review and comment in 2014.

Dated: December 29, 2009.

Thomas Chapman,

COL, EN Commanding.

[FR Doc. 2010–686 Filed 1–14–10; 8:45 am]

BILLING CODE 3720–58–P

DEPARTMENT OF DEFENSE

Department of the Army; Corps of Engineers

Notice of Solicitation for Estuary Habitat Restoration Program

AGENCY: Department of the Army, U.S. Army Corps of Engineers, DoD.

ACTION: Notice of solicitation for project applications.

SUMMARY: Congress has appropriated limited funds to the U.S. Army Corps of Engineers (Corps) and the National Oceanic and Atmospheric Administration (NOAA) for implementation of the Estuary Habitat Restoration Program as authorized in Section 104 of the Estuary Restoration Act of 2000, Title I of the Estuaries and Clean Waters Act of 2000 (Pub. L. 106–457) (accessible at *http://www.usace.army.mil/CECW/ERA/Pages/home.aspx)*. On behalf of the Estuary Habitat Restoration Council (Council) the Corps is soliciting proposals for estuary habitat restoration projects. The Council requests that all proposals address the potential effects of sea level change and other impacts related to climate change on the viability of the proposed restoration. This may take the form of considering climate change in the planning, design, siting, and construction of the project, or in testing new restoration technologies that may help to alleviate effects of climate change. This document describes project criteria and evaluation criteria the Council will use to determine which projects to recommend. Recommended projects must provide ecosystem benefits, have scientific merit, be technically feasible, and be cost-effective. Proposals selected for Estuary Habitat Restoration Program funding may be implemented in accordance with a cost-share agreement with the Corps; or a cooperative agreement with the Corps or NOAA, subject to availability of funds.

In addition to this solicitation and the application form, a Supplemental Guide for Prospective Applicants is available at: *http://www.usace.army.mil/CECW/*

Notice of Completion & Environmental Document Transmittal

Mail to: State Clearinghouse, P.O. Box 3044, Sacramento, CA 95812-3044 (916) 445-0613
For Hand Delivery/Street Address: 1400 Tenth Street, Sacramento, CA 95814

SCH # 2010012027

Project Title: Lower San Joaquin River Feasibility Study
Lead Agency: San Joaquin Area Flood Control Agency
Contact Person: Doug Edwards
Mailing Address: 1325 J Street
Phone: (916) 557-7026
City: Sacramento
Zip: 95814
County: Sacramento

Project Location: County: San Joaquin
City/Nearest Community: Stockton, Manteca, Lathrop
Cross Streets: ______
Zip Code: ______
Longitude/Latitude (degrees, minutes and seconds): ____° ____′ ____″ N / ____° ____′ ____″ W
Total Acres: ______
Assessor's Parcel No.: ______
Section: ______ Twp.: ______ Range: ______ Base: ______
Within 2 Miles: State Hwy #: ______
Waterways: ______
Airports: ______
Railways: ______
Schools: ______

Document Type:

CEQA:
☑ NOP
☐ Early Cons
☐ Neg Dec
☐ Mit Neg Dec

☐ Draft EIR
☐ Supplement/Subsequent EIR
(Prior SCH No.) ______
Other: ______

NEPA:
☑ NOI
☐ EA
☐ Draft EIS
☐ FONSI

Other:
☑ Joint Document
☐ Final Document
☐ Other: ______

RECEIVED
JAN [illegible] 2010
STATE CLEARING HOUSE

Local Action Type:

☐ General Plan Update
☐ General Plan Amendment
☐ General Plan Element
☐ Community Plan

☐ Specific Plan
☐ Master Plan
☐ Planned Unit Development
☐ Site Plan

☐ Rezone
☐ Prezone
☐ Use Permit
☑ Land Division (Subdivision, etc.)

☐ Annexation
☐ Redevelopment
☐ Coastal Permit
☐ Other: ______

Development Type:

☐ Residential: Units ______ Acres ______
☐ Office: Sq.ft. ______ Acres ______ Employees ______
☐ Commercial: Sq.ft. ______ Acres ______ Employees ______
☐ Industrial: Sq.ft. ______ Acres ______ Employees ______
☐ Educational: ______
☐ Recreational: ______
☐ Water Facilities: Type ______ MGD ______

☐ Transportation: Type ______
☐ Mining: Mineral ______
☐ Power: Type ______ MW ______
☐ Waste Treatment: Type ______ MGD ______
☐ Hazardous Waste: Type ______
☑ Other: Flood risk management and ecosystem restoration

Project Issues Discussed in Document:

☑ Aesthetic/Visual
☑ Agricultural Land
☐ Air Quality
☑ Archeological/Historical
☑ Biological Resources
☐ Coastal Zone
☑ Drainage/Absorption
☑ Economic/Jobs

☑ Fiscal
☑ Flood Plain/Flooding
☐ Forest Land/Fire Hazard
☑ Geologic/Seismic
☑ Minerals
☑ Noise
☐ Population/Housing Balance
☑ Public Services/Facilities

☑ Recreation/Parks
☐ Schools/Universities
☐ Septic Systems
☐ Sewer Capacity
☑ Soil Erosion/Compaction/Grading
☐ Solid Waste
☐ Toxic/Hazardous
☑ Traffic/Circulation

☑ Vegetation
☑ Water Quality
☑ Water Supply/Groundwater
☑ Wetland/Riparian
☑ Growth Inducement
☑ Land Use
☑ Cumulative Effects
☐ Other: ______

Present Land Use/Zoning/General Plan Designation:
Various

Project Description: *(please use a separate page if necessary)*
The EIS/EIR will be prepared to analyze environmental impacts associated with a range of alternatives for providing flood damage reduction and ecosystem restoration along the lower (northern) portion of the San Joaquin River system. Alternatives may include, but are not limited to, a combination of one or more of the following flood damage reduction measures: adding modifying, and/or re-regulating storage on major tributaries; new transitory storage within flood plains, increasing conveyance by raising levees; widening channels and floodway areas; dredging; and constructing or modifying weirs and bypasses; and various floodplain management measures. Ecosystem restoration measures may include, but are not limited to, restoring riparian, wetland, and floodplain habitats, and/or constructing setback levees for habitat restoration.

Note: The State Clearinghouse will assign identification numbers for all new projects. If a SCH number already exists for a project (e.g. Notice of Preparation or previous draft document) please fill in.

Reviewing Agencies Checklist

Lead Agencies may recommend State Clearinghouse distribution by marking agencies below with and "X".
If you have already sent your document to the agency please denote that with an "S".

_____ Air Resources Board
X Boating & Waterways, Department of
_____ California Highway Patrol
X Caltrans District #10
_____ Caltrans Division of Aeronautics
_____ Caltrans Planning
X Central Valley Flood Protection Board
_____ Coachella Valley Mtns. Conservancy
_____ Coastal Commission
_____ Colorado River Board
_____ Conservation, Department of
_____ Corrections, Department of
X Delta Protection Commission
_____ Education, Department of
_____ Energy Commission
X Fish & Game Region #2, 3
X Food & Agriculture, Department of
_____ Forestry and Fire Protection, Department of
_____ General Services, Department of
_____ Health Services, Department of
_____ Housing & Community Development
_____ Integrated Waste Management Board
X Native American Heritage Commission

X Office of Emergency Services
X Office of Historic Preservation
_____ Office of Public School Construction
X Parks & Recreation, Department of
_____ Pesticide Regulation, Department of
_____ Public Utilities Commission
X Regional WQCB #5
_____ Resources Agency
_____ S.F. Bay Conservation & Development Comm.
_____ San Gabriel & Lower L.A. Rivers & Mtns. Conservancy
_____ San Joaquin River Conservancy
_____ Santa Monica Mtns. Conservancy
_____ State Lands Commission
_____ SWRCB: Clean Water Grants
_____ SWRCB: Water Quality
_____ SWRCB: Water Rights
_____ Tahoe Regional Planning Agency
_____ Toxic Substances Control, Department of
X Water Resources, Department of

_____ Other: _____
_____ Other: _____

Local Public Review Period (to be filled in by lead agency)

Starting Date January 15, 2010
Ending Date February 15, 2010

Lead Agency (Complete if applicable):

Consulting Firm:
Address:
City/State/Zip:
Contact:
Phone:

Applicant: San Joaquin Area Flood Control Agency
Address: 22 East Weber Avenue, Suite 301
City/State/Zip: Stockton, CA 95202-2317
Phone: (209) 937-7900

Signature of Lead Agency Representative: James B. Giottonini For **Date:** 01/14/2010

Authority cited: Section 21083, Public Resources Code. Reference: Section 21161, Public Resources Code.

REGISTRY
MAILING LIST

emailed via 1/15/10

First	Last	Title	Affiliation	Address	State/City/Zip	Phone	E-Mail
	Federal Participants						
Dennis	Cardoza	Congressman, 18th District	U.S. House of Representatives	Merced County Administration Bldg., 2222 M Street, Suite 305	Merced, CA 95340	(209) 383-4455	
Jerry	McNerney	Congressman, 11th District	U.S. House of Representatives	PO Box 12022	Pleasanton, CA 94588	(925) 833-0643	
Barbara	Boxer	Senator	U.S. Senate	501 I Street, Suite 7-600	Sacramento, CA 95814	(916) 448-2787	
Dianne	Feinstein	Senator	U.S. Senate	One Post Street, Suite 2450	San Francisco, CA 94104	(415) 393-0707	
Gary	Prost	Field Representative for Congressman McNerney	U.S. Government	2222 Grand Canal Blvd., #7	Stockton, CA	(202) 476-5552	gary.prost@mail.house.gov
Deedee	D'Adamo	Sr. Policy Advisor for Congressman Cardoza	U.S. Government	1010 10th Street, Suite 5800	Modesto, CA 95354	(209) 527-1914	DDADAMO@EARTHLINK.NET
Colonel	Chapman	District Commander	U.S. Army Corps of Engineers	1325 J Street	Sacramento, CA 95814	(916) 557-7490	Thomas.C.Chapman.COL@usace.army.mil
Frank	Piccola	Chief, Planning	U.S. Army Corps of Engineers	1325 J Street	Sacramento, CA 95814	(916) 557-6735	Francis.C.Piccola@spk01.usace.army.mil
Kevin	Knuuti	Chief, Engineering Div.	U.S. Army Corps of Engineers	1325 J Street	Sacramento, CA 95814	(916) 557-7623	Kevin.Knuuti@usace.army.mil
Brandon	Muncy	Chief, Civil Works Br.	U.S. Army Corps of Engineers	1325 J Street	Sacramento, CA 95814	(916) 557-6682	Brandon.C.Muncy@spk01.usace.army.mil
Michelle	Williams	Project Manager	U.S. Army Corps of Engineers	1325 J Street	Sacramento, CA 95814	(916) 557-7098	Michelle.R.Williams@usace.army.mil
Cindy	Tejeda	South Pacific Division	U.S. Army Corps of Engineers	1455 Market Street	San Francisco, CA 94105	(415) 503-6572	cindy.l.tejeda@usace.army.mil
Kathy	Schaefer	Region IX Engineer	Federal Emergency Management Agency	1111 Broadway, Suite 1200	Oakland, CA 94607-4052	(510) 292-9075	kathleen.schaefer@dhs.gov
Susan	Jones	Director	U.S. Fish & Wildlife Service	2800 Cottage Way, Rm. W-2605	Sacramento, CA 95825-1888	(916) 414-6600	susan_p_jones@fws.gov
	State Participants						
Jose	Morales	Rep for Lois Wolk Sen District #5	Rep. for Senator Lois Wolk	State Capitol, Room 5066	Sacramento, CA 95814	(916) 651-4005	jose.morales@sen.ca.gov
Jeff	Denham	Senator	State Senate, 12th District	1231 8th Street, Suite. 175	Modesto, CA 95354	(209) 577-6592	senator.denham@sen.ca.gov
Bob	Wiedman	Dep Dist Dir Rep for Dave Cogdill Sen Dist #14	Rep. for Senator Cogdill	1308 W. Main Street, Ste. C	Ripon, CA 95366	(209) 599-8540	bob.wiedman@sen,ca.gov
Dave	Cogdill	Senator	State Senate, 5th District	1308 W. Main Street, Ste. C	Ripon, CA 95366	(209) 599-8540	bob.wiedman@sen.ca.gov
Brian	Regnart	Rep for Bill Berryhill, Assembly Dist #26	Rep. for Assemblymember Berryhill	4557 Quail Lakes Dr. Ste C-3	Stockton, CA 95207	(209) 473-6972	adam.struck@asm.ca.gov
Bill	Berryhill	Assemblymember, 26th District	State Assembly	4557 Quail Lakes Dr. Ste C-3	Stockton, CA 95207	(209) 473-6972	adam.struck@asm.ca.gov
Anne	Baird	Rep. for Alyson Huber, Assembly Dist #10	Rep. for Assemblymember Huber	218 W. Pine Street	Lodi, CA 95240	(209) 333-5330	anne.baird@asm.ca.gov
Alyson	Huber	Assemlymember, 10th District	State Assembly	218 W. Pine Street	Lodi, CA 95240	(209) 333-5330	anne.baird@asm.ca.gov
Victor	Francovich	Rep Cathleen Galgiani, Assembly Dist #17	Rep. for Assemblymember Cathleen Galgiani	31 E. Channel St., Suite 306	Stockton, CA 95202	(209) 948-7479	victor.francovich@asm.ca.gov
Cathleen	Galgiani	Assemblymember, 17th District	State Assembly	31 E. Channel St., Suite 306	Stockton, CA 95202	(209) 948-7479	victor.francovich@asm.ca.gov
		Planning Department	CALTRANS District 10	1976 E Charter Way / East Dr. Martin Luther King Jr. Blvd	Stockton, CA 95205-7015	(209) 948-7543	robert_boswell@dot.ca.gov
Jay	Punia	Executive Officer	Central Valley Flood Protection Board	3310 El Camino Avenue, Rm LL40	Sacramento, CA 95821	(916) 574-0609	jpunia@water.ca.gov
Ben	Carter	Board President	Central Valley Flood Protection Board	3310 El Camino Avenue, Rm LL40	Sacramento, CA 95821	(916) 574-0609	lpendleb@water.ca.gov
Teri	Rie	Board Member	Central Valley Flood Protection Board	3310 El Camino Avenue, Rm LL40	Sacramento, CA 95821	(916) 574-0609	teririe@comcast.net
Gary	Bardini	Chief of Flood Management	Department of Water Resources	3310 El Camino Avenue, Rm LL60	Sacramento, CA 95821	(916) 574-0601	gbardini@water.ca.gov
Eric	Koch		Department of Water Resources	3310 El Camino Avenue, Rm 140	Sacramento, CA 95821	(916) 574-0385	ekoch@water.ca.gov
	Local Participants						
Leroy	Ornellas	Board Member	Board of Supervisors, District 5	44 N. San Joaquin St., 6th Flr., Ste 627	Stockton, CA 95202	(209) 468-3113	lornellas@sjgov.org
Carlos	Villapudua	Chairman	Board of Supervisors, District 1	44 N. San Joaquin St., 6th Flr., Ste 627	Stockton, CA 95202	(209) 468-3113	cvillapadua@sjgov.org

REGISTRY
MAILING LIST

First	Last	Title	Affiliation	Address	State/City/Zip	Phone	E-Mail
Larry	Ruhstaller	Vice Chairman	Board of Supervisors, District 2	44 N. San Joaquin St., 6th Flr., Ste 627	Stockton, CA 95202	(209) 468-3113	lruhstaller@sjgov,org
Steve J	Bestolarides	Board Member	Board of Supervisors, District 3	44 N. San Joaquin St., 6th Flr., Ste 627	Stockton, CA 95202	(209) 468-3113	sbestolarides@sjgov.org
Ken	Vogel	Board Member	Board of Supervisors, District 4	44 N. San Joaquin St., 6th Flr., Ste 627	Stockton, CA 95202	(209) 468-3113	kvogel@sjgov.org
Anne	Castillou	Project Development/Habitat Plan	County of San Joaquin, Council of Governments	555 E Weber Avenue	Stockton, CA 95202-3016	(209) 235-0449	castillou@sjcog.org
Steve	Mayo	Project Development/Habitat Plan	County of San Joaquin, Council of Governments	555 E Weber Avenue	Stockton, CA 95202-3016	(209) 235-0449	mayo@sjcog.org
David E.	Wooten	Office of the County Counsel	San Joaquin County	44 N. San Joaquin St., 6th Flr., Ste 679	Stockton, CA 95202	(209) 468-2980	
Terrence	Dermody	Special Water Counsel	San Joaquin County	44 N. San Joaquin St., 6th Flr., Ste 679	Stockton, CA 95202	(209) 468-2980	trpd@aol.com
Phonxay	Keokham	Rep. County Administrators Office	San Joaquin County	222 E. Weber Avenue, Room 707	Stockton, CA 95202	(209) 468-3203	pkeokham@sjgov.org
Tom	Flinn	P W Director	San Joaquin County	1810 E. Hazelton Avenue	Stockton, CA 95202	(209) 468-3100	tflinn@sjgov.org
Steve	Winkler	Deputy PW Director	San Joaquin County	1810 E. Hazelton Avenue	Stockton, CA 95202	(209) 468-3031	swinkler@sjgov.org
Mel	Lytle	Water Resources Coordinator	San Joaquin County	1810 E. Hazelton Avenue	Stockton, CA 95202	(209) 468-9360	mlytle@sjgov.org
Butch	Waddle	Channel Maint Superintendant	San Joaquin County	1810 E. Hazelton Avenue	Stockton, CA 95202	(209) 468-9698	lwaddle@sjgov,org
Mark	Connelly	Engineering Serv Mgr	San Joaquin County	1810 E. Hazelton Avenue	Stockton, CA 95202	(209) 953-7617	mconnelly@sjgov.org
Candis	Oldham	Program Assistant	San Joaquin County	1810 E. Hazelton Avenue	Stockton, CA 95202	(209) 468-3174	coldham@sjgov.org
Katina	Conn	Management Analyst II	San Joaquin County	1810 E. Hazelton Avenue	Stockton, CA 95205	(209) 468-3061	kconn@sjgov.org
Ron	Baldwin	Director, Emergency Ops	San Joaquin County	2101 E. Earhart Ave., Suite 300	Stockton, CA 95206	(209) 953-6200	rbaldwin@sjgov.org
Kerry	Sullivan	Director, Community Development Department	San Joaquin County	1810 E. Hazelton Avenue	Stockton, CA 95202	(209) 468-3121	ksullivan@sjgov.org
Susan	Palmeri	Director, Stockton Metro Airport	San Joaquin County	5000 S. Airport Way	Stockton, CA 95206	(209) 953-6000	spalmeri@sjgov.org
Donna	Heran, REHS	Environmental Health Department	San Joaquin County	600 E. Main Street	Stockton, CA 95202	(209) 468-3420	dbrownfield@sjcehd.com
Steve	Salvatore	Director, Public Works	City of Lathrop	390 Towne Centre Drive	Lathrop, CA 95330-9358	(209) 941-7491	ssalvatore@ci.lathrop.ca.us
		Planning Department	City of Lathrop	390 Towne Centre Drive	Lathrop, CA 95330-9358	(209) 941-7290	
		Planning Department	City of Lodi	221 West Pine Street	Lodi, CA 95241-1910	(209) 333-6711	
Jim	Stone	Dept. of Public Works	City of Manteca	1001 W. Center St.	Manteca, CA 95337	(209) 825-2592	jstone@ci.manteca.ca.us
Ken	Zuidervaart	Director of Planning	City of Ripon	259 N. Wilma Avenue	Ripon, CA 95366	(209) 599-2108	kzuidervaart@cityofripon.org
Jim	Giottonini	Executive Director	SJAFCA	22 E. Weber Ave., Suite 301	Stockton, CA 95202-2317	(209) 937-8339	jim.giottonini@ci.stockton.ca.us
Diana	Lowery	Councilmember Dist 4, SJAFCA Vice-Chairperson	Stockton City Council/SJAFCA Board	425 N. El Dorado Street	Stockton, CA 95202	(209) 937-8244	diana.lowery@ci.stockton.ca.us
		Administrator/Engineer	City of Stockton	City Hall, 425 N. El Dorado Street	Stockton, CA 95202	(209) 937-8460	
Connie	Cochran	Public Information Officer	City of Stockton	City Hall, 425 N. El Dorado Street	Stockton, CA 95202	(209) 937-8827	connie.cochran@ci.stockton.ca.us
Kevin	O'Rourke	Interim City Manager	City of Stockton, City Manager's Office	City Hall, 425 N. El Dorado Street	Stockton, CA 95202	(209) 937-8212	CityManager@ci.stockton.ca.us
Richard E.	Nosky, Jr.	City Attorney	City of Stockton, Office of the City Attorney	City Hall, 425 N. El Dorado Street	Stockton, CA 95202	(209) 937-8333	
John	Luebberke	Assistant City Attorney	City of Stockton, Office of the City Attorney	City Hall, 425 N. El Dorado Street	Stockton, CA 95202	(209) 937-8333	
Mike	Niblock	Director	City of Stockton, Community Development Services	345 N. Eldorado Street	Stockton, CA 95202	(209) 937-8561	
Mark	Madison	Director	City of Stockton Municipal Utilities District	2500 Navy Drive	Stockton, CA 952026	(209) 937-8700	Mark.Madison@ci.stockton.ca.us
Tony	Tovar	Senior Civil Engineer	City of Stockton Municipal Utilities District	2500 Navy Drive	Stockton, CA 952026	(209) 937-8790	Antonio.Tovar@ci.stockton.ca.us
Lance	Calkins	Chief	City of Stockton, Fire Department	City Hall, 425 N. El Dorado Street	Stockton, CA 95202	(209) 937-8801	Lance.Calkins@ci.stockton.ca.us
	Reclamation Districts						

REGISTRY
MAILING LIST

First	Last	Title	Affiliation	Address	State/City/Zip	Phone	E-Mail
Susan	Dell'Osso	Project Director	River Islands at Lathrop	73 W. Stewart Road	Lathrop, CA 95330	(209) 879-7900	sdellosso@cambaygroup.com
Ramon	Batista	Director of Planning	River Islands at Lathrop	73 W. Stewart Road	Lathrop, CA 95330	(209) 879-7900	rbatista@cambaygroup.com
John	Cain	Director, Restorations Program	Natural Heritage Institute	100 Pine St., Suite 1550	San Francisco, CA 94708	(415) 693-3000	jcain@n-h-i.org
Dante	Nomellini	Principal Attorney	RD 17	PO Box 1461	Stockton, CA 95201	(209) 465-5883	ngmplcs@pacbell.net
Chris	Neudeck	Kjeldsen-Sinnock, Nuedeck	RD1,2,17,524,544,2023,2027,2030, 2040, 2042,2089,2095,2113,2115, 2119, 2126	P.O. Box 844	Stockton, CA 95201	(209) 946-0268	cneudeck@ksninc.com
Drew	Meyers	District Trustee	RD 1608, Smith Track	3868 Fourteen Mile Dr.	Stockton, CA 95219	(209) 403-1223	cdrusn9057@aol.com
Al	Hoslet	Principal Attorney	RD 2042, Bishop Tract	311 East Main Street, Suite 504	Stockton, CA 95202	(209) 943-5551	ahoslett@sbcglobal.net
Anthony	Lopes	Siegfried Engineering	RD 2074	3244 Brookside Road	Stockton, CA 95219	(209) 943-2021	ajlopes@siegfriedeng.com
Jeff	Kasper	Deputy Port Director	Port of Stockton	2201 W. Washington Street	Stockton, CA 95203	(209) 946-0246	portmail@stocktonport.com
	Consultants						
Scott	Brown	Supervising Engineer	Parsons Brinckerhoff	3840 Rosin Court, Suite 200	Sacramento, CA 95834	(916) 567-2506	brownsc@pbworld.com
Cheryl	Creson	Local Business Executive	Parsons Brinckerhoff	3480 Rosin Court, Suite 200	Sacramento, CA 95834	(916) 526-2500	creson@pbworld.com
Dave	Peterson	Principal	Peterson Brusted Inc.	1180 Iron Point Road, Suite 260	Folsom, CA 95630	(916) 792-6285	dpeterson@pbieng.com
Barry	O'Regan	Vice President	Peterson Brusted Inc.	1180 Iron Point Road, Suite 260	Folsom, CA 95630	(916) 608-2232	boregan@pbieng.com
Jeff	Twitchill	Office Principal	Wood Rogers	3301 C Street, Bdlg 100B	Sacramento, CA 95816	(916) 326-5225	jtwitchell@woodrogers.com
Michael	Conrad	Office Principal	Michael Baker Jr., Inc.	1730 I Street, Ste. 100	Sacramento, CA 95834	(916) 329-3169	mconrad@mbakercorp.com
	Other Agenicies/ Individuals/Groups						
Will	Price	Professor	UOP, Business and Engineering Management	3601 Pacific Ave.	Stockton, CA 95211	(209) 946-2638	wprice@pacific.edu
Ravi	Jain	Dean	UOP, School of Engineering and Computer Science	Baun 3rd Floor, 3601 Pacific Ave.	Stockton, CA 95211	(209) 946-3066	rjain@pacific.edu
Ted	Leland	Vice President	UOP, Community Relations	3601 Pacific Ave.	Stockton, CA 95211		tleland@pacific.edu
Mark	Plovnick	Director	UOP, Office of Economic Development	3601 Pacific Ave.	Stockton, CA 95211	(209) 946-2466	mplovnick@pacific.edu
Will	Stringfellow	Professor	UOP, Environmental Research Center	Sears 116, 3601 Pacific Ave.	Stockton, CA 95211	(209) 946-2497	wstringfellow@pacific.edu
Camilla	Saviz	Professor	UOP, School of Engineering and Computer Science	Anderson 208, 3601 Pacific Ave.	Stockton, CA 95211		csaviz@pacific.edu
Gary	Litton	Professor	UOP, School of Engineering and Computer Science	Anderson 205, 3601 Pacific Ave.	Stockton, CA 95211	(209) 946-3070	glitton@pacific.edu
Walter	McInnis	President	San Joaquin Audobon Society	PO Box 7755	Stockton, CA 95267	(209) 473-3904	kaseyfoley@sbcglobal.net
John	Beckman	Executive Director	Building Industry Association of the Delta	315 N. San Joauqin Street, 202	Stockton, CA 95202	(209) 235-7831	johnb@biadelta.org
			Sierra Club Foundation - Delta Sierra Group	PO Box 9258	Stockton, CA 95208		dsg.webmaster@mlc.sierraclub.org
Barbara	Barrigan-Parrilla	Campaign Director	Restore the Delta	PO Box 691088	Stockton, CA 95269	(209) 479-2053	Barbara@restorethedelta.org
Linda	Fiack	Executive Director	Delta Protection Commission	14215 River Road	Walnut Grove, CA 95690	(916) 776-2290	dpc@citlink.net
Margit	Aramburu	Director	UOP Natural Resources Institute	Baun Hall, 3601 Pacific Avenue	Stockton, CA 95211	(831) 419-0905	margithind@comcast.net
Bill	Jennings	Executive Director	California Sportfishing Protection Alliance	3536 Rainier Avenue	Stockton, CA 95204	(209) 464-5067	deltakeep@ol.com
Jeremy	Terhune	San Joaquin Valley Representative	Friends of the Lower Calaveras	4555 Pershing Ave., #33-373	Stockton, CA 95207	(209) 922-8215	jterhune@defenders.org
James	Ramos	Chairman	Native American Heritage Commission	915 Capital Mall, Room 364	Sacramento, CA 95814	(916) 653-4082	nahc@pacbell.net
Katerine	Perez	Representative	Connections North Valley Yokut Tribe	PO Box 4123	Stockton, CA 95204		
		CEQA-ISR Division	San Joaquin Valley APCD	1990 E. Gettysburg Avenue	Fresno, CA 93726		

REGISTRY
MAILING LIST

	First	Last	Title	Affiliation	Address	State/City/Zip	Phone	E-Mail
		UTILITIES						
			Aqueduct Section	East Bay Mud	PO Box 228	Stockton, CA 95201		
	George	Biagi, Jr.	President	Central Delta Water Agency	PO Box 1461	Stockton, CA 95201	(209) 465-5883	
				North Delta Water Agency	921 11th Street, #703	Sacramento, CA 95814		
	John	Herrick	Counsel & Manager	South Delta Water Agency	4255 Pacific Avenue, Suite 2	Stockton, CA 95207	(209) 956-0150	jherrlaw@aol.com
	Anders	Christiansen	General Manager	Woodbridge Irrigation District	18777 N. Lower Sacramento Rd.	Stockton, CA 95258	(209) 369-6808	wid2000@softcom.net
	Kevin	Kauffman		Stockton East Water District	PO Box 5157	Stockton, CA 95205-7015		
			Manager of Engineering	Union Paciic RailRoad	833 East 8th Street	Stockton, CA 95206		
			Attention: Land Agent	PG&E Land Rights Office	4040 West Lane	Stockton, CA 95201		
	Ed	Harrington	General Manager	San Francisco Public Utilities District	1155 Market Street	San Francisco, CA 95103	(415) 554-1600	eharrington@sfwater.org
	Juan	Acosta	Director, Govt. Affairs	Burlington Northern & SFRR	2500 Lou Menk Dr.	Fort Worth, TX 76131		
				California Water Service	1550 W Fremont Street, Suite 100	Stockton, CA 95203	(209) 547-7900	
				COMCAST	6505 Tam O'Shanter Drive	Stockton, CA 95210	(800) 866-2278	
	Robert	Ocosta		Pacific Bell	44 W Yokuts Avenue	Stockton, CA 95207		
	Thomas	Samaniego	Sr. New Business Rep.	Pacific Gas & Electric, Estimating & Mapping Department	PO Box 930	Stockton, CA 95201-0930	(209) 942-1793	TJS7@pge.com
				AT&T	4950 Pacific Avenue	Stockton, CA 95207-2307	(209) 952-9161	
	Greg	Carney	President, Chief Operating Officer	Stockton Terminal Eastern RR Co.	1330 N. Broadway Avenue	Stockton, CA 95205	(209) 466-7001	greg@sterailroad.com

Edwards, Douglas M SPK

From:	Marlo Duncan [Marlo.Duncan@ci.stockton.ca.us]
Sent:	Wednesday, January 13, 2010 3:03 PM
To:	Marlo Duncan
Subject:	Public Scoping Meeting Notice
Attachments:	Scoping Meeting Notice Jan 27 at UOP.pdf

Scoping Meeting Notice Jan 27 ...

NOTICE OF INTENT TO PREPARE A JOINT ENVIRONMENTAL IMPACT STATEMENT AND ENVIRONMENTAL IMPACT REPORT FOR THE LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

Dear Mail Recipient,
Attached is a Public Scoping Meeting Notice concerning the Lower San Joaquin River Feasibility Study. The meeting will be held at 6:00 p.m. on Wednesday, January 27, 2010 at the University of the Pacific, Regent's Dining Room - Anderson Hall, 3601 Pacific Avenue, Stockton, CA. See attachment for further detail, available parking, etc.

You are receiving this email notification because you are on an mail registry list for public scoping meeting notices. If you wish to be removed from this mail recipient list, please reply to this email. Thank you.

Marlo A. Duncan
San Joaquin Area Flood Control Agency
22 E. Weber Avenue, Room 301
Stockton, CA 95202
Marlo.Duncan@ci.stockton.ca.us
(209) 937-7900
Visit our web site at www.sjafca.com <http://www.sjafca.com/>

SJAFCA
San Joaquin Area FLOOD CONTROL Agency
22 E. Weber Avenue, Room 301, Stockton, CA 95202-2317 (209) 937-7900

JOINT ENVIRONMENTAL IMPACT STATEMENT AND ENVIRONMENTAL IMPACT REPORT FOR THE LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

SCOPING MEETING NOTICE

DATE/TIME: WEDNESDAY, JANUARY 27, 2010 AT 6:00 P.M.

LOCATION: UNIVERSITY OF THE PACIFIC
REGENT'S DINING ROOM – ANDERSON HALL
3601 PACIFIC AVENUE, STOCKTON, CA

- Environmental Impact Study/Environmental Impact Report (EIS/EIR) Objectives, Process, and Opportunities for Public Input
- Lower San Joaquin River Feasibility Study EIS/EIR Discussion and Comments

Enclosures

Notice of Intent to Prepare a Joint Environmental Impact Statement and Environmental Impact Report for the Lower San Joaquin River Feasibility Study

AGENCIES: Department of the Army, U.S. Army Corps of Engineers; San Joaquin Area Flood Control Agency.

ACTION: Notice of intent/Notice of preparation.

SUMMARY: The action being taken is the preparation of a joint environmental impact statement/environmental impact report (EIS/EIR) for the Lower San Joaquin River Feasibility Study (LSJRFS). The EIS/EIR will be prepared in accordance with the National Environmental Policy Act (NEPA) and the California Environmental Quality Act (CEQA). The U.S. Army Corps of Engineers (USACE) will serve as lead agency for compliance with NEPA, and the San Joaquin Area Flood Control Agency (SJAFCA) will serve as lead agency for compliance with CEQA. The LSJRFS will evaluate alternatives, including a locally preferred plan, for providing flood damage reduction and ecosystem restoration along the lower (northern) portion of the San Joaquin River system in the Central Valley of California.

DATES: Written comments regarding the scope of the environmental analysis should be received at (see **ADDRESSES**) by February 15, 2010.

ADDRESSES: Written comments concerning this study and requests to be included on the LSJRFS mailing list should be submitted to Mr. Doug Edwards, U.S. Army Corps of Engineers, Sacramento District, Attn: Planning Division (CESPK-PD-R), 1325 J Street, Sacramento, CA 95814.

FOR FURTHER INFORMATION CONTACT: Mr. Doug Edwards via telephone at (916) 557-7026, e-mail at Douglas.M.Edwards@usace.army.mil, or regular mail at (see **ADDRESSES**).

SUPPLEMENTARY INFORMATION:

1. ***Proposed Action.*** USACE is preparing an EIS/EIR to analyze the environmental impacts associated with a range of alternatives for providing flood damage reduction and ecosystem restoration along the lower (northern) portion of the San Joaquin River system (Figure 1).

2. ***Alternatives.*** The EIS/EIR will address an array of alternatives for providing flood risk management alternatives that are intended to reduce flood risk within the project area. Alternatives analyzed during the investigation may include, but are not limited to, a combination of one or more of the following flood damage reduction measures: adding, modifying, and/or re-regulating storage on major tributaries; new transitory storage within flood plains, increasing conveyance by raising levees; widening channels and floodway areas; dredging; and constructing or modifying weirs and bypasses; and various floodplain management measures. Ecosystem restoration measures may include, but are not limited to, restoring riparian, wetland, and floodplain habitats, and/or constructing setback levees for habitat restoration.

3. ***Scoping Process.***

a. A public scoping meeting will be held to present an overview of the LSJRFS and the EIS/EIR process, and to afford all interested parties with an opportunity to provide comments regarding the scope of analysis and potential alternatives. **The public scoping meeting will be held at the University of Pacific, Regent's Dining Room, Anderson Hall, 3601 Pacific Avenue, Stockton, CA on January 27, 2010, from 6:00 – 8:00 p.m.**

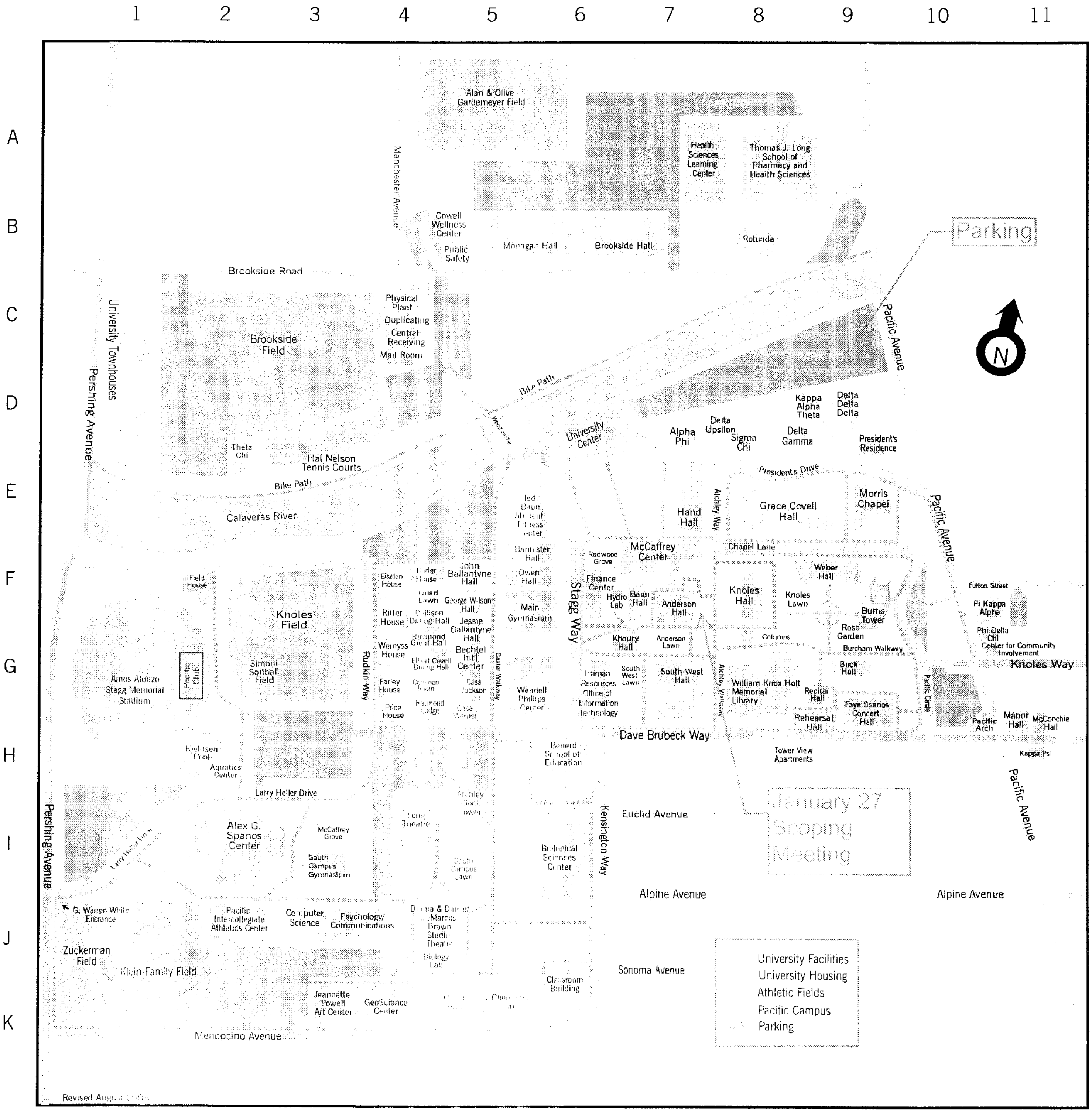
b. Potentially significant issues to be analyzed in depth in the EIS/EIR include project specific and cumulative effects on hydraulics, wetlands and other waters of the U.S., vegetation and wildlife resources, special-status species, esthetics, cultural resources, recreation, land use, fisheries, water quality, air quality, and transportation.

c. USACE is consulting with the State Historic Preservation Officer to comply with the National Historic Preservation Act and with the U.S. Fish and Wildlife Service and National Marine Fisheries Service to comply with the Endangered Species Act. USACE is also coordinating with the U.S. Fish and Wildlife Service to comply with the Fish and Wildlife Coordination Act.

d. A 45-day public review period will be provided for all interested parties individuals and agencies to review and comment on the draft EIS/EIR. All interested parties are encouraged to respond to this notice and provide a current address if they wish to be notified of the draft EIS/EIR circulation.

4. ***Availability.*** The draft EIS/EIR is currently scheduled to be available for public review and comment in 2014.

University of the Pacific



Campus Map

From: Marlo Duncan [Marlo.Duncan@ci.stockton.ca.us]
Sent: Tuesday, January 19, 2010 7:47 AM
To: Edwards, Douglas M SPK
Cc: Roger Churchwell; Williams, Michelle R SPK
Subject: RE: Public Scoping Meeting Notice

Yes! This posted on 1/15 ~ our web address is www.sjafca.com and the item can be found on our "News" page. Thanks,

Marlo A. Duncan
San Joaquin Area Flood Control Agency
22 E. Weber Avenue, Room 301
Stockton, CA 95202
Marlo.Duncan@ci.stockton.ca.us
(209) 937-7900
Visit our web site at www.sjafca.com <http://www.sjafca.com/>

>>> "Edwards, Douglas M SPK" <Douglas.M.Edwards@usace.army.mil>
>>> 1/15/2010 8:37 AM >>>

Did SJAFCA post the notice on its website?

Doug Edwards, PhD, AICP
Senior Environmental Planner
U.S. Army Corps of Engineers
1325 J Street, Planning Division
Sacramento, CA 95814-2922
(916) 557-7026

-----Original Message-----
From: Marlo Duncan [mailto:Marlo.Duncan@ci.stockton.ca.us]
Sent: Wednesday, January 13, 2010 3:03 PM
To: Marlo Duncan
Subject: Public Scoping Meeting Notice

NOTICE OF INTENT TO PREPARE A JOINT ENVIRONMENTAL IMPACT STATEMENT AND ENVIRONMENTAL IMPACT REPORT FOR THE LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

Dear Mail Recipient,
Attached is a Public Scoping Meeting Notice concerning the Lower San Joaquin River Feasibility Study. The meeting will be held at 6:00 p.m. on Wednesday, January 27, 2010 at the University of the Pacific, Regent's Dining Room - Anderson Hall, 3601 Pacific Avenue, Stockton, CA. See attachment for further detail, available parking, etc.

You are receiving this email notification because you are on an mail registry list for public scoping meeting notices. If you wish to be removed from this mail recipient list, please reply to this email. Thank you.

Marlo A. Duncan
San Joaquin Area Flood Control Agency
22 E. Weber Avenue, Room 301
Stockton, CA 95202
Marlo.Duncan@ci.stockton.ca.us
(209) 937-7900
Visit our web site at www.sjafca.com <http://www.sjafca.com/>

From: Marlo Duncan [Marlo.Duncan@ci.stockton.ca.us]
Sent: Tuesday, January 19, 2010 7:46 AM
To: Edwards, Douglas M SPK
Cc: Roger Churchwell
Subject: RE: Public Scoping Meeting Notice

The Stockton Record (on 1/15)
Manteca Bulletin (1/15)
Lodi News Sentinel (1/15)
Ripon Record (1/13)
Tracy Press (1/16)

>>> "Edwards, Douglas M SPK" <Douglas.M.Edwards@usace.army.mil>
>>> 1/15/2010 8:28 AM >>>

Thanks Marlo. In which newspapers were the notices published?

Doug Edwards, PhD, AICP
Senior Environmental Planner
U.S. Army Corps of Engineers
1325 J Street, Planning Division
Sacramento, CA 95814-2922
(916) 557-7026

-----Original Message-----
From: Marlo Duncan [mailto:Marlo.Duncan@ci.stockton.ca.us]
Sent: Wednesday, January 13, 2010 3:03 PM
To: Marlo Duncan
Subject: Public Scoping Meeting Notice

NOTICE OF INTENT TO PREPARE A JOINT ENVIRONMENTAL IMPACT STATEMENT AND ENVIRONMENTAL IMPACT REPORT FOR THE LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

Dear Mail Recipient,
Attached is a Public Scoping Meeting Notice concerning the Lower San Joaquin River Feasibility Study. The meeting will be held at 6:00 p.m. on Wednesday, January 27, 2010 at the University of the Pacific, Regent's Dining Room - Anderson Hall, 3601 Pacific Avenue, Stockton, CA. See attachment for further detail, available parking, etc.

You are receiving this email notification because you are on an mail registry list for public scoping meeting notices. If you wish to be removed from this mail recipient list, please reply to this email. Thank you.

Marlo A. Duncan
San Joaquin Area Flood Control Agency
22 E. Weber Avenue, Room 301
Stockton, CA 95202
Marlo.Duncan@ci.stockton.ca.us
(209) 937-7900
Visit our web site at www.sjafca.com <http://www.sjafca.com/>

ATTACHMENT 2

Agenda

PowerPoint Presentation

Handouts

Sign-in Sheet

Transcripts

Lower San Joaquin River Feasibility Study
NEPA/CEQA Scoping Meeting

January 27, 2010
6:00 - 7:30 P.M.

University of Pacific, Regent's Dining Room
3601 Pacific Avenue, Stockton, CA

AGENDA

1. **Housekeeping Items**
2. **Host's Welcome**
3. **Local Conditions**
4. **Corps Planning Process**
5. **Environmental Compliance**
6. **Comments**
7. **Open House**

Written comments concerning this study and requests to be included on the Lower San Joaquin River Feasibility Study mailing list should be submitted to Mr. Doug Edwards, U.S. Army Corps of Engineers, Sacramento District, Attn: Planning Division (CESPK-PD-R), 1325 J Street, Sacramento, CA 95814.

Mr. Doug Edwards can be contacted via telephone at (916) 557-7026, or e-mail at Douglas.M.Edwards@usace.army.mil.

US ARMY CORPS OF ENGINEERS

Lower San Joaquin River Feasibility Study

NEPA/CEQA Scoping Meeting
January 27, 2010
6:00 - 7:30 P.M.

University of Pacific, Regent's Dining Room
3601 Pacific Avenue, Stockton, CA

Lower San Joaquin River Feasibility Study

Local Perspective

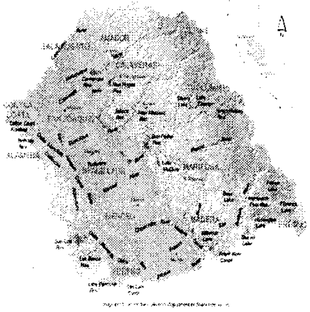
Jim Giottonini
Executive Director

San Joaquin Area Flood Control Agency
(SJAFCA)

www.SJAFCA.com

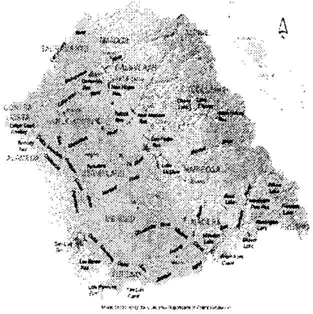
Lower San Joaquin River Feasibility Study

- Flood Damage Reduction Study with opportunities for Ecosystem Restoration
- Study cost estimate at $11Million
- Feasibility Cost Share Agreement signed on February 19, 2009
- Co-sponsors are SJAFCA, and the US Army Corps of Engineers and in the future the State and San Joaquin County Flood Control and Water Conservation District



Lower San Joaquin River Feasibility Study

- Locally we are at the "bottom of the bathtub"
- Streams and Rivers Include: Bear Creek, Mosher Slough, Calaveras River (New Hogan Reservoir), San Joaquin River, French Camp Slough (Duck Creek, and Littlejohns)
- On the west side, levees provide protection from the Delta, an infinite supply of Flood Waters
- Is there a threat of flooding? Living behind levees there is always the threat of a storm larger than the 100-year event or even a levee failure.

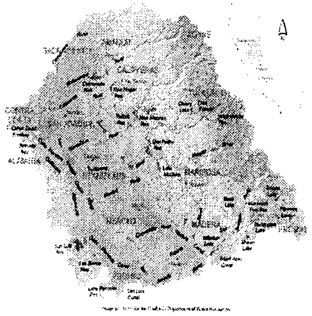


Lower San Joaquin River Feasibility Study

- Locally we are partnering with 10 Reclamation Districts including:

Mossdale (RD 17), Boggs Tract (RD 404), Rough &Ready (RD 403) Weber Tract (RD 828), Smith Tract (RD 1614) Brookside (RD 2074), Lincoln Village (1608), Shima Tract (RD 2115), Atlas Tract (RD 2126, Bishop Tract (RD 2042)

Also San Joaquin County Flood Control and Water Conservation District, and the City of Lodi.



Current Status of Levees

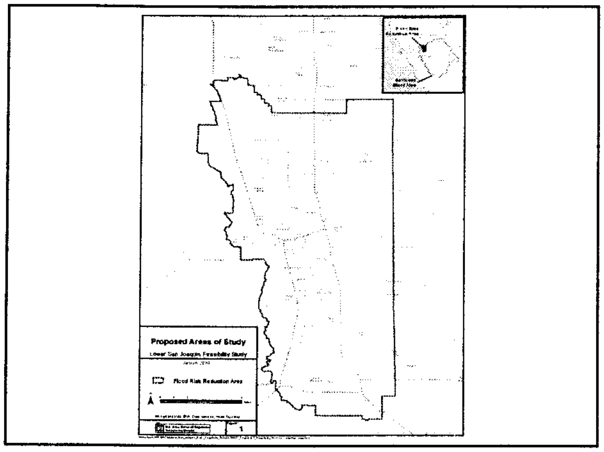
- The majority of our levees protecting urban areas meet FEMA 100-year flood protection requirements or are Provisionally Accredited by FEMA
- The levees along Smith Canal have been disaccredited by FEMA

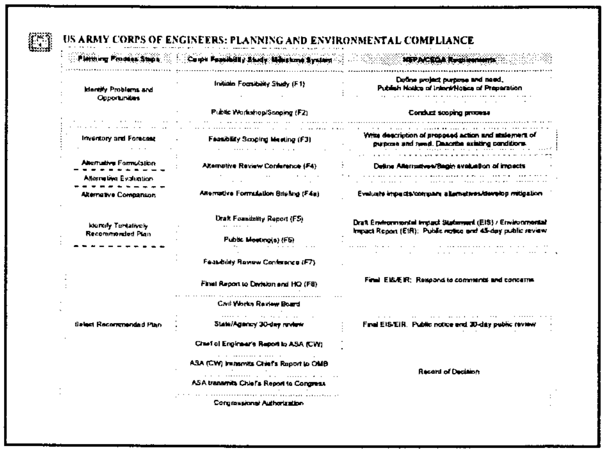
SB 5 Requirements

- New State legislation went into law in 2007.
  - Mandating 200-year level of protection for urban areas
    - Plan in place by 2015
    - Protection in place by 2025

U.S. ARMY CORPS OF ENGINEERS

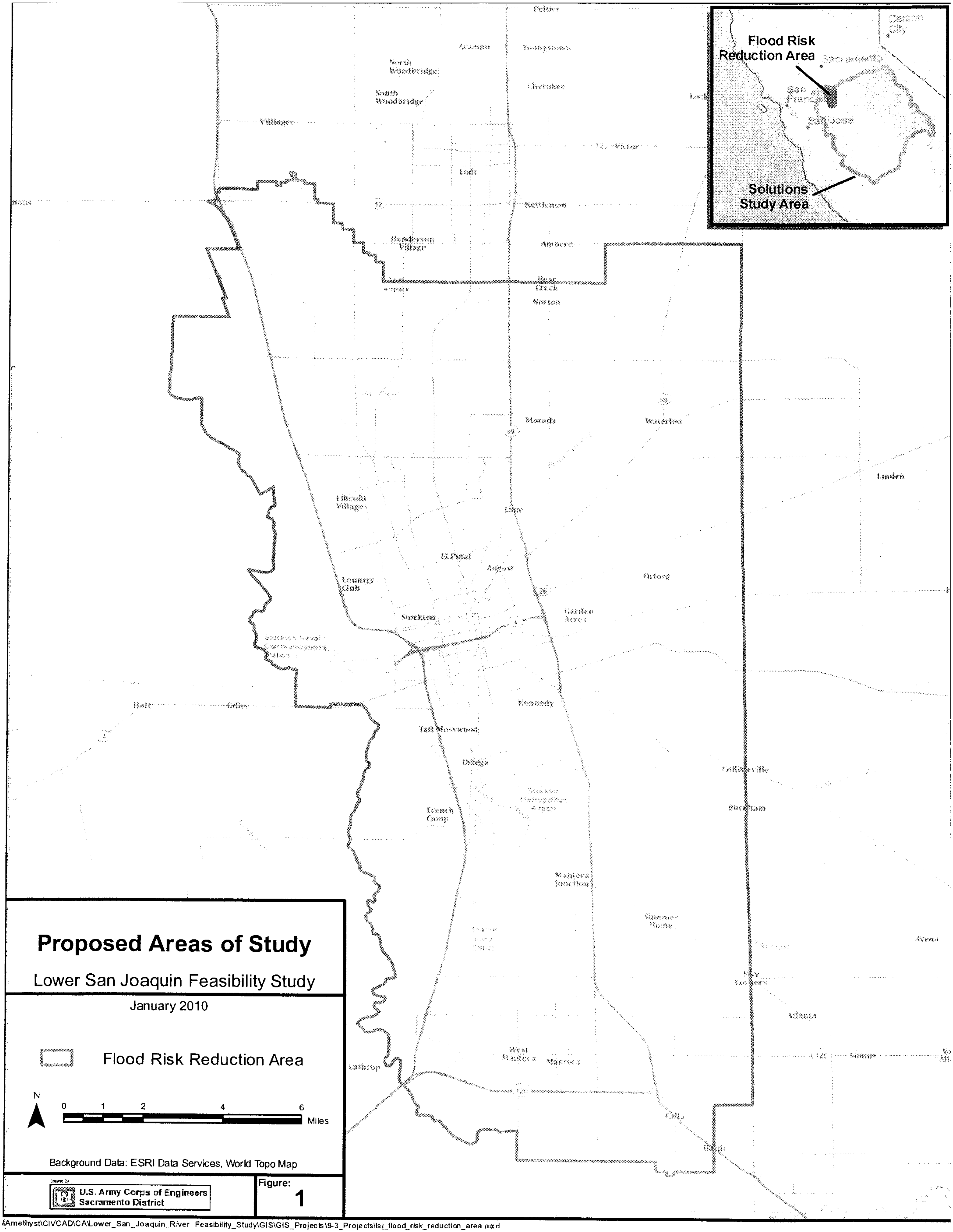
- Project Delivery Team
- Study Evolution and Milestones
- Corps Planning Process
- Environmental Compliance





US ARMY CORPS OF ENGINEERS: PLANNING AND ENVIRONMENTAL COMPLIANCE

Planning Process Steps	Corps Feasibility Study Milestone System	NEPA/CEQA Requirements
Identify Problems and Opportunities	Initiate Feasibility Study (F1)	Define project purpose and need. Publish Notice of Intent/Notice of Preparation
	Public Workshop/Scoping (F2)	Conduct scoping process
Inventory and Forecast	Feasibility Scoping Meeting (F3)	Write description of proposed action and statement of purpose and need. Describe existing conditions.
Alternative Formulation Alternative Evaluation	Alternative Review Conference (F4)	Define Alternatives/Begin evaluation of impacts
Alternative Comparison	Alternative Formulation Briefing (F4a)	Evaluate impacts/compare alternatives/develop mitigation
Identify Tentatively Recommended Plan	Draft Feasibility Report (F5) Public Meeting(s) (F6)	Draft Environmental Impact Statement (EIS) / Environmental Impact Report (EIR): Public notice and 45-day public review
	Feasibility Review Conference (F7) Final Report to Division and HQ (F8)	Final EIS/EIR: Respond to comments and concerns
	Civil Works Review Board	
Select Recommended Plan	State/Agency 30-day review	Final EIS/EIR. Public notice and 30-day public review
	Chief of Engineer's Report to ASA (CW)	
	ASA (CW) transmits Chief's Report to OMB ASA transmits Chief's Report to Congress	Record of Decision
	Congressional Authorization	



Proposed Areas of Study

Lower San Joaquin Feasibility Study

January 2010

▭ Flood Risk Reduction Area

Background Data: ESRI Data Services, World Topo Map

U.S. Army Corps of Engineers
Sacramento District

Figure: 1

\Amethyst\CIVCAD\CA\Lower_San_Joaquin_River_Feasibility_Study\GIS\GIS_Projects\9-3_Projects\lsj_flood_risk_reduction_area.mxd

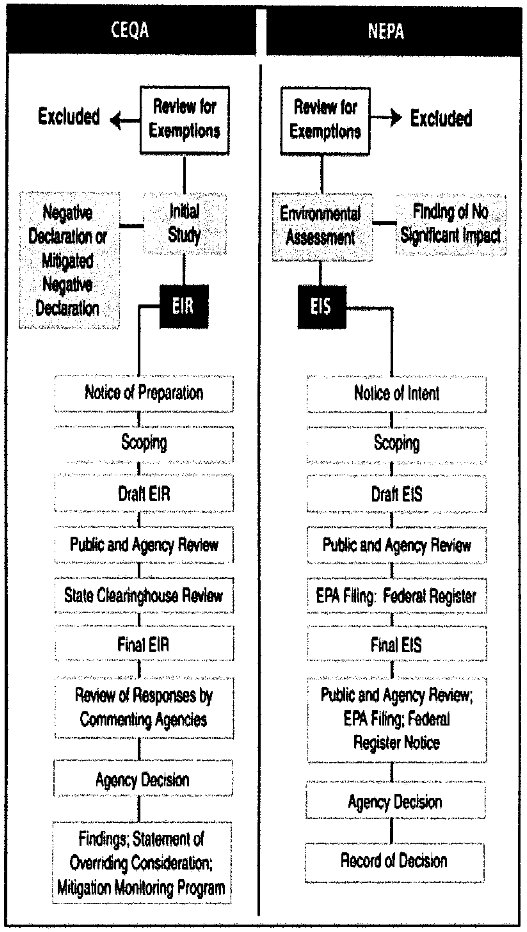
US CORPS OF ENGINEERS: NEPA/CEQA OVERVIEW

National Environmental Policy Act (NEPA): Requires that federal agencies prepare an Environmental Impact Statement (EIS) for major federal actions that may significantly affect the quality of the human and natural environment.

California Environmental Quality Act (CEQA): Requires that California state and local agencies prepare an Environmental Impact Report (EIR) for actions that significantly affect the quality of the human and natural environment.

EIS/EIR: An EIS/EIR is a public document that provides an assessment of the potential environmental impacts resulting from a proposed action and alternatives. The EIS/EIR will be comprised of the following sections:

Project Description
Project Alternatives
Existing Conditions
Environmental Consequences
Mitigation Measures



CEQA

Excluded ← Review for Exemptions

Negative Declaration or Mitigated Negative Declaration — Initial Study

EIR

Notice of Preparation
Scoping
Draft EIR
Public and Agency Review
State Clearinghouse Review
Final EIR
Review of Responses by Commenting Agencies
Agency Decision
Findings; Statement of Overriding Consideration; Mitigation Monitoring Program

NEPA

Review for Exemptions → Excluded

Environmental Assessment — Finding of No Significant Impact

EIS

Notice of Intent
Scoping
Draft EIS
Public and Agency Review
EPA Filing: Federal Register
Final EIS
Public and Agency Review; EPA Filing; Federal Register Notice
Agency Decision
Record of Decision

NEPA/CEQA FLOWCHART

Impact Analysis: Environmental resource areas to be evaluated in an EIS/EIR include, but are not limited to: air quality, biology, cultural resources, geology and soils, hazards and hazardous materials, hydrology and water quality, land use and planning, mineral resources, population and housing, public services, recreation, transportation and traffic.

Outreach: Official notification of project milestones (i.e. initiation of the study, availability of the draft and final EIS/EIR, and issuance of the record of decision) will be made through the following channels: the Federal Register, the State Clearinghouse, appropriate regional newspapers, the project mailing list, and lead agency websites.

Public Input: There are several opportunities for input regarding the project's potential to impact the quality of the human and natural environment.

Scoping Period (30 days)
Draft EIS/EIR (45 days)
Final EIS/EIR (30 days)

US ARMY CORPS OF ENGINEERS: PLANNING AND ENVIRONMENTAL COMPLIANCE

Planning Process Steps	Corps Feasibility Study Milestone System	NEPA/CEQA Requirements
Identify Problems and Opportunities	Initiate Feasibility Study (F1)	Define project purpose and need. Publish Notice of Intent/Notice of Preparation
	Public Workshop/Scoping (F2)	Conduct scoping process
Inventory and Forecast	Feasibility Scoping Meeting (F3)	Write description of proposed action and statement of purpose and need. Describe existing conditions
Alternative Formulation Alternative Evaluation Alternative Comparison	Alternative Review Conference (F4)	Define Alternatives/Begin evaluation of impacts
	Alternative Formulation Briefing (F4a)	Evaluate impacts/compare alternatives/develop mitigation
Identify Tentatively Recommended Plan	Draft Feasibility Report (F5)	Draft Environmental Impact Statement (EIS) / Environmental Impact Report (EIR): Public notice and 45-day public review
	Public Meeting(s) (F6)	
Select Recommended Plan	Feasibility Review Conference (F7)	Final EIS/EIR: Respond to comments and concerns
	Final Report to Division and HQ (F8)	
	Civil Works Review Board	
	State/Agency 30-day review	Final EIS/EIR: Public notice and 30-day public review
	Chief of Engineer's Report to ASA (CW)	Record of Decision
	ASA (CW) transmits Chief's Report to OMB	
	ASA transmits Chief's Report to Congress	
	Congressional Authorization	

US Army Corps of Engineers
BUILDING STRONG ®

SIGN IN SHEET / MAILING LIST

SCOPING MEETING: **LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY**

NAME	ADDRESS	ORGANIZATION	MAILING LIST?	
			YES	NO
Bob Hurst	4622 Saint Andrews Dr, Stockton	bob_hurst@msn.com	X	
Ellen Powell	1317 E Weber	Us Congress Cardoza	X	
Elyzabeth Hubert	1416 9th st room 1148, 95814	DWR - FESSRO	X	
John Brodie	3422 W. Hammer Ln. Ste A. 95219	SJ County RCD	X	
Michael Sabbaghian	3310 El Camino Ave, #140, Sacto	DWR - DFM		
STEVE MAYO	555 E. WEBER AVE STK 95202	SJCOG	X	
GEMMA BISCOCHO	22 E. WEBER AVE., #301, STOCKTON, CA 95202	SJAFCA		
Eric Elias	345 N. EL DORADO ST Stockton CA 95202	C.O.S. BUILDING	X	
ROGENE REYNOLDS	4444 W. UNDINE RD, STOCKTON CA 95206	LANDOWNER	X	
Max Vargas	31 E. Channel St. #306, Stockton, CA 95202	Assemblymember Galgiani's Office	✓	
Cheryle Lawson		SJAFCA		
Barry O'Regan	119 E Weber Avenue, Stockton CA	95201, Peterson Brustad	✓	
Katina Conn	San Joaquin County 1810 E Hazelton		✓	
Margit Aramburu	Pacific	Pacific	✓ elist	
RICHARD RILEY	591 N ALPINE RD STOCKTON 95215	COUNTY RESIDENT	✓	
JUAN NEIRA	22 E WEBER AVE	SJAFCA	✓	

US Army Corps of Engineers
BUILDING STRONG ®

SIGN IN SHEET / MAILING LIST

SCOPING MEETING: **LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY**

NAME	ADDRESS	ORGANIZATION	MAILING LIST?	
			YES	NO
John Giehl	844 W [illegible] Ave	—	X	
Russ Brown	630 K St Sacramento 95814	ICF International		X
Dennis Westcot	716 Valencia Ave, Davis 95616	SJRGA	Need to be X	
Jessica Ludy	1413 Pine, Martinez	American Rivers		x
WJStringfellow	3601 Pacific Ave, UOP, Sears Hall	Univ of Pacific	✓	
~~Teresa~~ Pacheco	—	W.I.I. for the Port of Stockton		
Dominick Gulli	1314 PALOMA STKN 95209	GREEN MTN ENGINEERING	✓	
John VanRyn	3422 W Hammer Lane Stockton 95219	USDA - NRCS	x	
Tom Rosten	227 Alvarado Way, Tracy CA 95376	R.D. 404	✓	

US Army Corps of Engineers
BUILDING STRONG ®

SIGN IN SHEET / MAILING LIST

SCOPING MEETING: **LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY**

NAME	ADDRESS	ORGANIZATION	MAILING LIST?	
			YES	NO
DAN Gifford	519 W. Locust St Lodi CA 95240	CDF+G	X	
W. J. Towey	6237 Embarcadero			X
Charlie Simpson	6653 Embarcadero Suite D Stk 95219	In Site Env	X	
Marlo Duncan	22 E Weber Ave #301 ~~#A~~ 95202	SJAFCA		X
Ravi Jain	3601 Pacific Ave Stockton	Univ. of the Pacific		X
Deborah Condon	2825 Watt Ave Sacramento ~~95821~~ 95211	DWR-DFM	X	
Julie Rentner	1301 L St. #4 Modesto 95354	River Partners	X	

1/27/2010

Notice of Intent to Prepare a Joint Environmental Impact Statement and Environmental Impact Report for the Lower San Joaquin River Feasibility Study

JUDITH: You came here to get information and have a presentation this evening, so I'll get on with it. First, I'm going to go through a couple of housekeeping details. My name is Judith Buethe. I'm to here facilitate the meeting this evening, and I want to thank the different parties for hosting the meeting. And in just a couple of minutes, we'll hear from the Dean of the School of Engineering. Again, this meeting is to provide information about the study that's underway. You'll hear speakers and also we'll provide the opportunity for you to provide comments that should be addressed in the environmental studies. So, it's kind of a listening evening. If you would, please hold your comments until the comment period, so we can go through the entire presentation, and if you haven't filled out a request-to-speak card, or a comment card, you still have plenty to have time to do so. Marlo back there will be happy to hand one to you. And when we do get to the comment period, I'm going to ask you to limit your comment to three minutes, so that everybody who is here has an opportunity to speak. And then if there's time at the end, and you still want to make a second comment, you'll be welcome to do that.

I want to ask, how did you hear about the meeting? First of all, how many saw an article or a notice in the newspaper? Okay. How many might have received a postcard or a letter? How many might have heard about it from a neighbor or a friend? Okay. And let's see, I also wanted to ask how many attendees are a part of the Lower San Joaquin River Feasibility Delivery Team? Okay. And how many people here are University of the Pacific students? How many are teachers or professors or deans? Okay. How many are with friends of the Calaveras River? Okay.

And I want to remind you to sign in, so that we can be sure that you're given notice as the project progresses. Before I review the agenda, there are a couple of people here that I'd like to introduce here representing their bosses. And first, representing Congressman Dennis Cardoza is Ellen Powell. Thank you, Ellen. And representing Assemblymember Cathleen Galgiani is Max Vargas. And do we have any others representing representatives of the state or national legislators? Okay. And again I remind you to sign in. That's good.

We will be starting and providing information from a variety of speakers. The first one that I'd like to introduce to you to is the Dean of the School of Engineering. He's also a professor here, Ravi Jain.

RAVI: I'm delighted that you selected this as your venue. Many people marvel at this beautiful campus. The last report said it is not as good as Harvard, but better than Yale. I wish to tell you that I have a long history with the Corps. U.S. Army Corps of Energy helped me finish my PhD. I worked with the Corps for many years. I had a secondary fellowship after my PhD. They sent me to Harvard to study public policy, public administration. I got a degree there. And then later on after five years, they gave me a year off, because I had almost spent a year extra some time working. And so they sent me to learn proper English. So I learned a great deal from many of civilians in the military officers who work with the Corps of Engineers. It's a great organization; and any time, any of your kids want to go to school and they want to study engineering or computer science, please let me know. It's a great place. If any of you are very wealthy, we have a new building. We still need only 1.4 million dollars; for equipment we need 2.5. But if you have a million you want to give to society, you have another opportunity. If you want to name the laboratory after you, that costs only half a million dollars. So it's a lovely campus, great people. You have some of the finest people working with you: Will Price. And he works very hard. He wants to make sure your meeting here is enjoyable, successful, and professionally rewarding. He has a great deal of experience in planning and, certain qualities in many other areas. He stayed back from going home just because of you today. Because usually he leaves by 5:30, 6:00 because he lives about an hour away from here. So I'm delighted. Thank you very much for inviting me and have a productive meeting. Any time you want to have any meetings here, please contact one of the faculty members because they are the key to the success of your enterprise. Thank you very much.

JUDITH: Thank you. One of the things I want to mention so you can be thinking about this, is that I mentioned an opportunity to comment, and there's several ways that you can comment. One is after the meeting, at the end of the presentation. By the way, we do have a public stenographer here who will be taking down comments. If you want to fill out one of the comment cards, and if you want to take one home with you and send it in within a few days; you're welcome to e-mail them, of course; or send a letter. So there's several different ways that you can comment. We do appreciate those comments in writing. And that's one of reasons why we have Kate here and why we provide those comment cards.

And with that, the next person I'd like to introduce is: Jim Giottonini of the San Joaquin County Area Flood Control Agency.

JIM: Thank you. I hope I can handle the PowerPoint. First, I'd like to introduce the SJAFCA staff. Well, would you please stand, Roger,___ , Cheryl and Marlo. It's a fairly small staff and you can ask any of these members for answers tonight. So please, avail yourself of that if you want to talk about the local perspective and why we're preparing this Feasibility Study.

So, basically it's a flood reduction project that we're undertaking. So, it has not only some flood benefits but restoration of all the projects. It's about 11 million dollars, plus a year project study. The cost-share agreement was signed on February 19th, 2009. Cosponsors are currently a joint powers agency between the City of Stockton, San Joaquin County and the San County Flood Control District. Many of you know it was formed in the mid '90s, during the last flood threat that we had back then. What we intend to do is as soon as the two parties have signed the cost-share agreement, we want to add the study, and we will talk about that in a few minutes. And then after, that the County Flood Control Water Conservation District will be to your [unclear]. Many of you have lived in the area and the river drains north. And then we have a lot of streams from the East. And considering that we are dealing with global warming, or whatever the term is called now, sea level rise, you know, since we're kind of lower in sea level, in some parts of your community, so that's a big threat that we also need to deal with. Not just the stream to the east of us. It's the threat of flooding. It's amazing how many people live next to a levee and don't give it a thought until maybe something breaks or something. We have the flood in '97. We evacuated Weston Ranch because the threat of flooding. So there, it's a levee. It's a levee - it's a manmade structure made of dirt. You know these things have a tendency to fail. They are designed for a hundred-year storm event. Locally -- we've partnered with reclamation districts and in the urban Stockton area, those are the areas west _________. As well, we've partnered with the City of Lodi because they have a sewer treatment plant that's kind of ___. So, so they're also interested and also the County Flood Control is a local partner. They are going to sign the cost-share agreement shortly. The majority of your levees protecting Stockton are certified by FEMA as providing hundred-year flood protection or their professional credibility, and I suggest talking about that if you want to corner one of the SJAFCA staff to talk about what a protective levee means. But the only area that has been put into the flood plain in the Stockton area by FEMA is the ______ area, and it was because of the levees on the north side of the ____ could not be certified to the FEMA standards requirements mainly because of ___ and _____ and swimming pools and all sorts of things. So it couldn't be certified. The real reason that we started the Feasibility Study is when legislation was passed. It was <his> legislation that was passed in '08. It requires us to have a plan in place to get to two-hundred-year flood protection by 2015. That's basically when we embarked on the project levees. Streams that come from the west, most of these are federal and state project levees. And so really the only way to improve those is to involve the Corps of Engineers and the state of California in order to increase the flood protection that we need because it is required by legislation. And I'm going to turn it over to Mike Sabbaghian. I'm sure I'm butchering that: Mike from the Department of Resources. I streamed over Judith. She was going to introduce him. Mike, it's all yours.

MIKE: Hello everyone. I'm not using the PowerPoint. I'm just speaking. So whatever it says on there, it's not mine. Since we live in California, we obviously have to get interested in flood protection within California to ensure that all the citizens are protected. Our effort has always continued. However, because lack of ____ is it today because the bonds that were passed in 2006. Bonds that you guys all voted for and passed, allowed us a significant amount of money -- almost five billion dollars -- to start moving some of the projects forward. Based on this, we have many flood protection measures and this Feasibility Study is one of those. Unfortunately to date, we haven't had the contract. But that's to be happening in the very near future due to several reasons. We had some issue with the bond money and we had some issues. So just because of the times and the time it takes to get some money in place. The intent to have the Feasibility Study as Jim mentioned, it is to get Federal authorization to be accomplished, and that's the key for the major projects and protection - to bring the level of protection to the point that's necessary for all their urban areas. In advance of that, the state's already funded what we call the early implementation program. We're funneling another 60 billion dollars right now. That actually started last year some in 2009 and 2008, and they're going to continue working on that in 2010 and '11, to do some improvements as it is to be completed. And we have great interest in the study for two reasons: Get the Federal authorization to actually bring the flood protection to the area; also secure the Federal credit for the money that we already put in. Both the state and local and state of California are putting money in to grow the ideas for those projects as they advance to ensure that those monies are available for future projects. One thing else I want to do: I want to introduce ______ sitting in the back. Could you stand up? He is the project manager we have for this study. He recently was promoted to supervisor of the project and supervisor _____. The last couple of weeks we hired his replacement to take another project, so he's here answering questions, and obviously I will be here to answer your questions, also. Thank you.

JUDITH: Thank you. And the next person that we would like to introduce you to . . . His name is Michael Sabbaghian: S-A-B-B-A-G-H-I-A-N. And he's with the Department of Water Resources of California. Department of Water Resources. And the next person I'd like to introduce you to is Michelle Ulm. She is the project manager for the U.S. Corps of Engineers.

MICHELLE: Before I get too far along in my introduction, I'd like to thank the University of the Pacific, the faculty. I have been told UOP is gorgeous and we mentioned in our earlier meeting that the windows are refreshing when conducting a meeting. So as Judith mentioned, my name is Michelle, and I'm the project manager for the Corps of Engineer for the Lower San Joaquin River Project Feasibility Project. Our team is, our Project Delivery Team includes your local representatives and so that includes San Joaquin County and California Department of Water Resources. Your local supporting representation districts as well as University of the Pacific. Today our Project Delivery Team is here to execute our second project milestone and that is our public scoping meeting. We are here to listen to the concerns of the locals and get your input on the study. My colleagues from the Corps of Engineers Project Study, would you raise your hand? And project Environmental Planner, Doug Edwards, as well our Senior Oversight Deputy Chief Alicia _____.

They'll be here shortly presenting the Corps of Engineers planning process. But before we get into the planning process, I'd like to share a little bit about where the study came from. Because of expressed interest to the Corps of Engineers to study the problems and opportunities, through Congressional authorization, the Corps of Engineering was able to execute ____. And that resulted in the Feasibility Study we are conducting today. Once the Feasibility Study is completed, the project will go back to Congress for authorization for design and construction. During the length of the project, the project is funded through Congressional appropriations and the local agency contributions. Let's briefly discuss our upcoming Project Delivery Team milestones. We are here today, as I mentioned, conducting our public scoping meeting. In 2012, we will have ___ our project allows us Feasibility scoping meeting. [unclear] In 2015, we will make public our draft feasibility report and all the reports for public review. And finally, in 2016, we will submit our final report to Congress for approval and authorization. Now without further ado, I give you Stacey to talk about the planning process. Thank you, Stacey.

STACEY: Thank you, Michelle. Well, as Michelle alluded to, we're at the point of the planning process where we are asking for your input for the study area as explained in this next slide. Also, what you have on your handout -- it's not real clear on the screen here-- is called the study area. The major footprint is in the area and that's pretty much a watershed area, but kind of narrows that down and kind of shortens the scope, or not shortens the scope, but just reduces the vast area that we would have to look at for opportunities and problems within this area. The team, as defined in boxed area ___ as the primary focus area for opportunities, problems and opportunities to address flood risks reduction in our study. So, we have, have this primary area that we'll be looking at to reduce economic damages from flood risk or reduction in flood risks and flood damages as a focus of this study. So, this meeting is kind of the first step in this. You have this public workshop scoping meeting. This is where, where we want to know what you folks think. Your issues, concerns are to help the team, is worked on, identifies problems and opportunities within the study area. This is an opportunity for you folks to bring things to us that we may not be aware of, and so we can look at those in the course of this study process. The next major step that we'll be looking at is the Feasibility Scoping meeting or kind of beyond that to the Alternative Formulation Briefing. But the, the Feasibility Scoping meeting that we'll do is tied down without _____ and the informal kind of the baseline where is the system now. And then we'll,… the team with input from you folks, we'll start developing alternatives and the Alternative Formulation Briefing, which is shown as F4a on this slide, is where we will present the range of alternatives that the team developed to our leadership for a policy check. The policy check is whether the alternatives can be implemented within policy, and our leadership will provide to us and we'll finalize the document. So that's the next major thing. And then the draft report, which has many alternatives, which will be the national economic development plan, the local referred plan which as Mr._____ said, basically complies with the SP five requirements and any other alternative plans or range of alternatives that the team developed over time will be presented for public review and comment. So once the document is finalized and goes out for public review and those comments are addressed, then it goes up for change for approval. And that's pretty much the planning steps in here. And so, once it goes from, you know, the Feasibility Study is finalized, it will go to our division headquarters and on up, as Michelle alluded to, back to Congress for authorization of the plan as recommended by our leadership. And with that, I'm going to turn this over to Doug Edwards who will get into the meat of why were here tonight.

DOUG: Thank you. A couple of weeks ago, I was worried that the State of the Union was going to conflict with the start of the Lost final season and there was word they were taking it off, but apparently, the President was not aware there was a scoping meeting tonight. So we're a bit ____ sometimes, that will limit the crowds, but where we have so many people out here, because it is an important opportunity to reach out and say. I'm the Environmental Planner for the Corps of Engineers. My role in this PDT meeting study is through the column on your right-hand side here and the process which is integrated with the Corps planning process. My goal tonight is to kind of give a _____ on compliance and the process and how people can participate in that process very early on. They help out on that. Again, I'm going to be rather brief up here tonight, but on the backside of the handout that this plan and the compliance matrix and of the compliance process is going to be two ____ that are going to take place with the study. On the Federal side and that's for the National Environmental Policy Act, also known as NEPA. And for that, the Corps engineers will be the lead agency and then on the state/local side will be compliance with the state of California, CEQA. That's what you hear about all the time. And really, when you boil it down, there's two main purposes for both of those: the Federal and the State Environmental Compliance Act that I just mentioned. And that's the first one, is that the agency making decisions to implement the projects needed to identify and assignment to folks. And then when, the second thing that these studies really are out there to do, is to allow the public to have an opinion in the studies that are done, before the decisions are made. Again, that's simplified and there's some difference between them, but I don't think I need to go into that at this moment. For both of those, rather than one at a time, we're going do, we've determined to do a combined document, which is the scope of the study to form environmental compliance documentation, which is called, Environmental EIR study, which is the EIS. So we're doing an EIR/EIS for this study. And if you follow along, your term allows it, and maybe these will sink in and they'll make sense to you after a while. In doing both of those compliances, we're also accomplishing other compliance acts in relationships that you've probably heard before, including the Endangered Species Act, Clean Air and Water Act, and other such acts that State and Federal agencies must comply with. Then I quickly wanted to touch on the process. And the first step in that is the scoping process. Again, we're doing this at the very early stage. Scoping is 30 days and a minimum of 30 days, and it starts with the, the Notice of Intent or Notice of Preparation in the mail. Also known as NOI/NOP. That was distributed quite widely. We wanted input. It's important. And what we're trying to do is get input, as I mentioned before. There's a variety of ways to give input, too. Tonight, read it, make sure if you don't take a moment for a written comment, you can mail those to me or to the others. Make sure that you are on the mailer list and sign in if you want to be informed as it goes along. When we did the draft EIS/EIR, it will include the alternatives that will be provided along with this study. Look at the existing conditions and then, then the area. Once that document is ____, we have a draft of that document. Again, this is important that there will be public opportunity for you folks to provide comments on that draft. And again, if you get on the mailing list now, this is available to you and you will know how it will end up and you can read it. Provide all the comments you want, and by law, we have to consider all those comments. When we first started on this study and did a draft or an, excuse me, a final document, the public has to have a minimum of 30 days to review and then issue a final EIS/EIS 30 days before they issue or record a decision on that. And on that again, that's, that's a quick overview of the environmental process. And I'd like to thank you for your time and again invite you to give comments. Judith is going to come up here and do that.

1/27/2010

Notice of Intent to Prepare a Joint Environmental Impact Statement and Environmental Impact Report for the Lower San Joaquin River Feasibility Study

FACILITATOR: Judith Buethe

JOHN BRODIE
3422 W. HAMMER LANE, SUITE A
STOCKTON, CA. 95632

Thank you. I'm here representing the San Joaquin County Resource Conservation District. We work with local agriculture producers to conserve natural resources. The district is happy to see that dredging will be one of the considerations here for some of the channels. For a couple of reasons: One, it will improve channel capacity. For another, it will help with some conveyance. We've got a lot of sedimentation issues in the Delta and water areas, and that's one thing that the district is working to help our producers solve. But along with the dredging, we hope that you will consider the use of the dredge spoils to enhance our levees and that way we can solve two problems at once: the flooding problem and then the levee stabilization problem. We know our levees are weakening. So if you consider dredging and are considering the use of dredge pipes to help solve some of those levee problems, we think that will go a long way and the District itself stands ready to be a partner in this project. We have a history of partnering with other agencies and a very good history of successful habitat restoration and enhancement projects using both state and Federal dollars, and because of agriculture we do a lot of work with private landowners. And I'm sure you're going to need help with a lot of private landowners as well. We stand ready to help with that.

JUDITH: Thank you, Mr. Brodie. And then, Klaus Garcia.

KLAUS GARCIA
6237 EMBARCADERO DRIVER
STOCKTON, CA. 95219

He pretty much covered it. No comments. I'll make a comment. Okay? So I got three minutes here. I'm kind of new here to Stockton and the project engineer said something about like she wasn't from the area either and you know we have a lot of water usage and know some of the issues were as far as environment goes has to do with wildlife and something about "ecosystem restoration measures may include but not limited to restoring riparian, wetland, and floodplain habitats, and/or constructing setback levees." So I'm not really sure what setback levees are. I know this isn't a water issue, although it is in a sense. But maybe we can just take into consideration the fact that we've already moved into an area that was wetland and swamps and dredged everything and pushed the wildlife that we had into these small areas we call sloughs, and excuse me, I'm a little nervous. So, you know, just take into consideration the fact that, that how much impact we've already made on the environment, and maybe we can mitigate some of that. Or take into account our water usage and Southern California water usage and not let the environment be harmed too much. And, or, you know, actually make something that could help the wildlife in the area. This place is going to dry up and we got to stop pushing water out of here. And I don't know if that's one of the interests--to make this a larger collection reservoir. I say build more reservoirs down South.

ANONYMOUS: One question I have is this whole environmental impact and the environmental review process. This is all just for the Feasibility Study; is that correct? What kind of environmental impact could we have just doing the study? I don't really understand how there can be such an impact when we are just studying something. I'd like to second that gentleman's statement about making the channel deeper and making the levees higher.

ANONYMOUS: Sir, sir, this is a study and although there's an effect on the environment with an environmental study, the scope of the study perhaps can be influenced by them knowing that there's people out there that will go and talk.

KLAUS GARCIA: Ask Southern California water people who are empowered by whom, by proxy, by election, take control of the situation and make things and kill this area, rape this area, 'cause that's what's going to happen. So that's why I made that comment.

JUDITH: Next commenter.

ANONYMOUS: My comment is can someone explain that please? The San Joaquin River. The San Joaquin system is a fairly stronger system, you will recall, and are you going to be looking up the environment on the San Joaquin River? You want to request that that action be taken? That's what I want to know--if that's being considered.

JUDITH: So the answer to that is yes, but we would like to stick to comments this evening, and so that comment will be considered in the environmental study.

# # #

ATTACHMENT 3

Letters/Faxes

Emails

RIVER ISLANDS
AT LATHROP

February 8, 2010

Mr. Doug Edwards
U.S. Army Corps of Engineers
Sacramento District
Attn: Planning Division (CESPK-PD-R)
1325 J Street
Sacramento, CA 95814

Subject: Comments on Notice of Intent/Notice of Preparation for the Lower San Joaquin River Feasibility Study (LSJRFS)

Dear Mr. Edwards:

We are providing you the following comments in response to USACE's Notice of Intent/Notice of Preparation (NOI/NOP) for the proposed Lower San Joaquin River Feasibility Study Draft EIS/EIR.

By way of background, I am the Project Director of the River Islands master planned community located within the City limits of Lathrop. I am also the President of Reclamation District 2062 which has a boundary coterminous with the River Islands development on the Stewart Tract. As the project developer, we own the portion of the Paradise Cut Bypass located to the west of the Union Pacific Railroad west of I-5, and we control the land in the Bypass just west of the weir on the San Joaquin River through a long term option agreement. In essence, we own and/or control the entire area within the Paradise Cut Bypass which is the only bypass located in the South County and is instrumental for diverting flood waters away from urban areas along the San Joaquin River.

River Islands has worked for years analyzing the hydrology and hydraulics in the Lower San Joaquin River area. Our consultant, MBK Engineers of Sacramento, developed a HEC-RAS hydraulic model for the area from Vernalis to Grant Line Road and the Stockton Deep Water Channel which was reviewed extensively by DWR and the USACE during the process. We have expended over $1.5 million on the modeling effort. Additionally, we have already completed significant flood protection on the River Islands project and have taken about 25% of our project area out of the 200 year flood plain through a "super-levee" program.

During the approval process for the super levees, the project and the State of California were sued by several environmental groups including Natural Resources Defense Council and the Natural Heritage Institute. We successfully settled our disputes with these entities and have mutually agreed to work together on enlarging the Paradise Cut Bypass to divert more flood flows away from the urban areas of Lathrop and Stockton. Our agreement is to work together to enlarge the bypass in order to take 20" of flood flows off the San Joaquin River at Mossdale during the 100 year storm event. The River Islands development is not dependent upon the success of this extended project but we have been working and we will continue to work in good faith with our environmental partners on this extended project.

In summary, we are surprised and disappointed that the Stewart Tract is not included in the Flood Risk Reduction Zone as proposed in the LSJRFS. We believe that the Stewart Tract must be included in the Flood Risk Reduction Zone because of the following reasons:

- The project site is completely within the Secondary Zone of the legal Delta where urban development may take place and planning entitlements (specific plan approval, zoning, etc.) for the entire project have already been obtained by the City of Lathrop. The rest of the City of Lathrop (the area east of the San Joaquin River) is already included in the Flood Risk Reduction Zone, so the current proposal is in essence bifurcating Lathrop into two halves; one in which flood risk reduction makes sense, and another that apparently the member agencies of the LSJRFS feels does not.

- The first 4,300 dwelling units, the entire mixed use Town Center and 3 million square feet of the Employment Center are within the first phase of development and are included in a vesting tentative map with all project level approvals. The first two lakes, initial grading and utilities have been provided to this area and when market conditions improve, vertical construction will begin (e.g. homes). In other words, development of the Stewart Tract is imminent and permanent.

- The entire River Islands area will contain up to 30,000 residents and 20,000 employees who will ultimately reside and work on the Stewart Tract. To not include these 50,000 persons within in the Flood Risk Reduction Zone would be sending a message that protecting this significant sum of people is not important.

- Initial construction of high ground plateaus and the 300 foot wide super-levees has already been completed as mentioned and a large part of the River Islands project already meets a minimum 200 year level of flood protection. We are the only such area so designated in the region. As the USACE is aware, all areas of the Central Valley subject to flooding and slated for urban development must meet the new 200 year flood protection standard in the near future. River Islands is the only project we are aware of that currently meets this standard.

- Paradise Cut, the only flood control bypass in the south Delta region is owned and controlled by River Islands and already has CEQA approvals for the River Islands based flood protection improvements. These improvements include setback levees, eco-system/habitat improvements and improvements near Paradise Weir that will assist in bringing Paradise Cut closer to its original design capacity and alleviate downstream

> flooding impacts to the existing urban areas of Stockton, Lathrop and Manteca. As mentioned earlier, Paradise Cut is the linchpin of an extended flood bypass project and without River Islands' participation; the LSJRFS goal of better protecting hundreds of thousands of San Joaquin County residents affected by the LSJRFS will be much more difficult to meet.

It is interesting to note that other areas along the San Joaquin River, in particular the northern portion of the Bishop Tract north of Stockton, is included in the Risk Reduction Zone. That area is outside the City of Stockton city limits and faces several hurdles for development to occur there. Why would the USACE include that area but exclude a project that has full entitlements, initial infrastructure already completed and is fully within a member city's corporate limits? 'We do understand that Paradise Cut and the Stewart Tract are included in the proposed "Study Area" for the LSJRFS. That seems appropriate. However, it seems illogical to not include the River Islands development area in the Flood Reduction Zone where 50,000 people will reside in the future.

Additionally, there are hundreds, if not thousands of acres within the planning areas of Stockton and Manteca that are not currently within the corporate limits of those cities, do not yet have project level entitlements for development, are not included in SJFCA's boundaries and yet, are included within the Flood Risk Reduction Zone. To embark on such a large regional flood protection project but exclude the Stewart Tract which is already approved for significant new population, seems shortsighted and would result in an incomplete scope for the CEQA/NEPA study.

As we have offered to SJAFCA in the past, we have significant "in-kind" work that could be helpful to your analysis. As a participant in the process, we could make available the hydraulic information that we have already gathered for the area, as well as environmental data that we have developed for Paradise Cut. Paradise Cut could also be a significant habitat resource for the environmental impacts that are certain to require mitigation in conjunction with any improvements associated with the LSJRFS. As noted, our proposals for Paradise Cut include levee setbacks, habitat restoration and the creation of a sustainable habitat for the Riparian Brush Rabbit. By not including River Islands, the USACE and other LSJRFS partners would not receive the benefit of our previous and current modeling efforts and studies and the full participation of River Islands and the inclusion of Paradise Cut within the LSJRFS.

We appreciate the opportunity to comment. We request that we be placed on the mailing list for all future documents and studies relating to the LSJRFS and all environmental documentation. Any such documentation should be forwarded to my attention at the address shown on page one of this correspondence. Should you have any questions or comments regarding our comments, please contact me at (209) 879-7900 or at sdellosso@cambaygroup.com.

Sincerely,

Susan Dell'Osso

Susan Dell'Osso
Project Director

Attachment

cc: Steve Bestolarides, SJAFCA
John Herrick, SDWA
Alex Hildebrand, SDWA
Robert Brown, Reclamation District No. 2107
Colonel Thomas Chapman, U.S. Army Corps of Engineers
Robert Charney, Department of Water Resources
Roger Churchwell, SJAFCA
Mark Connelly, San Joaquin County Public Works Department
Thomas R. Flinn, San Joaquin County Public Works Department
Dale Fritchen, SJAFCA
Jim Giottonini, SJAFCA
Karna Harrigfeld, Herum Crabtree
Al Hoslett, Reclamation District No. 2107
Cary Keaten, City of Lathrop
Manuel Lopez, Chief Administrative Officer, San Joaquin County
Diana Lowery, SJAFCA
Tom Ruark, Acting City Engineer
Larry Ruhstaller, SJAFCA
Steve Salvatore, Director of Public Works
Monty Schmitt, NRDC
Michelle Williams, U.S. Army Corps of Engineers
Scott Woodland, DWR

City of Lathrop

Office of the City Manager

390 Towne Centre Dr. – Lathrop, CA 95330
Phone (209) 941-7220 – fax (209) 941-7248
www.ci.lathrop.ca.us

February 2, 2010

Mr. Doug Edwards
U.S. Army Corps of Engineers
Sacramento District
1325 J Street
Sacramento, CA 95814
Attn: Planning Division (CESPK-PD-R)

Subject: **Notice of Intent/Notice of Preparation – Lower San Joaquin River Feasibility Study**

Dear Mr. Edwards:

The City of Lathrop is providing you the following comments in response to the Corps' Notice of Intent/Notice of Preparation (NOI/NOP) for the proposed draft EIS/EIR, regarding Lower San Joaquin River Feasibility Study.

In short, the City is concerned about the lack of the entire City of Lathrop being included with the Flood Risk Reduction Zone boundaries. The proposed boundaries "split" the City of Lathrop in half (see attached Exhibit A). Lathrop is the only urban jurisdiction in the proposed area in which this happens. Both Stockton and Manteca are wholly included within the proposed boundaries and even areas outside their City limits are included. The areas east of the City limits are rural and may not ever be developed, but there are some areas outside of City limits adjacent to the San Joaquin River that could face extensive permitting for flood control improvements.

The City respectfully requests that the entirety of the area within the City limits be included within the scope of the draft EIS/EIR and within the proposed Flood Risk Reduction area. This includes not only the City area within Reclamation District 17, but also Reclamation Districts 2062 and 2107. The Stewart Tract, in which all of Reclamation District 2062 is included, has been in the City limits of Lathrop since 1996. The entire area has been zoned for development and the first 4,300 dwelling units need no further entitlements in order to be constructed. The project developer has already embarked on their phased flood protection program and the first 900 acres of the project is flood protected to a 200 year level of protection as per State of California standards. We believe this is the only such area to achieve this designation within the Central Valley Flood Protection Board's jurisdiction.

Another request that we have is that the Study analyze improvements to Paradise Cut. While it makes sense not to include Paradise Cut in the proposed Flood Risk Reduction Zone, we do believe that this bypass will play a critical role in reducing flood risk along the San Joaquin River.

We request that the City be placed on the mailing list for all future documents and studies relating to the project. Please forward any applicable documents to:

City Manager
City of Lathrop
390 Towne Centre Drive
Lathrop, California 95330

Should you have any questions or comments regarding this letter, you may contact me at (209) 941-7220, or by email at ckeaten@ci.lathrop.ca.us.

Thank you for the opportunity to comment.

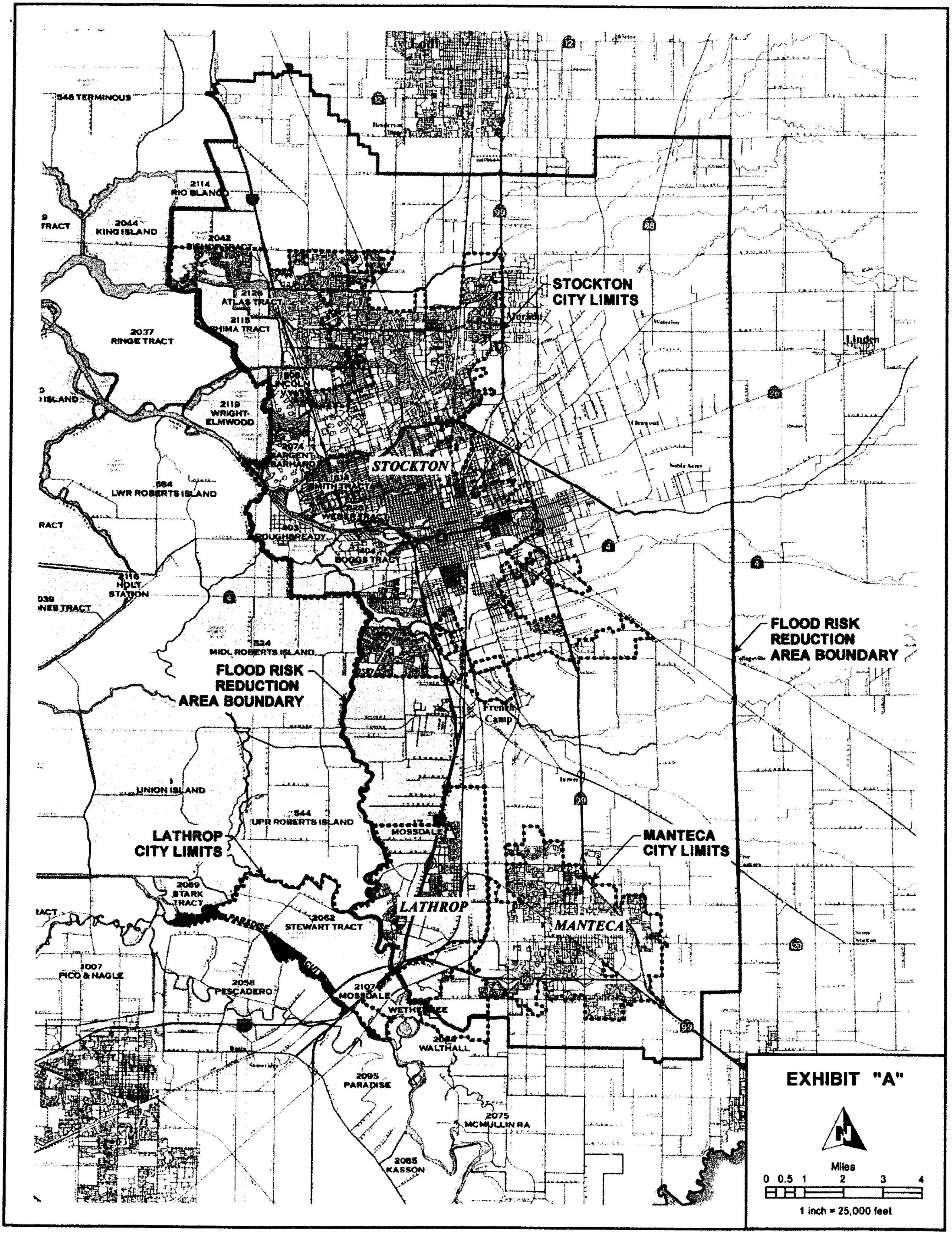
Sincerely,

Cary Keaten
City Manager

Enclosure

cc:
Mayor Kristy Sayles and the Lathrop City Council
Steve Salvatore, Director of Public Works
Tom Ruark, Acting City Engineer
Colonel Thomas Chapman, U.S. Army Corps of Engineers
Michelle Williams, U.S. Army Corps of Engineers
Robert Brown, Reclamation District No. 2107
Susan Dell'Osso, Reclamation District No. 2062
Al Hoslett, Reclamation District No. 2107
Robert Charney, Department of Water Resources
Manuel Lopez, County Administrator, San Joaquin County
Thomas R. Flinn, San Joaquin County Public Works Department
Mark Connelly, San Joaquin County Public Works Department
Steve Bestolarides, SJAFCA
Roger Churchwell, SJAFCA
Dale Fritchen, SJAFCA
Jim Giottonini, SJAFCA
Diana Lowery, SJAFCA
Larry Ruhstaller, SJAFCA
Karna Harrigfeld, Herum Crabtree

EXHIBIT "A"


FLOOD RISK REDUCTION AREA BOUNDARY
STOCKTON CITY LIMITS
FLOOD RISK REDUCTION AREA BOUNDARY
LATHROP CITY LIMITS
MANTECA CITY LIMITS
STOCKTON
LATHROP
MANTECA
Lodi
Linden
French Camp
Mossdale
548 TERMINOUS
2114 RIO BLANCO
2044 KING ISLAND
2042 BISHOP TRACT
2126 ATLAS TRACT
2115 SHIMA TRACT
2037 RINGE TRACT
1600 LINCOLN VILLAGE
2119 WRIGHT-ELMWOOD
2074 SARGENT-BARHARD
684 LWR ROBERTS ISLAND
SMITH TRACT
WEBER TRACT
403 ROUGH&READY
404 BOGGS TRACT
2116 HOLT STATION
524 MIDL ROBERTS ISLAND
1 UNION ISLAND
544 UPR ROBERTS ISLAND
MOSSDALE
2089 STARK TRACT
2062 STEWART TRACT
PARADISE CUT
1007 PICO & NAGLE
2058 PESCADERO
2107 MOSSDALE
WETHERBEE
2064 WALTHALL
2095 PARADISE
2075 MCMULLIN RA
2085 KASSON
12
99
88
26
4
120
5
205
Miles
0 0.5 1 2 3 4
1 inch = 25,000 feet
N

STATE OF CALIFORNIA
FACSIMILE COVER
10-2A-0049 (NEW 10/92)

ATTENTION: **Doug Edwards**		FROM: **Kathy Selsor** **Department of Transportation** **1976 East Charter Way** **Stockton, CA 95205**	
UNIT/COMPANY: San Joaquin County Area Flood Control Agency 1325 J Street Sacramento, CA 95814		DATE: **2/2/10**	TOTAL PAGES (Including Cover Page)
		FAX # (Include Area Code) (209) 948-7194	ATSS FAX 8-423-7194
DISTRICT/CITY		PHONE # (& Area Code) (209) 948-7190	ATSS 8-423-7190
PHONE # (& Area Code) (916) 557-7026	FAX # (& Area Code) **(916) 557-7856**	ORIGINAL DISPOSITION: Destroy ☐ Return ☐ Call for Pickup ☐	

COMMENTS:

SJ-Various
SCH# 2010012027
Lower San Joaquin River Feasibility Study

STATE OF CALIFORNIA—BUSINESS, TRANSPORTATION AND HOUSING AGENCY ARNOLD SCHWARZENEGGER, Governor

DEPARTMENT OF TRANSPORTATION
P.O. BOX 2048 STOCKTON, CA 95201
(1976 E. CHARTER WAY/1976 E. DR. MARTIN
LUTHER KING JR. BLVD. 95205)
TTY: California Relay Service (800) 735-2929
PHONE (209) 941-1921
FAX (209) 948-7194

Flex your power!
Be energy efficient!

February 2, 2010

10-SJ-Various
SCH#2010012027
SJCO. Flood Agency

Doug Edwards
San Joaquin County Area Flood Control Agency
1325 J Street
Sacramento, CA 95814

Dear Mr. Edwards:

The California Department of Transportation (Department) appreciates the opportunity to have reviewed the Notice of Preparation (NOP) for the Lower San Joaquin River Feasibility Study draft Environmental Impact Report. The Department has the following comments:

Hydraulics:
Modification, including the increase of conveyance and channel widening, to the San Joaquin River System will potentially impact adjacent and/or spanning infrastructure. Potential scour and substandard hydraulic efficiency around bridges will need to be analyzed and mitigated if necessary. Modified levees may impact the roads atop the levee or adjacent to the levee. Encroachments to the State right of way may require modifications and upgrades to drainage systems.

If you have any questions or would like to discuss our comments in more detail, please contact Kathy Selsor (209) 948-7190 (e-mail: kathy_selsor@dot.ca.gov) or me at (209) 941-1921.

Sincerely,
Kathy Selsor for
TOM DUMAS, CHIEF
OFFICE OF METROPOLITAN PLANING

c: SMorgan CA Office of Planning & Research

"Caltrans improves mobility across California"

N&B **NEUMILLER & BEARDSLEE**
A PROFESSIONAL CORPORATION • ATTORNEYS & COUNSELORS
ESTABLISHED 1903

73671-30655

John W. Stovall

509 WEST WEBER AVENUE
FIFTH FLOOR
STOCKTON, CA 95203

POST OFFICE BOX 20
STOCKTON, CA 95201-3020

(209) 948-8200
(209) 948-4910 FAX

FROM MODESTO:
(209) 577-8200
(209) 577-4910 FAX

January 25, 2010

Via U.S. Mail & Email: Douglas.M.Edwards@usace.army.mil

Mr. Doug Edwards
U.S. Army Corps of Engineers
Sacramento District
Attn: Planning Division (CESPK-PD-R)
1325 J Street
Sacramento, CA 95814

Re: LSJRFS Mailing List

Dear Mr. Edwards:

I will not be able to attend the Public Scoping Meeting o the Lower San Joaquin Feasibility Study that will be held on January 27, 2010. However, I would like to request that you add my name to the SJSRFS mailing list. Here is my contact information:

John W. Stovall
Attorney at Law
Neumiller & Beardslee
P.O. Box 20
Stockton, CA 95201-3020
Phone: (209) 948-8200
Fax: (209) 948-4910
Email: jstovall@neumiller.com

Thank you for your courtesy and cooperation.

Very truly yours,

JOHN W. STOVALL
Attorney at Law

JWS/ect

SAN JOAQUIN COUNTY
MOSQUITO & VECTOR CONTROL DISTRICT

JOHN R. STROH
MANAGER

February 1, 2010

Mr. Doug Edwards
U.S. Army Corps of Engineers, Sacramento District
Attn: Planning Division (CESPK-PD-R)
1325 J Street
Sacramento, CA 95814

San Joaquin Area Flood Control Agency
22 E. Weber Avenue, Room 301
Stockton, CA 95202-2317

Re: Joint Environmental Impact Statement and Environmental Impact Report for the Lower San Joaquin River Feasibility Study

Dear Doug Edwards:

The San Joaquin County Mosquito and Vector Control District (the District) has received the Notice of Intent to Prepare a Joint Environmental Impact Statement and Environmental Impact Report for the Lower San Joaquin River Feasibility Study. As the local agency responsible for surveillance and control of mosquitoes and mosquito-borne diseases, we strongly recommend that the EIS/EIR include an analysis of wetland development and management as it relates to 1) potential-increases in local and regional mosquito populations, and 2) promotion of mosquito-borne diseases (e.g. West Nile virus).

Seasonal and semi-permanent wetlands are a significant source of mosquitoes in San Joaquin County and the Northern San Joaquin Valley. The District's ability to respond to increased mosquito populations and mosquito-borne disease outbreaks is finite. The development of wetlands and other aquatic features capable of producing mosquitoes will need to be planned, built, and managed to prevent mosquito development.

Do not hesitate to contact me at (209) 982-4675 if you any questions or need additional information.

Sincerely,

John R. Stroh
Manager

7759 SOUTH AIRPORT WAY, STOCKTON, CALIFORNIA 95206-3918
(209) 982-4675 • FAX (209) 982-0120

California Natural Resources Agency
DEPARTMENT OF FISH AND GAME
North Central Region
1701 Nimbus Road, Suite A
Rancho Cordova, CA 95670
(916) 358-2900
http://www.dfg.ca.gov

***ARNOLD SCHWARZENEGGER**, Governor*
***John McCamman**, Director*

January 28, 2010

Doug Edwards
U.S. Army Corp of Engineers
Sacramento District, Attn: Planning Division (CESPK-PD-R)
1325 J Street
Sacramento, CA 95814

Dear Mr. Edwards:

The Department of Fish and Game (DFG) has reviewed the Notice of Preparation of a draft joint Environmental Impact Statement/Environmental Impact Report (DEIS/DEIR) for the Lower San Joaquin River Feasibility Study (project) (SCH #2010012027). The project consists of a plan to analyze environmental impacts associated with a range of alternatives for providing flood damage reduction and ecosystem restoration along the lower (northern) portion of the San Joaquin River system. Alternatives may include, but are not limited to, a combination of one or more of the following flood damage reduction measures: adding modifying and/or re-regulating storage on major tributaries, new transitory storage within flood plains, increasing conveyance by raising levees, widening channels and floodways areas, dredging and constructing or modifying weirs and bypasses, and various floodplain management measures. The project is located on the San Joaquin River and its tributaries in San Joaquin County.

Wildlife habitat resources consist of the lower San Joaquin River, its tributaries, and their associated floodplains. Significant natural resources include habitat for sensitive species of terrestrial and aquatic wildlife.

We recommend that the DEIR discuss and provide adequate mitigation for the following concerns:

1. The project's impact upon fish and wildlife and their habitat. To facilitate the environmental analysis the DEIS/DEIR should contain maps which depict the amounts and kinds of habitat that will be affected by various alternatives.

2. The project's impact upon significant habitat such as wetlands including vernal pools and riparian habitat. The project should be designed so that impacts to wetlands are avoided. Mitigation should be provided for unavoidable impacts based upon the concept of no net loss of wetland habitat values or acreage.

3. The project's impact to special status species including species which are State and Federal-listed as threatened and endangered.

4. The project's growth inducing and cumulative impacts upon fish, wildlife, water quality, and vegetation.

5. The DEIS/DEIR should provide an analysis of specific alternatives which reduce impacts to fish, wildlife, water quality and vegetation.

6. The DEIS/DEIR should contain an evaluation of the proposed projects consistency with the applicable land use plans, such as General Plans, Specific Plans, Watershed Master Plans, and the San Joaquin Multi-Species Habitat Conservation Plan.

The DEIS/DEIR should consider and analyze whether implementation of the proposed project will result in reasonably foreseeable potentially significant impacts subject to regulation by the DFG under Section 1600 et seq. of the Fish and Game Code. In general, such impacts result whenever a proposed project involves work undertaken in or near a river, stream, or lake that flows at least intermittently through a bed or channel, including ephemeral streams and water courses. Impacts triggering regulation by the DFG under these provisions of the Fish and Game Code typically result from activities that:

- Divert, obstruct, or change the natural flow or the bed, channel or bank of any river, stream, or lake;
- Use material from a streambed; or
- Result in the disposal or deposition of debris, waste, or other material where it may pass into any river stream, or lake.

In the event implementation of the proposed project involves such activities, and those activities will result in reasonably foreseeable substantial adverse effects on fish or wildlife, a Lake or Streambed Alteration Agreement (LSAA) will be required by the DFG. Because issuance of a LSAA is subject to review under the California Environmental Quality Act (CEQA), the DEIS/DEIR should analyze whether the potentially feasible mitigation measures set forth below will avoid or substantially reduce impacts requiring a LSAA from the DFG.

This project will have an impact to fish and/or wildlife habitat. Assessment of fees under Public Resources Code Section 21089 and as defined by Fish and Game Code Section 711.4 is necessary. Fees are payable by the project applicant upon filing of the Notice of Determination by the lead agency.

Pursuant to Public Resources Code Sections 21092 and 21092.2, the DFG requests written notification of proposed actions and pending decisions regarding this project. Written notifications should be directed to this office.

Thank you for the opportunity to review this project. If the DFG can be of further assistance, please contact Mr. Dan Gifford, Staff Environmental Scientist, telephone (209) 369-8851 or, Mr. Jeff Drongesen, Acting Conservation Program Manager, telephone (916) 358-2919.

Sincerely,

Jeff Drongesen
Acting Conservation Program Manager

ec. Jeff Drongesen
Dan Gifford
Department of Fish and Game
North Central Region

jdrongesen@dfg.ca.gov
dgifford@dfg.ca.gov

Ellen McBride
U.S. Fish and Wildlife Service
2800 Cottage Way, Room W2605
Sacramento, CA 92825-1888

San Joaquin River Group

- **Modesto Irrigation District**
- **Turlock Irrigation District**
- **South San Joaquin Irrigation District**
- **San Joaquin River Exchange Contractors**

716 Valencia Ave
Davis, CA 95616-0153
(530) 758-8633
(530)297-2603-Fax

- **Merced Irrigation District**
- **Oakdale Irrigation District**
- **Friant Water Authority**
- **City and County of San Francisco**

---

9 February 2010

Mr. Doug Edwards
U.S. Army Corps of Engineers
Sacramento District
Attn: Planning Division (CESPK-PD-R)
1325 "J" Street
Sacramento, CA 95814-2922

Subject: Notice of Intent to Prepare a Joint EIS/EIR for the Lower San Joaquin River Feasibility Study

Your Notice of Intent for this study is based on the need to provide 200-year flood protection for the Stockton Area. Your Notice of Intent indicates that you will be looking at a range of alternatives such as adding, modifying, and/or re-regulating storage on major tributaries to the San Joaquin River as well as widening and deepening channels in the Stockton area.

During your January 27th scoping meeting it was learned from the San Joaquin Area Flood Control Agency (SJAFCA) that the Corps would be working with DWR to model upstream flow and reservoir operations but they could not provide me with any additional information. Any study of upstream flow and reservoir operations must include all the upstream operators as the present operations are regulated by a number of agencies for a number of beneficial uses including agriculture, municipal, recreation, clean hydropower and fish management. Experience teaches that having all parties involved working together leads to more effective and lasting solutions to complex problems.

Solving ongoing problems with loss of critical habitat and dissolved oxygen problems must also be considered in the proposed study. The study must take into account the ongoing problems with salmon survival during outmigration caused in part by the previous widening and deepening of the channels in the Stockton area. The loss of critical habitat caused by the present levee system may be in part responsible for low salmon survival in the Lower San Joaquin River. In addition the periodic dissolved oxygen problems in the Stockton Deep Water Ship Channel caused by previous widening and deepening of this channel by the Corps must be eliminated before any further channel modifications should be considered.

We appreciate the opportunity to comment on the proposed study and look forward to working with the Corps in development of flood protection alternatives that enhance all the River uses. If you have any questions, please contact me.

Dennis Westcot

Dennis W. Westcot
Project Administrator
San Joaquin River Group Authority

cc: SJRGA Managers

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
REGION IX
75 Hawthorne Street
San Francisco, CA 94105

FEB 12 2010

Doug Edwards, PhD, AICP
U.S. Army Corps of Engineers
1325 J Street, Planning Division, CESPK-PD-R
Sacramento, CA 95814

Subject: Scoping Comments for the Lower San Joaquin River Feasibility Study, in the Central Valley of California.

The Environmental Protection Agency (EPA) has reviewed the Notice of Intent to prepare an environmental impact statement (EIS) for the Lower San Joaquin River Feasibility Study (LSJRFS) in the Central Valley of California. Our scoping comments are provided pursuant to the National Environmental Policy Act (NEPA), Council on Environmental Quality (CEQ) regulations (40 CFR Parts 1500-1508), and our NEPA review authority under Section 309 of the Clean Air Act.

Per our conversation on January 27th, 2010, the scope of work for this project has not yet been identified nor have any detailed alternatives been suggested. We further understand that the U.S. Army Corps of Engineers (USACE) released the LSJRFS NOI to solicit comments pertaining to the geographic area depicted in Figure #1of the NOI and its geomorphology; therefore, another NOI will be released for the LSJRFS project prior to release of a DEIS. This letter provides general comments for the LSJRFS area and does not address specific impacts that may result from actual work performed as a result of this study.

The DEIS should clearly describe the effects of the project on water quality, river flows, channel alignment, riparian and oak habitat, floodplain habitat, anadromous fish, and other sensitive species. If the project involves floodplains, there may be a risk of introducing pollutants such as mercury, boron, selenium and arsenic. We recommend the DEIS evaluate potential impacts to biota and human health and discuss strategies to avoid or reduce impacts. Methylation of mercury on periodically wetted floodplains is a particular concern, and the project will need to consider requirements that would be applicable through the Delta methylmercury TMDL (currently draft).

Of specific interest is the relationship between this proposal and other large scale planning for the Delta and San Joaquin. The DEIS should address potential direct, indirect, and cumulative effects on efforts to restore the San Francisco-San Joaquin River Bay Delta and San Joaquin River such as San Joaquin River Restoration Program and the Bay Delta Conservation Plan.

EPA advocates an integrated management approach which balances flood control, water quality, water supply, and fisheries restoration with other beneficial uses of the Lower San Joaquin River, such as wetlands, wildlife habitat and municipal water supply. The project design should give full consideration to water quality, habitat, and ecosystem functions in floodplains and riparian areas.

We recommend the DEIS include a description of climate change implications for the Lower San Joaquin River project area. For example, describe and evaluate projected climate change effects such as sea level rise and increased frequency of high intensity storms. The DEIS should consider the consequences of these effects on levees protecting Lower San Joaquin River and the proposed levee improvements. A clear evaluation of all project-related air emissions should be evaluated in the DEIS. Offsets should not be considered for air quality conformity applicability analysis.

We understand that the methods for determining flood risk, appropriate flood protection levels, and management of levee vegetation have been evolving over the years. The DEIS should provide a detailed description of the current Federal Emergency Management Agency (FEMA) floodplain management and insurance regulations, the FEMA and USACE flood risk assessment for the Lower San Joaquin River area, and USACE levee vegetation management policies. We recommend USACE contact FEMA's Region IX Mitigation Division, Map Modernization Unit to insure that the latest regulation guidelines are integrated into the DEIS. Where possible, levee setback alternatives should be considered.

We appreciate the opportunity to review this NOI. When the new NOI is released, please send a copy to our office. Likewise when the DEIS is released for public review, please send one hard copy and one CD ROM to the address above (mail code: CED-2). If you have any questions, please contact me at (415) 972-3521, or contact James Munson, the lead reviewer for this project. James can be reached at (415) 972-3800 or munson.james@epa.gov.

Sincerely,

Connell Dunning

FOR Kathleen M. Goforth, Manager
Environmental Review Office
Communities and Ecosystems Division

EPA DETAILED SCOPING COMMENTS FOR THE LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY, IN THE CENTRAL VALLEY OF CALIFORNIA

Floodplain and Riparian Habitat

Maximize restoration and enhancement of functioning floodplains and riparian habitat where feasible. EPA is especially interested in evidence that the project design gives full consideration to habitat and ecosystem functions in floodplains and riparian areas. We believe ecosystem restoration provides an excellent opportunity to enhance and restore such areas.

> ***Recommendations:***
> Options for incorporating riparian and floodplain habitat, which are important elements of river system restoration, should be addressed in the project alternatives. The Draft Environmental Impact Statement (DEIS) should also explain how monitoring and program assessments will track how floodplains and riparian areas respond to restoration actions and whether functions are being restored.

Water Quality

Water quality impairments could interfere with restoration success. Therefore, we urge development of an analytic framework and plan of action for information gathering and assessment to better target and address problems. Any water storage strategies should take into consideration water diversion practices in place that could diminish Lower San Joaquin River volume.

> ***Recommendation:***
> The DEIS should evaluate the potential effects of storage alternatives on water quality, flows, hydrology, fisheries, and riparian habitat.

Conduct analysis at a level of detail (spatial and temporal) that allows for pinpointing water quality problems and remedies. It will be important to identify in the DEIS, areas and periods of time during the year when water quality conditions could have effects on the food web or direct impacts on fish and other animals.

> ***Recommendation:***
> The DEIS should conduct analysis at a level of detail (spatial, i.e., reach-specific, and temporal) that allows for pinpointing problems and remedies. For example, describe when the presence of agricultural use chemicals may be a limiting factor for restoration goals.

From:	Edwards, Douglas M SPK
Sent:	Tuesday, January 26, 2010 1:44 PM
To:	'Taylor, Theresa J'
Cc:	Kleinsmith, Douglas H
Subject:	RE: ER 10/74
Attachments:	lsj_flood_damage_reduction_area.pdf

Theresa and Doug,

Attached is a figure showing the Flood Risk Reduction Area and Solutions Study Area for the Lower San Joaquin River Feasibility Study. As this point, we are at the front end of a feasibility study to address flooding of a large watershed. As such, the details of proposed alternative will emerge gradually over the course of the study. I will add you to the project mailing list to ensure you receive all the information as it becomes available.

Don't hesitate to contact me if you have any questions.

Doug

Doug Edwards, PhD, AICP
Senior Environmental Planner
U.S. Army Corps of Engineers
1325 J Street, Planning Division
Sacramento, CA 95814-2922
(916) 557-7026

-----Original Message-----
From: Taylor, Theresa J [mailto:TTaylor@usbr.gov]
Sent: Tuesday, January 26, 2010 1:23 PM
To: Edwards, Douglas M SPK
Cc: Kleinsmith, Douglas H
Subject: ER 10/74

Dear Mr. Edwards,

Thanks for returning my call. Attached is what we call an "Assignment Memo" that the Department of the Interior sends out to its agencies when they find a project that may be of interest. I am sending my co-worker, Doug Kleinsmith, a copy of this message as I am thinking your project is within his Regional Office boundaries. Would you please reply to both of us with the map you have? If it is too big to send, maybe we could just talk over the phone as to where the project falls. Please call me if you have any questions. I appreciate your help!

Sincerely,

Theresa Taylor

San Joaquin River Group

- **Modesto Irrigation District**
- **Turlock Irrigation District**
- **South San Joaquin Irrigation District**
- **San Joaquin River Exchange Contractors**

716 Valencia Ave
Davis, CA 95616-0153
(530) 758-8633
(530)297-2603-Fax

- **Merced Irrigation District**
- **Oakdale Irrigation District**
- **Friant Water Authority**
- **City and County of San Francisco**

9 February 2010

Mr. Doug Edwards
U.S. Army Corps of Engineers
Sacramento District
Attn: Planning Division (CESPK-PD-R)
1325 "J" Street
Sacramento, CA 95814-2922

Subject: Notice of Intent to Prepare a Joint EIS/EIR for the Lower San Joaquin River Feasibility Study

Your Notice of Intent for this study is based on the need to provide 200-year flood protection for the Stockton Area. Your Notice of Intent indicates that you will be looking at a range of alternatives such as adding, modifying, and/or re-regulating storage on major tributaries to the San Joaquin River as well as widening and deepening channels in the Stockton area.

During your January 27th scoping meeting it was learned from the San Joaquin Area Flood Control Agency (SJAFCA) that the Corps would be working with DWR to model upstream flow and reservoir operations but they could not provide me with any additional information. Any study of upstream flow and reservoir operations must include all the upstream operators as the present operations are regulated by a number of agencies for a number of beneficial uses including agriculture, municipal, recreation, clean hydropower and fish management. Experience teaches that having all parties involved working together leads to more effective and lasting solutions to complex problems.

Solving ongoing problems with loss of critical habitat and dissolved oxygen problems must also be considered in the proposed study. The study must take into account the ongoing problems with salmon survival during outmigration caused in part by the previous widening and deepening of the channels in the Stockton area. The loss of critical habitat caused by the present levee system may be in part responsible for low salmon survival in the Lower San Joaquin River. In addition the periodic dissolved oxygen problems in the Stockton Deep Water Ship Channel caused by previous widening and deepening of this channel by the Corps must be eliminated before any further channel modifications should be considered.

We appreciate the opportunity to comment on the proposed study and look forward to working with the Corps in development of flood protection alternatives that enhance all the River uses. If you have any questions, please contact me.

Dennis Westcot

Dennis W. Westcot
Project Administrator
San Joaquin River Group Authority

cc: SJRGA Managers

Edwards, Douglas M SPK

From:	Edwards, Douglas M SPK
Sent:	Tuesday, January 19, 2010 1:00 PM
To:	'Munson.James@epamail.epa.gov'
Subject:	FW: San Joaquin River Feasibility Study
Attachments:	lsj_flood_damage_reduction_area.pdf

James,

Please see attached and don't hesitate to contact me if you have any questions.

Doug

Doug Edwards, PhD, AICP
Senior Environmental Planner
U.S. Army Corps of Engineers
1325 J Street, Planning Division
Sacramento, CA 95814-2922
(916) 557-7026

-----Original Message-----
From: Edwards, Douglas M SPK
Sent: Tuesday, January 19, 2010 10:13 AM
To: 'Munson.James@epamail.epa.gov'
Subject: RE: San Joaquin River Feasibility Study

Hi James,

Got your voice mail and email. I'm working on getting an electronic version of the map and will send it to you as soon as possible.

Doug

Doug Edwards, PhD, AICP
Senior Environmental Planner
U.S. Army Corps of Engineers
1325 J Street, Planning Division
Sacramento, CA 95814-2922
(916) 557-7026

-----Original Message-----
From: Munson.James@epamail.epa.gov [mailto:Munson.James@epamail.epa.gov]
Sent: Tuesday, January 19, 2010 8:44 AM
To: Edwards, Douglas M SPK
Subject: San Joaquin River Feasibility Study

Hi Mr. Edwards,

I also left a message regarding the San Joaquin River Feasibility Study.
The summary references a “Figurer #1”. Is this a map?

We could really use any information you have in the way of maps or coordinates to define the project area.

Thanks,

James Munson
Environmental Protection Specialist
Environmental Review Office
U.S. EPA, Region IX
75 Hawthorne Street
San Francisco, Ca 94105
(415)972-3800, Fax:(415)947-3562

From:	Edwards, Douglas M SPK
Sent:	Tuesday, January 26, 2010 1:58 PM
To:	'Christine Joab'
Subject:	RE: Public Scoping Meeting for the LSJRFS and EIS/EIR
Attachments:	lsj_flood_damage_reduction_area.pdf

Christine,

Attached is a figure showing the Flood Risk Reduction Area and Solutions Study Area for the Lower San Joaquin River Feasibility Study. As this point, we are at the front end of a feasibility study to address flooding of a large watershed. As such, the details of proposed alternative will emerge gradually over the course of the study. I will add you to the project mailing list to ensure you receive all the information as it becomes available.

Don't hesitate to contact me if you have any questions.

Doug

Doug Edwards, PhD, AICP
Senior Environmental Planner
U.S. Army Corps of Engineers
1325 J Street, Planning Division
Sacramento, CA 95814-2922
(916) 557-7026

-----Original Message-----
From: Christine Joab [mailto:CJoab@waterboards.ca.gov]
Sent: Thursday, January 21, 2010 6:49 PM
To: Edwards, Douglas M SPK
Cc: Christine Joab
Subject: Public Scoping Meeting for the LSJRFS and EIS/EIR

Mr. Edwards,

My name is Christine Joab and I am one of the Central Valley Regional Water Quality Control Board (Central Valley Water Board) staff that is working on a Total Maximum Daily Load (TMDL) in the lower San Joaquin River. I saw the meeting announcement for the Public Scoping Meeting for the Lower San Joaquin River Feasibility Study and EIS/EIR posted on the Delta News Newsletter.

I am interested in being added on the mailing list for this project. Staff from the Central Valley Water Board's TMDL unit will not be available to attend this meeting but we (Dissolved Oxygen, Pesticides, and Mercury TMDL staff) are interested in knowing more about this feasibility Study and EIS/EIR. Is there anyway we can get a copy of the presentation that you will be presenting at the Public Scoping Meeting or at least some other documentation outlining your alternatives?

Thanks.

Christine Joab
Environmental Scientist
San Joaquin TMDL & NPS Unit
Regional Water Quality Control Board,
Central Valley (Region 5 Sacramento)
11020 Sun Center Dr., Ste 200
Rancho Cordova, CA 95670-6114
Phone: (916) 464-4655
Fax: (916) 464-4800

From: Edwards, Douglas M SPK
Sent: Thursday, January 21, 2010 1:12 PM
To: 'kathy_selsor@dot.ca.gov'
Subject: LSJRFS Figure 1
Attachments: lsj_flood_damage_reduction_area.pdf

Attached is the figure you requested via phone today. Let me know if you have any questions.

Doug

Doug Edwards, PhD, AICP
Senior Environmental Planner
U.S. Army Corps of Engineers
1325 J Street, Planning Division
Sacramento, CA 95814-2922
(916) 557-7026

From:	Trujillo, Elvia [etrujillo@neumiller.com]
Sent:	Monday, January 25, 2010 4:19 PM
To:	Edwards, Douglas M SPK
Cc:	Stovall, John
Subject:	Lower San Joaquin Feasibility Study
Attachments:	LSJRFS Mailing List.pdf

Mr. Edwards:

Attached you will find a letter from John Stovall requesting that you please add his name to the LSJRFS mailing list. If you should have any questions, please don't hesitate to call.

Thank you,

Elvia C. Trujillo

Assistant to

John W. Stovall

Neumiller & Beardslee
P.O. Box 20
Stockton, CA 95201-3010
(209) 948-8200 Phone
(209) 948-4910 Fax
etrujillo@neumiller.com

N&B **NEUMILLER & BEARDSLEE**
A PROFESSIONAL CORPORATION • ATTORNEYS & COUNSELORS
ESTABLISHED 1903

John W. Stovall

509 WEST WEBER AVENUE
FIFTH FLOOR
STOCKTON, CA 95203

POST OFFICE BOX 20
STOCKTON, CA 95201-3020

(209) 948-8200
(209) 948-4910 FAX

FROM MODESTO:
(209) 577-8200
(209) 577-4910 FAX

73671-30655

January 25, 2010

Via U.S. Mail & Email: Douglas.M.Edwards@usace.army.mil

Mr. Doug Edwards
U.S. Army Corps of Engineers
Sacramento District
Attn: Planning Division (CESPK-PD-R)
1325 J Street
Sacramento, CA 95814

Re: LSJRFS Mailing List

Dear Mr. Edwards:

I will not be able to attend the Public Scoping Meeting o the Lower San Joaquin Feasibility Study that will be held on January 27, 2010. However, I would like to request that you add my name to the SJSRFS mailing list. Here is my contact information:

John W. Stovall
Attorney at Law
Neumiller & Beardslee
P.O. Box 20
Stockton, CA 95201-3020
Phone: (209) 948-8200
Fax: (209) 948-4910
Email: jstovall@neumiller.com

Thank you for your courtesy and cooperation.

Very truly yours,

JOHN W. STOVALL
Attorney at Law

JWS/ect

Environmental Protection Specialist

Bureau of Reclamation

Policy and Administration

Mailstop: 84-55000

PO Box 25007

Denver, CO 80225

303-445-2806 (office)

303-445-6683 (FAX)

ttaylor@usbr.gov

From: oepchq@ios.doi.gov [mailto:oepchq@ios.doi.gov]
Sent: Thursday, January 21, 2010 3:44 PM
To: Sutton, Loretta B; Rai, Vijai N; Taylor, Theresa J; Stephanie Nash; Dickey, Marchelle; Johnson, Brenda J; Lecain, Gary D; Morlock, Dale; Kraus, Victoria; Treasure, Donald W; Singleton, Ellen; Meyer, Elizabeth A; Wilson, Judith; Carriero, Joe; Runkel, Roxanne; Demarest, Chip; Perez, John A; Port, Patricia; oepcsfn@aol.com
Subject: ENVIRONMENTAL REVIEW (ER) NEW POSTING NOTIFICATION: ER 10/74

This e-mail alerts you to an ER request from the Office of Environmental Policy and Compliance (OEPC). To access electronic ERs visit the OEPC Natural Resources Management Team website at: http://www.doi.gov/oepc/nrm.html <http://www.doi.gov/oepc/nrm.html%20> Under Quick Links select: Environmental Review Distributions (Bureau ER Notifications). For assistance, please contact the Natural Resources Management Team, at 202-208-5464.

Edwards, Douglas M SPK

From: Nicky Suard [sunshine@snugharbor.net]
Sent: Friday, January 22, 2010 8:23 AM
To: Edwards, Douglas M SPK
Subject: San Juaquin River eir/eis

The notice refers to Figure 1, which is apparently a map showing the area of the study. Can you email me the draft map please?

Thank you.

Nicole (Nicky) Suard, Esq

Edwards, Douglas M SPK

From: Edwards, Douglas M SPK
Sent: Tuesday, January 26, 2010 1:55 PM
To: 'Nicky Suard'
Subject: RE: San Juaquin River eir/eis

Attachments: lsj_flood_damage_reduction_area.pdf

lsj_flood_damage_r
eduction_are...

Please see attached and don't hesitate to contact me if you have any questions.

Doug

Doug Edwards, PhD, AICP
Senior Environmental Planner
U.S. Army Corps of Engineers
1325 J Street, Planning Division
Sacramento, CA 95814-2922
(916) 557-7026

-----Original Message-----
From: Nicky Suard [mailto:sunshine@snugharbor.net]
Sent: Friday, January 22, 2010 8:23 AM
To: Edwards, Douglas M SPK
Subject: San Juaquin River eir/eis

The notice refers to Figure 1, which is apparently a map showing the area of the study. Can you email me the draft map please?

Thank you.

Nicole (Nicky) Suard, Esq

From:	Charlie Simpson [csimpson@insite-env.com]
Sent:	Thursday, January 28, 2010 10:07 AM
To:	Edwards, Douglas M SPK
Cc:	Vicki Jordan
Subject:	Lower San Joaquin River Feasibility Study

Hi Doug,

It was a pleasure to meet you last night at UOP. Best of luck on the project, and I'll look forward to seeing you again at future events.

My firm, InSite Environmental, has done literally hundreds of urban development and other projects within the study area (the smaller area, Stockton, Lathrop, Manteca), many of them adjacent to waterways, including Smith Canal. If I or InSite may be of assistance as the LSJRFS process unfolds, even informally, I would be pleased to be able to do so.

Again, best wishes.

Charlie

Charlie Simpson, Principal
InSite Environmental, Inc.
6653 Embarcadero Drive, Suite Q
Stockton, CA 95219
209-472-8650
csimpson@insite-env.com <mailto:csimpson@insite-env.com>

From: William Luce [wluce@friantwater.org]
Sent: Tuesday, February 02, 2010 6:55 AM
To: Edwards, Douglas M SPK
Subject: Notice of Intent to Prepare a Joint Environmental Impact Statement and Environmental Impact Report for the Lower San Joaquin River Feasibility Study

Dear Mr. Edwards:

Would you please add me to the mailing list for the Lower San Joaquin River Feasibility Study?

Thank you.

Bill Luce, P.E., Resources Manager

Friant Water Authority

1974 N. Gateway Blvd., Suite #104

Fresno, CA 93727

Office: 559-562-6931

Cell: 559-802-0091

Fax: 559-562-6308

Email: wluce@friantwater.org <mailto:wluce@friantwater.org>

Edwards, Douglas M SPK

From: Richard Riley [rrly951@yahoo.com]
Sent: Wednesday, February 03, 2010 9:49 AM
To: Edwards, Douglas M SPK
Subject: Requested Comments-Lower San Joaquin River Feasibility Study

Thank-You for holding the meeting January 27. Thanks also goes to the University of Pacific for a very nice location for the meeting.
I am just an interested county resident that attended the meeting. I have no true direction to give you. I was overwhelmed by the scope of the area under discussion. It is more to the East, when I thought it would be more to the West. It would have been very beneficial to individuals like myself if someone would have given more examples of what other studies like this one included. I still do not understand what is truly involved. My knowledge of what the U.S. Army Corps of Engineers does, amount of authority with such projects, etc is very limited.
All I can add is that you should hold another presentation at a public building more the center of the propsed study area. Most residents have no idea about all of this.

Sincerely,
Richard Riley
Stockton

Edwards, Douglas M SPK

From: Jherrlaw@aol.com
Sent: Monday, February 08, 2010 4:14 PM
To: Edwards, Douglas M SPK
Cc: sdellosso@cambaygroup.com
Subject: Comment LSJR Feasibility Study

Attachments: JH to Corps re River Islands 2-15-10.pdf

JH to Corps re River Islands 2...

Please see attached

JOHN HERRICK, Esq.
4255 Pacific Ave. Ste. 2
Stockton, CA 95207
(209) 956-0150 ph
(209) 956-0154 fax

SOUTH DELTA WATER AGENCY
4255 PACIFIC AVENUE, SUITE 2
STOCKTON, CALIFORNIA 95207
TELEPHONE (209) 956-0150
FAX (209) 956-0154
E-MAIL Jherrlaw@aol.com

Directors:
Jerry Robinson, Chairman
Robert K. Ferguson, Vice-Chairman
Natalino Bacchetti
Jack Alvarez
Mary Hildebrand

Engineer:
Alex Hildebrand
Counsel & Manager:
John Herrick

February 15, 2010

Mr. Doug Edwards
U.S. Army Corps of Engineers
Sacramento District
Attn: Planning Division (CESPK-PD-R)
1325 J Street
Sacramento, CA 95814

Re: Comments on Notice of Intent/Notice of Preparation for the Lower San Joaquin RiverFeasibility Study (LSJRFS)

Dear Mr. Edwards:

On behalf of the South Delta Water Agency I would like to add our support and endorsement to River Islands' request that its project be included in the Flood Reduction Zone. As stated by Ms. Dell'Osso, the Project Director of River Islands, their proposed improvements to Paradise Cut are a necessary part of any overall solution to San Joaquin River flood control.

Paradise Cut is the designed overflow channel for high San Joaquin River flows. Unfortunately, over the years it has degraded to the point where it will no longer hold its originally designed capacity. River Islands proposes to restore much of this capacity, while at the same time significantly increasing habitat. River Islands has also worked with SDWA on additional plans to improve flood flow capacity throughout the area, and is an eager partner to actually begin work.

As Ms. Dell'Osso also stated, River Islands' has already done significant engineering and modeling on its and other local flood control proposals. This information is not just a starting point for analyzing local issues, but the basis on which solutions will rest. It is inconceivable to us that River Islands' project for improvements to Paradise Cut would not be included in your Flood Reduction Zone analysis or as part of your Feasibility Study. We strongly encourage you to include it.

/ / / /

Please feel free to contact me if you have any questions.

Very truly yours,

JOHN HERRICK

cc: Susan Dell'Osso

From: Dennis Westcot [westcot-sjrga@sbcglobal.net]
Sent: Tuesday, February 09, 2010 2:36 PM
To: Edwards, Douglas M SPK
Cc: amontgomery@waterboards.ca.gov
Subject: NEPA/CEQA Scoping Meeting

Attachments: Final Letter to US Army Corps on Lower SJR.pdf

Final Letter to US Army Corps ...

Mr. Edwards:

Attached are comments from the San Joaquin River Group Authority on the NEPA/CEQA Scoping work being done on providing 200-year flood protection as part of the Lower San Joaquin River Feasibility Study. A signed copy will follow in the mail.

If you have any questions, please call me.

Dennis W. Westcot
Project Administrator
San Joaquin River Group Authority
716 Valencia Ave.
Davis, CA 95616-0153
Phone: (530) 758-8633
FAX: (530) 297-2603
e-mail: westcot-sjrga@sbcglobal.net

APPENDIX A-2

DRAFT FISH AND WILDLIFE COORDINATION ACT REPORT (DRAFT CAR)

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

United States Department of the Interior

FISH AND WILDLIFE SERVICE
Sacramento Fish and Wildlife Office
2800 Cottage Way, Suite W-2605
Sacramento, California 95825-1846

In Reply Refer to:
08ESMF00-
2014-CPA-0012

JUN 24 2014

Ms. Alicia Kirchner
Chief, Planning Division
Sacramento District
U. S. Army Corps of Engineers
1325 J Street
Sacramento, California 95814

Subject: Transmittal of draft Fish and Wildlife Coordination Act report for the Lower San Joaquin River Feasibility Study

Dear Ms. Kirchner:

Please find enclosed for your review the U.S. Fish and Wildlife Service's draft Fish and Wildlife Coordination Act report for the Lower San Joaquin River Feasibility Study. By copy of this letter, we also request review and comment by the National Marine Fisheries Service and California Department of Fish and Wildlife.

If you have any questions, please contact Steven Schoenberg of my staff at (916) 414-6564.

Sincerely,

Daniel Welsh

Daniel Welsh
Acting Field Supervisor

Enclosure

cc:
Tanis Toland, COE, Sacramento, CA
Michael Hendrick, NMFS, Sacramento, CA
Tom Kelley, USEPA, San Francisco, CA
John Kleinfelter, CDFW, Sacramento, CA
Eva Olin, CDFW, Stockton, CA
Ruth Darling, DWR, Sacramento, CA
Juan Neira, SJAFCA, Stockton, CA

UNITED STATES DEPARTMENT OF THE INTERIOR

FISH AND WILDLIFE SERVICE

DRAFT FISH AND WILDLIFE COORDINATION ACT REPORT FOR THE LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

PREPARED BY:

Steven Schoenberg, Senior Fish and Wildlife Biologist
U.S. Fish and Wildlife Service
Habitat Conservation Division
Sacramento Fish and Wildlife Office
Sacramento, California

PREPARED FOR:

U.S. Army Corps of Engineers
Sacramento District
Sacramento, California

June 2014

DRAFT FISH AND WILDLIFE COORDINATION ACT REPORT FOR THE LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

INTRODUCTION

This document constitutes the Fish and Wildlife Service's (Service) draft detailed report on the U.S. Army Corps of Engineers (Corps) Lower San Joaquin River Feasibility Study. The Lower San Joaquin River Feasibility Study covers a region that includes the communities of Stockton, Lathrop, and Manteca, where there is a significant risk associated with flooding. The study has planning objectives to reduce flood risk, develop plans to address and communicate residual flood risks, and develop ecosystem restoration or enhancement features coincident with flood risk management.

There has been no prior formal (signed) coordination document prepared by the Service on the Feasibility Study. Prior involvement included attendance at one kick-off meeting in 2009 and a 2-day site visit on May 29-30, 2013. On August 15, 2013, we provided a staff-level Planning Memorandum outlining the potential effects of the project in the north central, and south areas of the project, based on the 2013 site visit, and prior involvement on a separate, Section 408 proposal known as the Reclamation District (RD) 17 Phase III Seepage Area Project; this project overlaps one of the elements of the Feasibility Study, but differs in the extent and types of work (Service 2013). A formal coordination document was not prepared for the 408 project due to lack of Corps funding. However, we did attend a site visit in March 2011, and submitted a comment letter on the Draft Environmental Impact Statement/Report (DEIS/R) for the RD 17 Phase III Seepage Area Project, which we appended to the 2013 Planning Memorandum for this project (Service 2011).

Coordination between the Service and other resource agencies has thus far has been limited to brief initial contact with the National Marine Fisheries Service (NMFS) and the Environmental Protection Agency. These agencies had previously commented on the RD 17 Phase III project as well in 2011. The California Department of Fish and Wildlife (CDFW) was contacted regarding their general guidance on mitigation for urban trees, but not specifically on the Feasibility Study. The limited nature of our coordination effort reflects: (a) the uncertain nature of project alternatives, which underwent repeated revision - culminating in a final array of alternatives received June 3, 2014, (b) the limited coordination funding the Service received from the Corps, and (c) lack of information on habitat quantities impacted by project alternatives (partial information provided May 30, 2014).

Information considered in this report includes observations during site visits, draft materials transmitted informally for our use by the Corps on this project (e.g., image files, tables, powerpoint presentations, draft narratives for the Feasibility Study), the DEIS/R for the RD 17 Phase III Seepage Project (Corps and RD 17 2011), and other materials in our files.

PROPOSED PROJECT

The Corps proposes to use a variety of structural approaches (termed "measures") used alone or in combination at different locations. Below, we describe these measures, how they are applied to each potential project element, and then the combinations of elements which form the alternatives.

Structural Measures:

Cutoff Walls: This measure is used to address seepage issues. Sites are cleared, grubbed, and the levee is degraded at least one half its height. A 3-foot minimum width trench is excavated to impermeable soil (variable depth) and filled with bentonite slurry during this excavation; after the slurry has cured, an impervious cap is installed. Finally, the levee is reconstructed.

Slope Reshaping: This measure is used to restore levees to Corps design criteria for sideslope and crown width. It is done by clearing and grubbing the waterside crest, crown, and landside slope, removing at least the top 2 feet of material. Suitable material is then placed on the landside and the slope shaped to meet Corps design criteria.

Levee Raise: For this study, this raising measure would be done to achieve 200-year protection and/or for sea level rise protection, depending on location. The work is done the same way as with slope reshaping, except that after suitable material is placed, the levee is rebuilt to a greater specified height.

Seepage Berm: This measure is also used to address seepage issues. It is a berm built on the landside of the levee, usually ranging from 150-200 feet wide, and is 3-5 feet high. Construction involves clearing and grubbing, and placement of successive layers of sand, gravel and soil (with filter fabric between the gravel and soil).

New Levee: This measure is used to reduce flood risk of outflanking, or as an alternative to repairing existing levees by setting the levee back. Construction involves clearing and grubbing, then excavating an inspection trench. Material is placed, watered, and compacted, then shaped to design specifications. A cutoff wall is also installed, if determined by inspection to be needed. Slopes are then armored with stone riprap as needed, and the remainder reseeded with grasses to prevent erosion.

Erosion Protection: This measure applies to areas which could be subject to high flows, tides, or wave action, during large events; and includes areas of the Delta Front and RD 17 work elements (see below), depending on alternative. It entails placing stone riprap on the entire slope of the levee from toe to crown.

Seismic Remediation: Used to reduce deformations during earthquakes, this measure involves installing a grid of soil-cement mix columns. It requires clearing, grubbing, and degrading the levee to one-half its height. The columns are created using a deep soil mixing auger, and then the levee is reconstructed with suitable material.

Closure Structure: This measure is used at the mouths of Smith Canal and Fourteenmile Slough. It involves installing a gate panel that is raised and lowered by rubber air bladders. This structure is attached to a concrete foundation. As needed, a sheet pile floodwall is installed to tie the structure into adjacent levee or high ground area.

Control Structure and Bypass Channel: This measure would allow use of the old Mormon Slough, or "Mormon Channel," as a flood bypass which runs from the Stockton Diverting Canal to the San Joaquin River. By taking off the peak of flood flows, this measure is an alternative to improvements along the Stockton Diverting Canal and portions of the lower Calaveras River. To do this, a box culvert with a 12-foot-high radial gate would be installed where Mormon Channel meets the Stockton Diverting Canal. There are a number of low-water crossings which need to be removed or replaced with bridges, some channel widening, and several culvert modifications. It is designed to carry 1,200 cubic feet per second at most, and would be operated not more than every 2 years or so. The amount of flow and duration varies with the size of the event but it would be intermittent, flowing a few days, every few years. There may be other necessary work in Mormon Channel, such as remediation of any contaminants present in the slough, and restoration actions involving earthwork or plantings, but these have not yet been described.

Work Elements:

The Corps developed work elements described by location, with individual or combinations of structural measures in reaches, or collections of reaches, as follows:

Mosher Slough: In Mosher Slough, the Corps would use cutoff walls and levee raise as needed for sea level rise protection.

Delta Front (Shima Tract, Fivemile, Tenmile, and Fourteenmile Sloughs): In these reaches, the Corps would install cutoff walls and place erosion protection. Fourteenmile Slough would also receive seismic protection and a closure structure. Slope reshaping would be used in all of Tenmile and portions of Fourteenmile Slough. One section in the vicinity of Buckley Cove Marina would have slope reshaping and a seismic fix.

Smith Canal: A closure structure between Brown's Island and Dad's Point, and a short floodwall would be built (see also above, Structural Measures).

Calaveras River: Cutoff walls and some slope reshaping would be used for the north bank between the San Joaquin River and Cherryland Avenue, as well as for the south bank between the San Joaquin River and the Stockton Diverting Canal. The extent of this work is reduced in Alternative 9b compared to Alternatives 8a and 8b (see Alternatives, below).

Stockton Diverting Canal: Cutoff walls would be installed in the entire south levee between old Mormon Slough and the Calaveras River.

Mormon Channel: Work in this reach involves a control structure and other work, as described above (see Structural Measures).

San Joaquin River: This term applies to installation of cutoff walls on the right bank of the San Joaquin River from Burns Cutoff extending south and east to the north bank of French Camp Slough near Horton Avenue, and on a separate section from 2,100 feet upstream of the Calaveras River to the Smith Canal Closure Structure (this latter section would also be raised). Some slope reshaping would also be done in Burns Cutoff. For Alternative 8a, this element would be extended east by the construction of a section of about a mile of new levee on Duck Creek between French Camp Slough to the rail yard.

RD 17: This element involves various measures applied to levee sections bordering RD 17, beginning at the south bank of French Camp Slough 600 feet southeast of Carolyn Weston Boulevard, continuing south along the right bank of the San Joaquin River to Lathrop Road, and turning east at the southern end of the existing tie back levee. It involves cutoff walls along French Camp Slough, a large section of cutoff walls and slope reshaping along the mainstem San Joaquin River, significant sections of seepage berms, levee reshaping, raising certain sections near Stewart Tract, and a new setback levee section in the vicinity of Old River. For the southernmost, east-west dryland section of levee, this levee would be extended east by a new levee section, the existing levee would be raised, and both existing and new levees would receive erosion protection.

Alternatives:

The alternatives include a No Action alternative (Alternative 1), and six action alternatives consisting of implementing different combinations of the various elements using structural measures, shown below:

Alternative 1: No Action. Under this alternative, the Corps would not participate in flood risk management.

Alternative 7a: Delta Front, Mosher Slough, Lower Calaveras River, Smith Canal, and San Joaquin River.

Alternative 7b: Delta Front, Mosher Slough, Lower Calaveras River, Smith Canal, San Joaquin River, and RD 17.

Alternative 8a: Delta Front, Mosher Slough, Lower Calaveras River, Stockton Diverting Canal, Smith Canal, and more work on the San Joaquin River (i.e., includes a new levee section on Duck Creek). This is the Corps' preferred alternative, or "tentatively selected plan."

Alternative 8b: Delta Front, Mosher Slough, the Lower Calaveras River, the Stockton Diverting Canal, Smith Canal, San Joaquin River, and RD 17.

Alternative 9a: Delta Front, Mosher Slough, less work on the Lower Calaveras River (north bank terminating at North Pershing Avenue, south bank terminating at about I-5), Smith Canal, and San Joaquin River.

Alternative 9b: Delta Front, Mosher Slough, less work on the Lower Calaveras River (north bank terminating at North Pershing Avenue, south bank terminating at about I-5), Smith Canal, San Joaquin River, and RD 17.

Vegetation ETL requirement: The Corps has determined that a vegetation free zone, as required by the Corps' Engineering Technical Letter 1110-2-571 (ETL), would be established for all elements of this project at the time of construction of flood features in each reach. This vegetation free zone extends from 15 feet landward of the levee to 15 feet waterward of the levee and includes the levee slopes and crown. This area may be seeded with native grasses and forbs, but no woody vegetation would be planted or allowed to grow on or within 15 feet of levees. The Corps will establish an operation and maintenance manual which will require routine measures to maintain these vegetation free zones.

Operation and Maintenance (O&M): To establish, reestablish, or maintain the required O&M and inspection road on the landside of the levee, the Corps has determined for this project, that trees and shrubs would be removed from the landside levee from the levee toe approximately 20 feet landward on new levees and between 10 and 20 feet on existing levees, consistent with existing O&M agreements (i.e., O&M can exceed the ETL requirement). This O&M easement would be maintained clear of trees and shrubs through routine O&M (up to four times per year).

Borrow Areas: The estimated area of disturbance to obtain materials for the proposed work ranges from 132-461 acres; locations have not been specified.

Staging: Additional areas of disturbance would also be involved in construction for staging. We have no information on the locations or estimates of area at this time.

Mitigation: According to draft material provided May 30, 2014, the Corps stated that compensatory mitigation would be used to mitigate for project impacts; however, no information has yet been provided on any locations or quantities for this mitigation.

EXISTING BIOLOGICAL RESOURCES

Vegetation:

Existing resources were evaluated by conducting a site visit to representative sites in portions of the project area on May 29-30, 2013, in which the Service participated. The Corps developed an estimate of the area cover-types within the impact area of potential elements of the project using Google Earth, in which the impact boundaries were laid over satellite aerial photography. The impact area was divided into polygons of various cover-types which were summarized in tables. The Corps provided this information to the Service on April 30, 2014. Previously (March 1, 2011), the Service also participated in a more extensive site visit of RD 17, including most - but not all - of the areas included in the RD 17 element proposed in this study. Notes and photographs by Service staff from both site visits were also used to describe the resources. Below, we first describe the vegetation, then the cover-types, for the project area.

Although much of the vegetation in the project area is in a highly altered and fragmented landscape, it is a large area (~676 acres, all elements combined; ~42-53 levee miles affected by construction, depending on alternative), and varies considerably with location and even within a levee segment. For example, on Mosher Slough, there are dense trees to the west of Don Avenue, but only scattered ones to the east. On the Delta Front, there are few trees, scattered on both the land- and watersides of the impact areas. Elsewhere, on Fourteenmile Slough, levees are heavily rocked with individual trees, perhaps saved by local ordinances, and plantings associated with encroachments (e.g., gardens, boat docks). Wetland vegetation margins were seen during the site visit wherever soil and standing water occurred, even in the dryland levee area of the Delta Front, but usually not when there was a rock riprap toe, which was the much more common condition. The relatively barren appearance of the Stockton Diverting Canal, typed as ruderal, was consistent with the limited ground observations made during the site visit; although there were native and non-native trees adjacent to the canal (a Swainson's hawk was observed landside near such trees during the site visit). Here and in much of the Calaveras River sections that are under consideration, the channels are heavily maintained. However, there are often urban trees on the landside near enough to the levees that they would be affected by construction. These trees may be used by wildlife, such as hawks, which forage in the predominant dry portions of channels.

The old Mormon Channel actually has some significant vegetation in the form of sections with mature oaks and other understory riparian species, mainly between South Wilson Way and the Stockton Diverting Canal. Further west, the vegetation is sparse until Commerance Street, where the vegetation becomes thicker again, possibly due to water provided by tidal influence.

The vegetation along the San Joaquin River and French Camp Slough, including RD 17, varies considerably with location. There is more frequent woody riparian along the waterside of the levees of French Camp Slough, particularly the left (south) bank. There is also landside woody vegetation near and on the levee, either urban or associated with golf courses. Woody vegetation is much less frequent along the mainstem San Joaquin (both the San Joaquin and RD 17 elements) primarily because the levees, which are heavily rocked, form the land-water margin interface and do not provide a substrate for establishment. The vegetation which is present is limited and occurs in various forms: isolated trees or shrubs (or small groups) on the levee or at the land-water interface; shrub-scrub on levees sections that may be less vigorously maintained; limited groves of large trees on the landside (valley oak, in RD 17); and portions where the levee is set back a modest distance from the river, creating a riparian berm or oxbow (cottonwood riparian, in RD 17 and French Camp Slough). These berm areas in RD 17 are relatively infrequent and discontinuous, but could provide important fish and wildlife values due to the location. A significant quantity of this remaining vegetation is within the impact footprint, particularly within the San Joaquin River, RD 17, and Mormon Channel elements.

Wildlife:

The types of wildlife in the project area also vary with location. In urbanized areas, the impact areas are often riprapped and bordered by homes or other developments on the landside, with occasional trees at most. The adjacent waterside habitat varies with location - it can be a dry maintained floodway, open water, or open water with nearby marsh vegetation. These areas often lack ground cover or a soil layer. The most likely wildlife to occur there are those adapted to human disturbance such as house sparrow, house finch, rock pigeon, mourning dove, American crow, gulls, Norwegian rat, raccoon, and opossum. Great egret, great blue heron, and a number of species of ducks and other waterbirds would be expected in and near wetlands or other waters in the project area.

Where there is remnant forest or shrub cover, a much wider variety of wildlife can be expected, including birds such as acorn woodpecker, black phoebe, house wren, oak titmouse, western kingbird, yellow warbler, and spotted towhee, and mammals like beaver, cottontail rabbit, and (rarely) the listed riparian brush rabbit - which is known from locations of RD 17. Raptors such as the Swainson's and red-tailed hawk would be expected to be present where there are mature trees adjacent to agricultural lands (RD 17, old Mormon Channel), or urban trees adjacent to maintained dry floodways (Stockton Diverting Canal).

Fisheries:

Similarly, fish diversity and abundance would be expected to vary in the project area with urbanization, permanence of water, and tidal influence. In the nontidal urbanized waters to the east, one would expect introduced species such as mosquitofish, catfish, and carp, and perhaps a few others. In the tidal areas, including all of the sloughs and mainstem San Joaquin River, a much greater variety of fish species, both native and non-native species, are likely to be present. The San Joaquin River, its tributaries, and sloughs, are considered a major migration corridor for important anadromous species, and can also provide rearing habitat for these species. Species of major significance include fall-run chinook salmon, delta smelt, Sacramento splittail, and white and green sturgeon. Many other native and nonnative species are also likely present in the tidal waterways, including catfish, black bass, sunfish, and minnow species.

Endangered Species:

A current list of federally-listed, endangered and threatened species is provided (Appendix A). The Service has consultation responsibility for all species other than anadromous fishes, which are the responsibility of NMFS. Of these, there have been recent sightings of the endangered riparian brush rabbit in portions of RD 17. Elderberry occurs within portions of the impact area of the project in a number of locations (often as individual shrubs within the levee cross-section), and is the host plant of the threatened valley elderberry longhorn beetle - whose range includes the project area - with nearby records on the Calaveras, Cosumnes, Middle, San Joaquin, and Stanislaus Rivers. The threatened giant garter snake is also known from the project vicinity, with nearby records on the Stockton Diverting Canal and Pixley Slough, among other locations. All of the listed fishes may be present in the mainstem San Joaquin River and adjacent waters that are part of the proposed project.

Cover-types, Resource Categories, and Mitigation Goals:

The Service's Mitigation Policy (Policy) (FR 46:15 January 23, 1981) provides general guidance in making recommendations to conserve fish and wildlife resources. Under the Policy, resources are assigned to one of four Resource Categories, with a mitigation goal consistent with the values provided to fish and wildlife and the rarity of that habitat (cover-type). A mitigation goal is assigned ranging from "no loss of existing habitat value" (Resource Category 1) for the most valuable kinds of habitat, to "minimize loss of habitat value" (Resource Category 4) for the less valuable and most common kinds of habitat. Application of the policy involves designating cover-types which may be affected, and assigning evaluation species based on the sensitivity of those species to the project action, their role in the ecosystem, or association with Service-wide resource management issues such as anadromous fish and migratory birds. We then state the Resource Category, the rationale for that selection, and the corresponding mitigation goal.

Oak woodland: This cover-type is characterized by an overwhelming dominance by oaks, usually valley oak, with other species like box elder, blue and live oak, and black walnut as associates. Understory can be grass only, or include shrubs like poison oak and wild grape. It provides important resting, nesting, cover, and forage functions for deer and squirrels, and is especially important in the project area for birds like the red-shouldered hawk, which would be an evaluation species. It is present in groves near RD 17, the old Mormon Slough channel, and in portions of French Camp Slough which could be affected by the project. Due to the importance of oak woodland to the evaluations species and limited extent in the project area, we designate it Resource Category 2, with a mitigation goal of no net loss of in-kind habitat value.

Riparian Forest: This cover-type is characterized by an overstory which is often dominated by cottonwood, and which includes other species like California sycamore, valley oak, box elder, and Oregon ash; the understory includes willow species, grape, wild rose, blackberry, poison oak, and elderberry. Riparian forest supports a relatively high diversity of bird and mammal species, including woodpeckers, squirrels, rabbits, towhees, salamanders, and others which utilize different layers and niches within the forest. It is present in the project area in the forms of sporadic patches or individual trees throughout the project area at the levee toe, and on waterside berms or oxbows, where these exist, such as in RD 17, French Camp Slough, and elsewhere. Appropriate evaluation species reflecting this use would be the downy woodpecker. Due to the importance to the evaluation species, limited extent in the project area and regionally, we designate it Resource Category 2, with a mitigation goal of no net loss of in-kind habitat value.

Riparian Scrub-shrub: This cover-type consists of shrub species as just described for riparian forest. It also supports a wide variety of species, often is dominated by willows in the project area with a significant component of Himalayan blackberry, but does not support birds or species such as woodpeckers or hawks that use larger mature trees for forage or nesting. Individual elderberry plants are present in the project area, often within the existing levee (e.g., RD 17 south of Weston Boulevard; Dos Reis Park). The riparian scrub-shrub in the project area supports two listed species, the endangered riparian brush rabbit - which has been documented in the RD 17 element, and the threatened valley elderberry longhorn beetle - which was last documented in the region in a sighting along Middle River, near the project area. An appropriate evaluation species which uses this habitat would be the yellow warbler. Due to the importance to the evaluation species and limited extent in the project area, we designate it as Resource Category 2, with a mitigation goal of no net loss of in-kind habitat value.

Annual Grassland: This cover-type consists exclusively of annual grasses and, in the project area, is dominated by common grasses like ripgut brome, foxtail barley, weeds such as yellow starthistle and Italian thistle, and others. It is present on levee slopes and adjacent landside and water side areas throughout the project area that are not rocked, as well as within the upper portions of the Calaveras River and Stockton Diverting Canal floodways, which are dry outside of the flood season. Much of this area is subject to regular mowing as a maintenance and fire control activity. These areas do have wildlife value such as to foraging hawks, and their prey such as the California vole, which could serve as an evaluation species. However, this cover-type is relatively common in the region. Due to this abundance, we designate it Resource Category 4, with a mitigation goal to minimize loss of habitat value.

Orchard: This cover-type consists of fruit or nut trees, and is present in the impact footprint of the Mormon Channel bypass element. It does have value to some common mammals and bird species, although generally not to hawks. This cover-type is locally common in the planning area, but does provide somewhat greater values to fish and wildlife than grassland. Scrub jay would be an appropriate evaluation species. Considering its importance and abundance, we designate orchard as Resource Category 3, with a mitigation goal of no net loss of habitat value, while minimizing loss of in-kind habitat value.

Wetland: This cover-type occurs in or near permanent or temporary waters, and features wetland plants such as cattails, tules, and others. It provides cover and forage for songbirds associated with wetlands such as the tricolored and red-winged blackbirds, and western meadowlark, as well as wading birds like the great egret, which would serve as an evaluation species. It is sporadically present in the impact area of the project; in or near the margins of ditches, sloughs, and other waterways, usually as small-to-moderate sized patches or thin strips wherever there is an intersection of soil with shallow water. It is relatively uncommon in the planning area, although there are significant wetlands in at least Fourteenmile Slough, and it may be present in some abundance in French Camp Slough. Due to the importance to the evaluations species and limited extent in the project area, we designate wetland as Resource Category 2, with a mitigation goal of no net loss of in-kind habitat value.

Shaded Riverine Aquatic Cover: Shaded Riverine Aquatic Cover (SRA cover) is defined as the zone of interface of water with the land margin, projected over the water to the maximum extent of overhead vegetation. The habitat value within the SRA cover zone varies with factors such as water depth, overhead cover from nearby riparian trees, instream cover elements such as wood, boulders, and submerged vegetation, and the type of aquatic substrate. SRA cover is considered essential habitat to a variety of fish species, and is used as cover, forage, spawning, and rearing habitat for fishes, both anadromous species and resident native and nonnative fishes. It also provides habitat for birds such as the kingfisher. An appropriate evaluation species would be the chinook salmon, for which evaluation models for SRA cover are available. SRA cover is extremely limited in the project footprint as well as the region, the result of clearing and bank protection from prior flood control, urban development, and/or navigation projects. Due to the vital importance to the evaluations species and very limited extent in the project area, we designate SRA cover as Resource Category 2, with a mitigation goal of no net loss of in-kind habitat value.

Urban landscaping: Urban landscaping is a term applied to trees which are planted in or near residences, golf courses, parks, and other developed grounds. These are typically non-native species or varieties of native species which are obtained from nurseries for shade and aesthetic values. There is urban landscaping near as well as within the project footprint in residential areas, where some plants have been placed within the maintenance zone of existing levees, including the cross-section of the levee itself (considered encroachments). Urban landscaping can have wildlife value particularly when, as here, it is in proximity to other cover-types like annual grassland or riparian forest or scrub. We do not typically designate evaluation species or a mitigation goal for urban landscaping, which is abundant in the planning area. Rather, the Service would recommend mitigation consistent with either State or local ordinances governing removal and replacement of this type of vegetation[1].

Agriculture: Agriculture exists in portions of the project footprint, in the RD 17 and Mormon Channel elements, in the form of row crops. These are harvested regularly, leaving fallow, tilled ground. This cover-type provides forage and habitat for ground-dwelling small mammals like the California vole, which are prey items for hawks; either of which can serve as evaluation species. It is common in the planning area and region, so we designate it Resource Category 4, with a mitigation goal to minimize loss of habitat value.

Disturbed Areas: This term is used to encompass other areas that lack vegetation and/or are so frequently disturbed as to have minimal or no resource value. It would include the upper rocked faces of levees (outside of any actual or potential SRA cover), roads whether paved or not, structures (homes, boat docks), and manicured lawns and shrubs. These areas do not have an evaluation species or mitigation goal.

---

[1] For example, if the State or local ordinance specified equal replacement of trees greater than 3-inches on the basis of 1:1 diameter at breast height, say a 3-inch tree is compensated by the planting of three, one-inch saplings, the Service would recommend the same. Such policies/requirements have not been researched at the time of this draft report.

FUTURE WITHOUT THE PROJECT (NO-ACTION ALTERNATIVE)

Under the no-action alternative, the various deficiencies in the project levees would remain. ETL non-compliant vegetation would probably remain, including native and non-native shrubs, trees, and other cover in and near the various levees. The future in RD 17 without the project would depend on whether or not separate action is taken to improve those levees, such as with the proposed Section 408 project for phase III. If the RD 17 phase III project were not built (by either the Section 408 project or the Feasibility Study), planned expansions of the Cities of Stockton and Lathrop would likely not occur. Under this scenario, the habitat conditions would remain as current, with relaxed maintenance of the existing levees, allowing limited shrub-scrub, oak woodland, and riparian forest, to remain. The adjacent landscape would continue as annual grassland or agriculture. If RD 17 phase III were built separately, it would have similar effects to the RD 17 element of the Feasibility Study (see Future With The Project, below).

FUTURE WITH THE PROJECT

The future with the project is considered in this report in several ways: (1) description of the effects of the project in project elements, or similar groups of elements based on examination of aerial imagery and calculations provided by the Corps; and (2) a quantitative comparison between alternatives of the amounts of habitat loss, also provided by the Corps, for those alternatives for which calculations are available (Table 1). We have not conducted any ground verification of these impacts, nor can we judge the habitat quality factors beyond gross vegetation stature as depicted on the aerial images. In all of the affected footprint areas, we assume the levee slopes and easements would be maintained, free of vegetation, as stated by the Corps in their description of the alternatives.

Mosher Slough: Levee raise and cutoff wall construction in this reach is expected to impact 6.5 acres of woody riparian on the waterside slope and easement, much of which appears to be mature from aerial imagery, and about 8 acres of landside riparian. This would be a significant loss within this particular area.

Delta Front: Effects vary considerably in these elements. A lot of the western front work within Shima Tract and along Fourteenmile Slough appears to be bare ground with sparse vegetation, which may be ruderal, but this cannot be confirmed from aerial imagery. Waterside vegetation which would be affected appears to be more prominent beginning at the Marina and south to about the Brookside development, which has much less vegetation due to the heavily rocked slope. Then portions of the work in the slough, such as the right side adjacent to West Swain Road, display much better vegetation. The work boundary also appears to intersect vegetation on the west side of the levee along Brookside Road south to March Lane; the type and quality of this vegetation cannot be identified from imagery. There would still be significant unaffected marsh and riparian vegetation remaining and unaffected by the work, in islands and shallow areas within Fourteenmile Slough and its right bank along Fourteenmile Drive[2].

---

[2] This area is not currently shown in Corps-provided imagery as in the footprint, however, it may be affected by maintenance requirements. This has not been resolved at the time of this draft report.

Calaveras River: The Corps has calculated a total of 36.5 acres of woody riparian impacts here on the land- and watersides including easements. The work appears to stay away from most of the vegetation along the northerly portion of Tenmile Slough Road, but along the right bank (Brookside Drive) would affect all of the waterside trees and encroachments, the only vegetation on these maintained sections, as well as possible natural vegetation (south of Pinehurst Circle). There would be significant effects on vegetation on the left (south) bank (River Road) as well, noted by the Corps on the landside; however, the project footprint appears to intersect significant waterside vegetation as well. The amount of waterside vegetation and hence the impacts, become less going east, and the effects are mostly ruderal and landside woody vegetation (probably urban landscaping), east of North Pacific Avenue. Although the losses are significant, there would be a substantial amount of unaffected habitat remaining between the levees.

Stockton Diverting Canal: As reported by the Corps, imagery does appear to confirm that the bulk of the impact of work in this reach is on ruderal vegetation (~34 acres) or disturbed lands, with occasional lost landside trees. The impacts here, would therefore be temporary in nature, and this vegetation would regrow after the work is done.

Mormon Channel: As stated above, there is some mature vegetation in this relict channel, and it is probably better described as riparian forest, oak forest and riparian scrub/shrub based on our limited ground observations during the site visit than "mixed trees and shrubs" as reported by the Corps, of which 10 acres would be impacted. These estimated impacts should be considered minimums, as the thickness of this vegetation varies, and surveys may reveal more may have to be removed for the bypass channel to convey the intended 1,200 cfs capacity. Of the impacts disclosed, the woody vegetation in these locations appears (both in satellite images and during the prior site visit) to be quite mature and of apparent high habitat quality (e.g., west of Walker Lane to Wilson Way), but of lesser quality west of Wilson Way. There could be some benefits if wetland habitat were enhanced in some way, or other restoration actions were taken.

San Joaquin River: From Burns Cutoff to French Camp Slough, there is limited vegetation on the waterside of the levee, but the project work would remove a significant part (12.5 acres on both slopes and easements) of what is left, because most of this length (a few miles) is completely rocked. There would be more riparian vegetation affected by work on French Camp Slough (35.5 acres), but some of this appears to be landside golf course landscaping and, as with the Calaveras element, there would be a significant amount of unaffected riparian vegetation remaining between the levees. Nevertheless, the effects would be significant, particularly on the left (south) bank bordering the Weston Ranch development. During the site visit, we observed trees not only adjacent to, but in some cases on the existing levee cross-section.

RD 17: This work includes a fairly long (~8+ mile) right bank section of the mainstem San Joaquin River. There are long sections with rock and only occasional waterside trees as the only vegetation, many of which are in the impact footprint of the work or O&M easement where vegetation would not be allowed. We noted some such trees or shrubs were on the margin of the impact zone or within the waterside easements, but were not marked by polygons as having been impacted, so the impacts reported are probably a minimum. Overall, the Corps estimates 34+ acres of woody riparian are on the slopes and easements that would be impacted. For the northern half of this element, from French Camp Slough to about Manila Road, this would result in removal of virtually all of the remaining woody vegetation. Most of the vegetation would be removed in the southern

Table 1. Existing vegetation within the entire potential project footprint (Alternatives 8a, 8b and 9b). The project footprint is comprised of the construction footprint and constructed features plus the easements required for operation and maintenance and for the USACE ETL no vegetation zone. Vegetation numbers are in acres except for shaded riverine aquatic habitat (SRA), which is provided in linear feet. Information provided by the Corps, unpublished, June 3, 2014.

	Mosher Creek	**Delta Front[1]**	**Calaveras**	**SDC[2]**	**Mormon Channel**	**San Joaquin River downstream of French Camp Slough**	**French Camp Slough & Duck Creek**	**San Joaquin River along RD17**	**TOTAL Existing within project footprint and easements**
SRA	6,790	9,880	9,290	0	0	7,420	7,760	23,940	56,180
Waterside Slope									
Woody Riparian	2.5	2.5	2.5	0.03		5.5	13.0	16.0	42.0
Wetlands	--	--	1.0	--		--	--	--	1.0
Irrigated Grass/Park	--	3.5	--	--		--	--	--	3.5
Ruderal	0	12.5	56.0	17.0		1.0	0	3.5	90.0
Waterside Easement									
Woody Riparian	4.0	5.5	7.5	0.03		1.0	7.0	7.0	32.0
Wetlands	--	0.5	1.0	--		--	0.5	--	2.0
Ruderal	2.0	4.5	0.5	11.5		5.0	2.5	26.0	52.0
Landside Slope									
Woody Riparian	8.0	1.0	15.5	0.5		3.0	12.0	4.5	44.5
Irrigated Grass/Park	--	2.0	3.5	--		--	--	--	5.5
Ruderal	0	20.5	30.0	17.0		12.0	8.0	54.5	142.0
Landside Easement									
Woody Riparian	7.0	3.5	11.0	1.0		3.0	3.5	7.0	36.0
Wetlands	--	--	0.2	--		0.3	--	0.01	0.5
Irrigated Grass/Park/Golf Course	--	1.0	2.0	--		1.0	--	--	4.0

Table 1. Existing vegetation within the entire potential project footprint (Alternatives 8a, 8b and 9b). The project footprint is comprised of the construction footprint and constructed features plus the easements required for operation and maintenance and for the USACE ETL no vegetation zone. Vegetation numbers are in acres except for shaded riverine aquatic habitat (SRA), which is provided in linear feet. Information provided by the Corps, unpublished, June 3, 2014.

	Mosher Creek	**Delta Front**[1]	**Calaveras**	**SDC**[2]	**Mormon Channel**	**San Joaquin River downstream of French Camp Slough**	**French Camp Slough & Duck Creek**	**San Joaquin River along RD17**	**TOTAL Existing within project footprint and easements**
Orchard/vineyard	--	--	1.5	0.5		--	--	--	2.0
Row/Field crops	--	--	--	--		--	--	4.0	4.0
Ruderal	1.5	10.5	20.0	11.0		3.0	6.0	27.5	79.5
Levee Crown									
Paved/Graveled/Scraped	2.0	11.5	25.0	14.0		1.5	3.0	33.5	90.5
Woody Riparian	3.0	--	--	--		--	4.0	--	7.0
New Levee									
Orchard/vineyard	--	--	--	--		--		8.0	8.0
Row/Field crops	--	--	--	--		--		14.0	14.0
Mormon Channel Project Construction Footprint									
Mixed Trees & Shrubs					10.0				10.0
Orchard/Vineyard					4.5				4.5
Row Crops/Field Crops					1.0				1.0
TOTALS									
SRA									**56,180**
Woody Riparian									**161.5**

Table 1. Existing vegetation within the entire potential project footprint (Alternatives 8a, 8b and 9b). The project footprint is comprised of the construction footprint and constructed features plus the easements required for operation and maintenance and for the USACE ETL no vegetation zone. Vegetation numbers are in acres except for shaded riverine aquatic habitat (SRA), which is provided in linear feet. Information provided by the Corps, unpublished, June 3, 2014.

	Mosher Creek	Delta Front[1]	Calaveras	SDC[2]	Mormon Channel	San Joaquin River downstream of French Camp Slough	French Camp Slough & Duck Creek	San Joaquin River along RD17	TOTAL Existing within project footprint and easements
Mixed Trees & Shrubs									10.0
Wetlands									3.5
Irrigated Grass/Park/Golf Course									13.0
Orchard/vineyard									14.5
Row/Field crops									19.0
Ruderal									363.5
Paved/Graveled/Scraped									90.5
TOTAL									675.5

[1] Delta Front = Fourteenmile Slough, Tenmile Slough, Fivemile Slough

[2] SDC = Stockton Diverting Canal

half of the element as well, although there would be some unaffected vegetation in the vicinity of the setback segment across from the Old River confluence, a few oxbows, and some other narrow waterside berms. In the east-west dryland portion of the levee work at the south end, impacts appear to be limited to ruderal vegetation and some agriculture (14 acres total for this element).

If the RD 17 levees were improved, as either part of this Feasibility Study or the Section 408 project (both designed with a fix-in-place approach), this would permit the near-term development of most of this adjacent land into residences and commercial/industrial structures; roughly 4,700 acres on which would be built 24,000 residences and about 800 acres of commercial property. This would include all lands up to the O&M easement of the improved levee. Habitat remaining after the RD 17 work would be limited to ETL compliant vegetation outside the O&M zones, probably very limited riparian on discontinuous waterside berms and oxbows. Wildlife would be at risk of disturbance from human activities, and movement between habitat patches would likely be impaired in that location. The outcome of formal consultation with the Service and NMFS under authority of the Endangered Species Act would ensure that any such project does not jeopardize the existence and recovery of any listed species and may include measures and/or other project alternatives to provide such assurance.

Overall impact estimates are available currently only for Alternatives 8a, 8b, and 9b, but suggest that the impacts in terms of habitat loss would be significant, reflecting the large scope of the project (43-50+ miles of levee)(Table 2). The amounts of loss, depending on alternative would be at least 300 acres ruderal (annual grassland); ~95-163 acres of native woodlands, and 10+ miles of SRA cover. However, the maximum resource effects would be associated with Alternative 9b, because it

Table 2. Vegetation and Land Type Effects by Alternatives (Corps-provided; * - information not yet available).

Land Cover-types	Alt 7a*	Alt 7b*	Alt 8a	Alt 8b	Alt9a*	Alt 9b
Natural Lands						
SRA			37,680	56,180		56,180
Waterside Riparian Trees and Shrubs			35.5	74		74
Landside Riparian & Landscape Trees and Shrubs			59.5	80.5		79
Mixed Trees & Shrubs			0	0		10
Agricultural Lands						
Orchards/Vineyards			2	10		14.5
Row/Field Crops						19
Developed/Disturbed Areas						
Irrigated Grass			13	13		13
Ruderal			255.5	363.5		<318.5
Paved/Graveled/Scraped			57	90.5		<76.5

includes the vegetation impacts of both RD 17 and Mormon Channel elements on top of other elements common to all alternatives. The least impacts would be Alternatives 7a and 8a (the Corps tentatively selected plan), which would impact neither of these elements; these would likely be similar because the difference in elements (8a and 8b include the Stockton Diverting Canal and upstream portions of the Calaveras River) mostly impact additional ruderal vegetation. While we expect that Alternative 9a may have more impact than Alternatives 8a or 7a on woody vegetation, it may have other - as yet to be defined or quantified - possible benefits to wetlands and/or aquatic habitats as part of the bypass element in Mormon Channel.

DISCUSSION

Since issuance of our 2013 Planning Memorandum, the Corps has developed additional information on the footprint, type of work, and potential habitat impact, of the various alternatives for the Feasibility Study. However, there are significant remaining uncertainties, and information gaps, that need to be addressed. For example, the Corps' use of Google Earth in developing impact estimates has significant limitations. It does not discern the quality of the affected habitat, in terms of vegetation species, height, diameters, associated ground cover, plant number (in many cases), health, and other characteristics - such as inundation frequency. In some cases, this method probably has some error in distinguishing woody and herbaceous vegetation and/or wetland vegetation. These characteristics are of importance to determining effects of the project, and the need for and amount of mitigation. Ground-based study is warranted, at least for the alternative that is to be constructed.

At this time, there is also considerable uncertainty in the actual habitat impacts, depending in part on vegetation allowances. The current alternatives all assume adherence to the Corps' ETL and O&M management standards throughout the project, which would result in the removal of large amounts of land- and waterside woody vegetation within easements. The Corps also has guidance for obtaining a variance from these management standards, which might allow some vegetation to remain in place, or regrow after construction, although this must assure the safety, integrity, and function of the system are retained, and it can be inspected, monitored and maintained, among other requirements. The Corps would likely make its decision based on information on engineering characteristics of the site(s) for which a variance request, if any, is made. Even if a variance were approved, vegetation allowed under that variance may be limited.

The effects on fish and wildlife resources would vary with location, which vary considerably in prior work and remaining habitat. The proposed work in the North Stockton area includes already maintained slopes, but would involve removal of a significant number of individual trees; in these locations, these individual trees are often the only waterside vegetation, and their removal would eliminate all the remaining habitat. In Central Stockton, work on the Stockton Diverting Canal would be largely temporary effects on grassland. Elsewhere, the Mormon Channel bypass would impact relatively mature vegetation, but could be compensated in part by other created cover-types. More information needs to be developed about this alternative, and the kinds and amounts of other cover-types it could support.

Fish and wildlife resources would be more substantially affected by work in the RD 17 element than most others for several reasons. The location is along a section of the mainstem that provides key habitat for important fish and wildlife resources, including both listed wildlife such as the riparian brush rabbit, an endemic which has a very limited distribution that includes this area, and the valley elderberry longhorn beetle. These habitats are also important to migratory birds such as Swainson's hawk (which likely use both land- and waterside large trees), as well as migratory fishes - including listed fishes - which use the limited SRA cover along the margin of this location. This habitat would be lost either through construction, or maintenance of the proposed project, as well as by related development once flood protection is in place. Construction in this reach using a fix-in-place approach (either as proposed in the Feasibility Study or the 408 study), would permanently foreclose any future restoration opportunity - such as a setback of any kind - that could enhance fish and wildlife generally, and specifically contribute to the recovery of these listed species. This is because the planned development to allow urban expansion would take up all of the remaining lands (now agricultural), behind the levee. Conversely, a careful consideration of setback levees in the Lower San Joaquin River portions of the Feasibility Study - including, but not limited to RD 17 - could preserve and enhance habitat, and improve the prospect of recovery of listed species.

Although we now have a rough idea of the construction footprint and quantity of habitat impacted for several - but not all - alternatives, there is no information yet on mitigation. This is an important consideration to determining the net effect of the project on fish and wildlife resources. The Service's preference would be to modify alternatives or elements of alternatives where there are significant effects on habitat. Consistent with our 2013 Planning Memorandum on this project, and our 2011 comments on the 408 project for RD 17 phase III, we recommend first avoiding impacts by developing an alternative involving setting back the levee as continuously as possible for this element. Where there are unavoidable impacts that cannot be replaced on-site, we recommend that compensatory mitigation be implemented as near as possible to the location of impacts, in a form that replaces the same habitat or habitat values (in-kind). In particular, for losses of SRA cover on the mainstem San Joaquin River, the preferred form and location of mitigation would be replacement SRA cover also on the mainstem, and within the planning area for the project. If mitigation on- or near- the impacted area is not possible, then it should be proposed in areas believed to be of high priority for conservation or restoration, and that would provide values similar to or greater than the impact area. The Corps should at least develop mitigation plans to offset losses of all natural cover-types affected by the project (SRA cover, oak woodland, riparian forest, riparian scrub-shrub, wetland). Additional mitigation for other kinds of tree loss (urban landscaping, orchard) may be appropriate if provided by local or State ordinance, but we defer any specific recommendation for mitigation for this impact pending ground-truthing of those losses, and coordination with State and Federal resource agencies.

None of the alternatives appears to address the stated planning objective of ecosystem enhancement or restoration beyond that required for mitigation of impacts. Examples of enhancement opportunities could include levee setbacks that would allow more habitat (RD 17, elsewhere on the mainstem San Joaquin River, and other locations, including Paradise Cut), protection to ensure the future of existing habitat (e.g., through the creation of conservation easements), or additional measures to facilitate restoration on otherwise protected lands, consistent with any flood control purpose (plantings, earthwork, habitat structure). We recommend the Corps review these opportunities, and include ecosystem restoration and enhancement elements in its preferred alternative that would achieve this planning objective.

RECOMMENDATIONS

For the proposed Lower San Joaquin River Feasibility Study, the Service recommends the Corps:

1. Resolve uncertainties and information gaps in the study, as follows:

    a) Determine vegetation impacts and future allowances in all project locations with certainty, prior to construction;
    b) Clarify the expected future habitat types, and locations, for the Mormon Channel bypass;
    c) Conduct ground-level assessment of vegetation losses, including but not limited to cover typing, species, height, diameter, substrate, and inundation frequency; and a habitat evaluation procedures study if deemed appropriate by the Service;
    d) Develop and propose mitigation to offset habitat losses, using the guidance provided in this report (see Discussion, above), with locations and quantities of all mitigation plantings, and plans for monitoring;
    e) Complete assessment of impacts for all alternatives; and
    f) Identify staging and borrow areas.

2. Develop a setback levee alternative for alternatives which include the RD 17 work element;

3. Initiate section 7 consultation with the Service on the effects of project construction, operation, and maintenance, on federally-listed species;

4. Conduct appropriate consultation with the CDFW on effects to State-listed species, and with NMFS, for effects to anadromous fisheries under their jurisdiction.

5. Develop enhancement and restoration opportunities for incorporation to the maximum extent possible into the preferred alternative for the project.

REFERENCES

U.S. Army Corps of Engineers and Reclamation District 17 [Corps and RD 17]. 2011. Draft Environmental Impact Statement/Environmental Impact Report for Phase 3 - RD 17 100-Year Levee Seepage Area Project. September 2011. Prepared by AECOM, Sacramento, California, for the Army Corps of Engineers, Sacramento District. ~300 pp.+appendices.

U.S. Fish and Wildlife Service [Service]. 2011. Letter from Daniel Welsh, Sacramento Fish and Wildlife Office, to John Suazo, Sacramento District. October 24, 2011. Subject: Comments on Draft Environmental Impact Statement/Environmental Impact Report for Phase 3 - RD 17 100-Year Levee Seepage Area Project. 8 pp.

______. 2013. Planning Memorandum for the Lower San Joaquin River Feasibility Study. August 15, 2013. Sacramento Fish and Wildlife Office. Sacramento, California. Transmitted by email from Steven Schoenberg, Sacramento Fish and Wildlife Office, to Tanis Toland, Sacramento District. 5 pp + attachment.

APPENDIX A: June 9, 2014, list of endangered and threatened species that occur in or may be affected by projects in the area of the Lower San Joaquin River Feasibility Study

U.S. Fish & Wildlife Service
Sacramento Fish & Wildlife Office
Federal Endangered and Threatened Species that Occur in or may be Affected by Projects in the Counties and/or U.S.G.S. 7 1/2 Minute Quads you requested

Document Number: 140609045025

Current as of: June 9, 2014

Quad Lists

Listed Species

Invertebrates

Branchinecta lynchi
- vernal pool fairy shrimp (T)

Desmocerus californicus dimorphus
- valley elderberry longhorn beetle (T)

Lepidurus packardi
- vernal pool tadpole shrimp (E)

Fish

Acipenser medirostris
- green sturgeon (T) (NMFS)

Hypomesus transpacificus
- Critical habitat, delta smelt (X)
- delta smelt (T)

Oncorhynchus mykiss
- Central Valley steelhead (T) (NMFS)
- Critical habitat, Central Valley steelhead (X) (NMFS)

Oncorhynchus tshawytscha
- Central Valley spring-run chinook salmon (T) (NMFS)
- winter-run chinook salmon, Sacramento River (E) (NMFS)

Amphibians

Ambystoma californiense
- California tiger salamander, central population (T)

Rana draytonii
- California red-legged frog (T)

Reptiles

Thamnophis gigas
- giant garter snake (T)

Mammals

Sylvilagus bachmani riparius
- riparian brush rabbit (E)

Plants

Cordylanthus palmatus
- palmate-bracted bird's-beak (E)

Quads Containing Listed, Proposed or Candidate Species:

STOCKTON EAST (461B)
MANTECA (461C)
STOCKTON WEST (462A)
LATHROP (462D)

County Lists

No county species lists requested.

Key:

(E) *Endangered* - Listed as being in danger of extinction.

(T) *Threatened* - Listed as likely to become endangered within the foreseeable future.

(P) *Proposed* - Officially proposed in the Federal Register for listing as endangered or threatened.

(NMFS) Species under the Jurisdiction of the National Oceanic & Atmospheric Administration Fisheries Service. Consult with them directly about these species.

Critical Habitat - Area essential to the conservation of a species.

(PX) *Proposed Critical Habitat* - The species is already listed. Critical habitat is being proposed for it.

(C) *Candidate* - Candidate to become a proposed species.

(V) Vacated by a court order. Not currently in effect. Being reviewed by the Service.

(X) *Critical Habitat* designated for this species

Important Information About Your Species List

How We Make Species Lists

We store information about endangered and threatened species lists by U.S. Geological Survey 7½ minute quads. The United States is divided into these quads, which are about the size of San Francisco.

The animals on your species list are ones that occur within, **or may be affected by** projects within, the quads covered by the list.

- Fish and other aquatic species appear on your list if they are in the same watershed as your quad or if water use in your quad might affect them.
- Amphibians will be on the list for a quad or county if pesticides applied in that area may be carried to their habitat by air currents.
- Birds are shown regardless of whether they are resident or migratory. Relevant birds on the county list should be considered regardless of whether they appear on a quad list.

Plants

Any plants on your list are ones that have actually been observed in the area covered by the list. Plants may exist in an area without ever having been detected there. You can find out what's in the surrounding quads through the California Native Plant Society's online Inventory of Rare and Endangered Plants.

Surveying

Some of the species on your list may not be affected by your project. A trained biologist and/or botanist, familiar with the habitat requirements of the species on your list, should determine whether they or habitats suitable for them may be affected by your project. We recommend that your surveys include any proposed and candidate species on your list. See our Protocol and Recovery Permits pages.

For plant surveys, we recommend using the Guidelines for Conducting and Reporting Botanical Inventories. The results of your surveys should be published in any environmental documents prepared for your project.

Your Responsibilities Under the Endangered Species Act

All animals identified as listed above are fully protected under the Endangered Species Act of 1973, as amended. Section 9 of the Act and its implementing regulations prohibit the take of a federally listed wildlife species. Take is defined by the Act as "to harass, harm, pursue, hunt, shoot, wound, kill, trap, capture, or collect" any such animal.

> Take may include significant habitat modification or degradation where it actually kills or injures wildlife by significantly impairing essential behavioral patterns, including breeding, feeding, or shelter (50 CFR §17.3).

Take incidental to an otherwise lawful activity may be authorized by one of two procedures:

- If a Federal agency is involved with the permitting, funding, or carrying out of a project that may result in take, then that agency must engage in a formal consultation with the Service.

  During formal consultation, the Federal agency, the applicant and the Service work together to avoid or minimize the impact on listed species and their habitat. Such consultation would result in a biological opinion by the Service addressing the anticipated effect of the project on listed and proposed species. The opinion may authorize a limited level of incidental take.

- If no Federal agency is involved with the project, and federally listed species may be taken as part of the project, then you, the applicant, should apply for an incidental take permit. The Service may issue such a permit if you submit a satisfactory conservation plan for the species that would be affected by your project.

  Should your survey determine that federally listed or proposed species occur in the area and are likely to be affected by the project, we recommend that you work with this office and the California Department of Fish and Game to develop a plan that minimizes the project's direct and indirect impacts to listed species and compensates for project-related loss of habitat. You should include the plan in any environmental documents you file.

Critical Habitat

When a species is listed as endangered or threatened, areas of habitat considered essential to its conservation may be designated as critical habitat. These areas may require special management considerations or protection. They provide needed space for growth and normal behavior; food, water, air, light, other nutritional or physiological requirements; cover or shelter; and sites for breeding, reproduction, rearing of offspring, germination or seed dispersal.

Although critical habitat may be designated on private or State lands, activities on these lands are not restricted unless there is Federal involvement in the activities or direct harm to listed wildlife.

If any species has proposed or designated critical habitat within a quad, there will be a separate line for this on the species list. Boundary descriptions of the critical habitat may be found in the Federal Register. The information is also reprinted in the Code of Federal Regulations (50 CFR 17.95). See our Map Room page.

Candidate Species

We recommend that you address impacts to candidate species. We put plants and animals on our candidate list when we have enough scientific information to eventually propose them for listing as threatened or endangered. By considering these species early in your planning process you may be able to avoid the problems that could develop if one of these candidates was listed before the end of your project.

Species of Concern

The Sacramento Fish & Wildlife Office no longer maintains a list of species of concern. However, various other agencies and organizations maintain lists of at-risk species. These lists provide essential information for land management planning and conservation efforts. More info

Wetlands

If your project will impact wetlands, riparian habitat, or other jurisdictional waters as defined by section 404 of the Clean Water Act and/or section 10 of the Rivers and Harbors Act, you will need to obtain a permit from the U.S. Army Corps of Engineers. Impacts to wetland habitats require site specific mitigation and monitoring. For questions regarding wetlands, please contact Mark Littlefield of this office at (916) 414-6520.

Updates

Our database is constantly updated as species are proposed, listed and delisted. If you address proposed and candidate species in your planning, this should not be a problem. However, we recommend that you get an updated list every 90 days. That would be September 07, 2014.

APPENDIX A-3

NATIONAL HISTORIC PRESERVATION ACT, SECTION 106, COMPLIANCE

(DRAFT PROGRAMMATIC AGREEMENT & CORRESPONDENCE)

LOWER SAN JOAQUIN FEASIBILITY STUDY

PROGRAMMATIC AGREEMENT
BETWEEN
THE U.S. ARMY CORPS OF ENGINEERS AND
THE CALIFORNIA STATE HISTORIC PRESERVATION OFFICER,
REGARDING
THE LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY,
SAN JOAQUIN COUNTY, CALIFORNIA

WHEREAS, the Lower San Joaquin Project (Project) is The Lower San Joaquin River Feasibility Study (LSJRFS) is being accomplished generally in accordance with the Corps Section 905(b) Analysis (Water Resources Development Act (WRDA) 1986, Public Law (PL) 99-662) dated 23 September 2004. The Section 905(b) Analysis was approved by the Commander, South Pacific Division (SPD) on 10 June 2005. The Section 905(b) Analysis was prepared in response to House Report 105-190, which accompanied the Energy and Water Development Appropriations Act of 1998 (PL 105-62) ; and

WHEREAS, the Corps is proceeding with the Project, and has determined that the approved project alternatives constitute an Undertaking as defined in the Advisory Council on Historic Preservation Procedures, 36 CFR § 800.16(y); and

WHEREAS, purpose of the feasibility study is to determine the level of Federal interest in providing increased flood protection by 2025, to develop a flood protection system that is adaptable to future changing physical and environmental conditions, and to implement improvements in the study areas as soon as possible. The Project study area is along the San Joaquin River parallel to the City of Stockton in San Joaquin County. A map of the Project study area is included as Appendix 1 to this programmatic agreement (PA); and

WHEREAS, the Corps has determined that effects on properties that are either included in, or are eligible for inclusion in the National Register of Historic Places (NRHP) cannot be fully determined prior to final approval of the Project and selection of approved alternatives; this agreement addresses all phases and segments of the Project; and

WHEREAS, the Corps has consulted with the California State Historic Preservation Officer (SHPO) pursuant to Section 106 of the National Historic Preservation Act of 1966 (Section 106), and the implementing regulations described under 36 CRF Part 800; and

WHEREAS, the Corps has consulted with the Advisory Council on Historic Preservation (ACHP) and the ACHP has [declined/chosen] to participate in a letter dated [Insert Date]; and

WHEREAS, the Corps has consulted with the San Joaquin Area Flood Control Agency, and the Department of Water Resources and has invited them to participate as concurring parties; and

WHEREAS, the Corps has contacted, and will continue to consult with, the Buena Vista Rancheria of Me-Wuk Indians, The California Valley Miwok Tribe, the Ione Band of Miwok

Indians, the Nototomne/Northern Valley Yokuts, and the Wilton Rancheria and invited them to consult on this agreement and participate as concurring parties; and

NOW, THEREFORE, the Corps and the SHPO agree that the proposed Undertaking shall be implemented in accordance with the following stipulations in order to take into account the effects of the Undertaking on historic properties and to satisfy the Corps' Section 106 responsibilities for all individual aspects of the Undertaking.

I. STIPULATIONS

The Corps shall ensure that the following measures are carried out:

Stipulation I
Professional Qualifications and Definitions

A. The Corps shall ensure that historic, architectural, and archaeological work conducted pursuant to this agreement is carried out by, or under the direct supervision of, a person or persons who meet the qualifications set by the Secretary of the Interior for Archaeology and Historic Preservation, in the appropriate discipline, as required by paragraph g of 36 CFR Part 61—Procedures For State, Tribal, And Local Government Historic Preservation Programs.

B. The definitions set forth in 36 CFR § 800.16, with amendments, effective August 5, 2004, are incorporated herein by reference and apply throughout this PA;

.

Stipulation II
Area of Potential Effects

A. The Corps shall define and document the area of potential effects for all defined alternatives (APE) in consultation with SHPO. Modifications of the APE may be made by mutual agreement of the signatories without amending this Agreement.

B. The APE shall include the footprint of all construction activities, staging areas, haul roads, and mitigation sites. The APE may also include sensitive structures within range of vibratory or sonic disturbances and historic properties and districts close enough to project construction that the integrity of their setting or feeling could be affected.

C. The APE may include portions of the Recommended Study Analysis Area indicated on the map included as Appendix 1.

D. In the event that the Corps and the SHPO cannot agree on an APE, the Corps shall resolve the dispute in accordance with Stipulation XV.

Stipulation III
Identification and Evaluation

A. The Corps shall acquire a current and complete records and literature search from the Central California Information Center at California State University, Stanislaus, prior to conducting archaeological surveys of the APE. Records and literature searches shall be considered complete and current for a period of five years after they are conducted unless, in the professional opinion of Corps archaeologists, more frequent updates are required.

B. The Corps shall maintain ongoing consultation with Native American Tribes and individuals, as described in Stipulation VI, to identify properties that are of religious and cultural significance to them and that may be eligible for the National Register. Traditional Cultural Properties will be evaluated in accordance with the guidance presented in National Register Bulletin 38: *Guidelines for Evaluating and Documenting Traditional Cultural Properties*.

C. The Corps shall complete and report the results of all required cultural resources inventories of the Undertaking's APE in a manner consistent with the "Secretary of the Interior's Standards and Guidelines for Identification" (48 FR 44720-23) and take into account the National Park Service’s publication, "The Archeological Survey: Methods and Uses" (1978: GPO stock #024-016-00091). Inventories shall include both archaeological surveys and inventories of historic buildings, structures, and districts as appropriate. The Corps shall include a geoarchaeological evaluation of the APE in its survey and shall undertake subsurface reconnaissance as appropriate. Surveys shall include areas not previously surveyed and those where the Corps, in consultation with SHPO, deems previous surveys to be inadequate, e.g. areas with changes in landscape due to fire, erosion, flooding episodes which may have exposed previously unknown cultural resources. The Corps will also include additional areas that may be affected by changes in the project design, borrow areas, haul roads, staging areas, extra work space, mitigation sites, and other ancillary areas related to the Undertaking. If identified cultural resources can be evaluated for the NRHP based on the results of survey, context statements, and historic documentation, then the Corps may request SHPO concurrence with those eligibility determinations without further study. The Corps shall submit reports produced after intensive surveys to SHPO for review. The Corps shall deliver its submissions to the SHPO by email, fax, or hand delivery, whichever is most expedient. SHPO shall have thirty (30) calendar days after receipt to provide comments to the Corps.

D. The Corps shall include in its site recordation documents all unrecorded archaeological sites, linear features, and isolates encountered in the course of the survey. The Corps shall prepare updated records of previously recorded sites as necessary. The Corps’ survey shall record all prehistoric sites and all historical sites, structures, buildings, and engineering features greater than forty-five (45) years of age. Historic sites to be recorded shall include, but not be limited to: commercial, residential, and ecclesiastical buildings, roads, trails, railways, bridges, levees, culverts, and agricultural features, including ditches.

E. The Corps shall use the California Department of Parks and Recreation (DPR) Form 523 to record all newly discovered historic or prehistoric archaeological sites and isolates, previously recorded archaeological sites, and where necessary, shall create updated site records using the DPR 523 Form. Isolates shall be numbered sequentially, plotted on a map, and recorded on a single table within the report. The Corps shall examine non-linear sites that extend outside of the

APE in their entirety unless access to land is prohibited or the scale of the resource makes doing so prohibitive. In the event access cannot be gained, the Corps shall consult with SHPO regarding appropriate means of evaluating a given site. The Corps shall record linear resources (i.e., railway, ditch, canal, levee, etc.) that appear on General Land Office (GLO) plat maps or are known from other archival data to be potentially significant either on their own merit or as a contributing element to a larger resource, e.g., district, or which have associated features or dateable artifacts on DPR 523 site Forms. The Corps will treat linear resources not mentioned on GLO plat maps, or those that appear on GLO plat maps, but which are not associated with features or dateable artifacts, or those that do not otherwise appear to be significant on the basis of known archival data as "isolated feature segments" and shall record them in tabular form. Such records shall include, at a minimum, a ground-truthed map of the linear feature within the APE. Historic structures and buildings shall be recorded using the Office of Historic Preservation, Historic Resources Inventory form.

F. If the National Register significance of a cultural resource requires testing or another form of formal evaluation, an Evaluation Plan (EP) will be developed to provide for consistent and thorough evaluation. The Corps shall ensure that EPs prepared for previously unevaluated cultural resources identified within the APE are consistent with the "Secretary of the Interior's Standards and Guidelines for Evaluation" (48 FR 44723-26) and include a research design and historic context, as appropriate. The Corps shall develop individual EPs to address different categories of potentially eligible historic properties. The Corps shall develop a Discovery Evaluation Plan (DEP) and a Construction Monitoring Plan (CMP) as components of an EP. An EP shall be used whenever the Corps, in consultation with the SHPO, determines that a cultural resource should be evaluated and use of the EP is essential to determine the boundaries and data potential of the site. If the Corps undertakes any archaeological testing, such testing shall be sufficient to define and delineate the site clearly, and to determine the site's eligibility for inclusion in the NRHP. Archaeological excavation undertaken by the Corps under this Stipulation shall not exceed four (4) cubic meters of soil or five percent (5%) of the surface of the site without consultation with the SHPO. Should the Corps, in consultation with the SHPO, determine that a given resource is eligible for the NRHP; a Historic Property Treatment Plan, as described under Stipulation VIII, shall be produced for that property.

G. The Corps shall submit the EP for concurrent review to the SHPO and invited Native American Tribes. The Corps shall deliver its submissions by email, fax, or hand delivery, whichever is most expedient. The Corps shall allow reviewers thirty (30) calendar days after receipt to comment on the draft EP. The Corps shall ensure that any comments received within that time are taken into account and considered for incorporation into the final EP, as appropriate. If the Corps cannot concur with comments made by SHPO and/or Native American Tribes, the Corps will resolve the dispute in accordance with Stipulation XV. Failure of the SHPO to comment within the specified time shall not preclude the Corps from finalizing and implementing the draft EP. The Corps shall ensure that the SHPO is expeditiously provided with copies of the final EP.

H. The Corps and the SHPO may develop standard protection plans (SPP) for classes of resources that occur commonly in the APE and that may be encountered unexpectedly during construction. SPPs shall include a clear description of the class or classes of resources covered and the specific actions that the Corps will take to mitigate or avoid adverse effects to those resources.

I. The Corps shall submit all SPPs for concurrent review to the SHPO and appropriate Native American Tribes. Submissions shall be delivered in the most expeditious manner possible: by email, fax, or hand delivery. The Corps shall allow reviewers thirty (30) calendar days after receipt to comment on the draft SPP. The Corps shall ensure that any comments received within that time are taken into account and considered for incorporation into the final SPP, as appropriate. If the Corps cannot concur with comments made by SHPO and/or Native American Tribes, the Corps will resolve the dispute in accordance with Stipulation XV. Failure of the SHPO to comment within the specified time shall not preclude the Corps from finalizing and implementing the draft SPP. The Corps shall ensure that the SHPO is expeditiously provided with copies of the final SPP.

J. The Corps, in consultation with SHPO, shall ensure that determinations of eligibility are made in accordance with the criteria set forth in 36 CFR §60.4 for all properties not covered by an SPP. This applies to all historic properties within the APE, including additional areas that may be affected by changes in the project design, borrow areas, haul roads, staging areas, extra work space, mitigation sites, and other ancillary areas related to the Undertaking. If the Corps and the SHPO cannot agree on the eligibility of a property for the NRHP, the Corps shall obtain a determination from the Keeper of the National Register in accordance with 36 CFR Part 63. The determination of the Keeper shall be final for purposes of this PA. Any other disputes shall be settled following the procedure set forth under Stipulation XV.

Stipulation IV
Reporting

In accordance with Stipulation III(C) and Stipulation III(F), the Corps will prepare draft survey and evaluation reports. The Corps will ensure those copies of draft survey and evaluation reports are submitted concurrently to the SHPO, the SQF, and other parties to this agreement for a thirty (30) day period, from receipt, for review and comment. Documents shall be considered delivered five (5) days after deposit in the United States mail. Failure by any reviewer to comment within this time period shall not preclude the Corps from allowing draft reports to be finalized. Within thirty (30) calendar days of finalizing the reports, the Corps shall provide all reviewers named in these stipulations copies of all final reports.

Stipulation V
Determinations of Effect

The Corps shall apply the Criteria of Adverse Effect pursuant to 36 CFR §800.5(a) (1) to all historic properties within the APE that will be affected by the Project. The Corps shall make determinations of effect in consultation with the SHPO and other interested parties. If it is determined that the project will result in no adverse effects to historic properties, then the Corps may issue a notice to proceed with construction. If adverse effects are unavoidable, the Corps shall develop a Historic Properties Treatment Plan following the procedures set forth under Stipulations VII and VIII.

Stipulation VI

II. Tribal Consultation and Treatment of Human Remains

A. The Corps shall ensure that the Tribes identified above are invited to participate in the development and implementation of the terms of this PA. The Corps shall also invite these Tribes to participate in the identification and evaluation of historic properties. The specific manner in which this Tribal involvement will occur shall be set forth in the HPTPs.

B. The Corps shall ensure that Native American human remains, grave goods, items of cultural patrimony, and sacred objects encountered during the Undertaking that are located on state or private land are treated in accordance with the requirements of California State Health and Safety Code, Section 7050.5, NRS 383.

(i) Stipulation VII

(a) Non-Federal Stakeholder Involvement

A. In consultation with interested Native American Tribes and the Native American Heritage Commission (NAHC), the Corps will identify historic properties of traditional religious and cultural importance.

B. Following the guidance provided in National Register Bulletin 38, the Corps shall seek comments from all potentially interested Native American Tribes or other appropriate group(s) when making determinations of eligibility for any Traditional Cultural Properties.

C. The Corps has consulted with the Department of Water Resources, and the San Joaquin Area Flood Control Agency in the development of this agreement. All agencies have been invited to be concurring parties and will be given the opportunity to comment on the identification and treatment of historic properties efforts detailed in Stipulations III, V, and VII.

The Corps shall invite the interested public and Native American Tribes, to provide input on the identification, evaluation, and proposed treatment of historic properties. The Corps shall involve the interested public through letters of notification, public meetings, and/or site visits, as the Corps deems appropriate.

D. The Corps shall allow all reviewers thirty (30) calendar days after receipt to provide comments to the Corps. The Corps shall take into account all comments provided by reviewers, and incorporate them into the final survey and evaluation reports, as appropriate. The Corps shall resolve disputes in accordance with Stipulation XV.

E. Pursuant to Section 36 CFR § 800.6(c)(2-3) of the ACHP's regulations, the Corps shall consider requests by consulting parties and others to become concurring parties to this Programmatic Agreement.

III. Stipulation VIII

Preparation of Historic Property Treatment Plans

The Corps, in consultation with all parties to this agreement shall ensure that a HPTP is developed for the mitigation of anticipated effects on historic properties that will result from the Project and that cannot be avoided. Further, the Corps, in consultation with SHPO, will ensure the development of location and property specific Data Recovery Plans.

A. Avoidance of adverse effects on historic properties is the preferred treatment approach. The HPTP shall discuss and justify the chosen approaches to the treatment of project historic properties and those treatment options considered, but rejected. If preservation of part or all of any historic properties is proposed, the treatment plan shall include discussion of the following:

1. Description of the area or portions of the historic properties to be preserved in-place, and an explanation of why those areas or portions of sites were chosen;
2. Explanation of how the historic properties will be preserved in-place, including both legal and physical mechanism for such preservation;
3. A plan for monitoring and assessing the effectiveness of mechanisms to preserve the historic properties; and
4. A plan for minimizing or mitigating future adverse effects on the historic properties, if preservation in-place mechanisms prove to be ineffective.

B. When avoidance is not feasible, the Corps, in consultation with the SHPO, shall ensure the development of an appropriate treatment plan designed to lessen or mitigate project-related effects to historic properties. For properties eligible under National Register criteria A, B, or C, as described in 36 CFR §60.4, the Corps may consider mitigation other than data recovery in the treatment plan (e.g., HABS/HAER recordation, oral history, historic markers, exhibits, interpretive brochures, or publications, etc.). Where appropriate, the Corps shall include a provision in the treatment plans stipulating the development of a publication for the general public, the content of such a document, and the minimum number of copies to be produced.

C. When data recovery is proposed, the Corps, in consultation with the SHPO, shall ensure the development of a data recovery plan that is consistent with the Secretary of the Interior’s Standards and Guidelines for Archeology and Historic Preservation and the ACHP’s “Recommended Approach for Consultation on Recovery of Significant Information from Archaeological Sites” (ACHP June 17, 1999 or most recent edition). Components to be included in research designs and data recovery plans are found in Appendixes 2 and 3 to this PA.

D. Each phase or segment specific treatment plan shall relate directly to the HPTP prepared for the project, providing specific direction for the execution of data recovery within any project segment. Appendix 3 lists components to be included in data recovery plans.

E. All parties to this agreement referenced in Stipulation VII shall have the opportunity to review and comment on the HPTP’s.

IV. Stipulation IX
Review of Treatment Plan

The Corps shall ensure that draft HPTPs are submitted concurrently to the SHPO, and all parties to this agreement for review and comment. The Corps shall allow reviewers thirty (30) calendar days after receipt of the draft HPTP to provide comments. The Corps shall take into account any comments received during this time and incorporate them into the final HPTP as appropriate. In the event that disputes are not easily remedied, the Corps shall resolve them in accordance with Stipulation XV. Failure to comment within this time shall not preclude the Corps from finalizing and implementing the HPTP. The Corps shall expeditiously provide all reviewers with copies of the final HPTP.

A. If the Corps revises the HPTP, it shall allow any party, including the SHPO, 15 calendar days to review the revised HPTP. Failure of the SHPO to comment within the specified time shall not preclude the Corps from finalizing and implementing the revised HPTP in accordance with the terms of this stipulation.

B. Once the reviewing parties determine that the HPTP is adequate, the Corps shall issue permission to proceed with the implementation of the plans.

C. The Corps shall provide final copies of the HPTP to SHPO and the ACHP.

Stipulation X
Modifications of Project Scope

(1) Identification and Evaluation

1. If modification of the project scope becomes necessary or if activities are proposed in ancillary areas, such as borrow or disposal areas that have not been previously surveyed for historic properties, the Corps shall ensure that the APE, as defined and described under Stipulation II (B), of the modified project or un-surveyed ancillary area is inventoried. Any properties located within those modified APEs that may be affected by the Undertaking shall be evaluated.

2. The Corps shall identify and evaluate such properties in the manner specified in Stipulations III through IV.

3. If the Corps discovers any historic properties eligible for listing on the NRHP in the modified APE, the Corps shall develop and implement a supplemental HPTP in the manner specified in Stipulation VIII.

B. The Corps may approve construction in any area subject to the provisions of this stipulation after the Corps and the SHPO have consulted and agreed, in writing, that such construction will not affect historic properties, or that the area does not contain historic properties.

Stipulation XI
Treatment of Confidential Information

To the extent consistent with the National Historic Preservation Act, Section 304, and the Archaeological Resources Protection Act, Section 9(a), cultural resources data will be treated as confidential by all Parties and is not to be released to any party not a Party to this agreement. In carrying out their responsibilities under this PA, the Federal Agency shall restrict disclosure of information in accordance with Section 304 of NHPA and implementing regulations, and other applicable non-disclosure provisions. Confidentiality concerns for properties that have traditional religious and cultural importance to the Tribes will be respected and will be protected to the extent allowed by law.

Stipulation XII
Notices To Proceed With Construction

The Corps may issue Notices to Proceed (NTP) for individual construction segments, defined by the Corps in its construction specifications, under any of the following conditions:

1. the Corps and SHPO have determined that there are no historic properties within the APE for a particular construction segment; or

2. the Corps and SHPO have determined that there will be no adverse effects caused to historic properties within the APE for a particular construction segment; or

3. the Corps after consultation with the SHPO and all other parties to this Agreement has implemented an adequate treatment plan for the construction segment; and

    (a) the fieldwork phase of the treatment option has been completed, and

    (b) the Corps has accepted and approved a summary of the fieldwork performed and a reporting schedule for that work.

V. Stipulation XIII
1. Unanticipated Discovery of Historic Properties

If properties potentially eligible for the NRHP are discovered during construction, the Corps shall cease ground disturbing activities until it has satisfied the provisions of 36 CFR §800.13(b), “discoveries without prior planning”. The Corps shall contact the SHPO and all other parties to this Agreement within 48 hours of the discovery. The SHPO has 48 hours to respond following initial contact by the Corps. The Corps shall provide the SHPO an opportunity to review and comment on proposed treatment in accordance with Stipulation VIII.

VI. Stipulation XIV
Curation

The Corps shall ensure that all cultural materials and associated records resulting from identification, evaluation, and treatment efforts conducted under this PA are curated in accordance with 36 CFR Part 79, except as specified in Stipulation VI. Archaeological items and materials from privately owned lands will be returned to the land owners if so requested. Prior to their return, these items and materials should be maintained in accordance with 36 CFR Part 79 until all specified analyses are complete.

Stipulation XV
Dispute Resolution

A. Should any signatory to this PA object within 15 calendar days to plans provided for review pursuant to this PA or to actions proposed or carried out pursuant to this PA, with the exception of determinations of NRHP eligibility (see Stipulation III [J]), the Corps shall notify the SHPO and consult to resolve the objection. If the Corps determines that the objection cannot be resolved, the Corps shall forward all documentation relevant to the dispute to the ACHP. Within 45 days after receipt of all pertinent documentation, the ACHP shall either:

1. Provide the Corps with recommendations that the Corps shall take into account in reaching the final decision regarding the dispute; or

2. Notify the Corps that it will comment pursuant to 36 CFR §800.7, and proceed to comment. Any ACHP comment provided in response to such a request shall be taken into account by the Corps in accordance with 36 CFR §800.7 with reference to the subject of the dispute.

B. Any recommendation or comment provided by the ACHP will pertain only to the subject of the dispute. The Corps's responsibility to carry out all actions required by this PA that are not subject of the dispute shall remain unchanged.

Stipulation XVI
Amendments, Noncompliance, and Termination

A. If any signatory believes that the terms of this PA cannot be carried out or are not being met, or that an amendment to its terms should be made, that signatory shall immediately consult with the other signatories to consider and develop amendments to this PA pursuant to 36 CFR §800.6(c)(7).

B If this PA is not amended as provided for in this stipulation, the Corps, or the SHPO may terminate it. The party terminating the PA shall provide all other signatories with an explanation in writing of the reasons for termination, in accordance with 36CFR §800.6(c)(8).

C. If this PA is terminated and the Corps determines that the Undertaking authorizing the project will proceed, the Corps shall comply with 36 CFR §800.3-800.6.

VII. Stipulation XVII
1. Duration of the PA

A. Five (5) years after the execution of the PA, the signatories shall meet to discuss project progress and the efficacy of the PA. Signatories will have the option to implement modifications or revisions to the PA at this point.

B. This PA will terminate ten (10) years after the date of execution. If the project is not yet complete, the signatories shall consult not less than 90 days prior to the tenth anniversary of the execution of this PA to reconsider its terms. Reconsideration may include continuation of the PA as originally executed, amendment, or termination. If the PA is terminated because the Undertaking no longer meets the definition of an "Undertaking" set forth in 36 CFR §800.16(y), Stipulation XVII (C) shall apply.

C. This PA shall be in effect through the Corps's implementation of the Undertaking, and shall terminate and have no further force or effect when the Corps, in consultation with the SHPO, determines that the terms of this PA have been fulfilled in a satisfactory manner and/or Corps involvement in the project has ended. The Corps shall provide the other signatories with written notice of its determination and of termination of this PA.

Stipulation XVIII
2. Effective Date

This PA shall take effect on the date that it has been fully executed by the Corps and the SHPO.

EXECUTION of this PA by the Corps and the SHPO; and its transmittal to the ACHP, and subsequent implementation of its terms, evince that the Corps has afforded the ACHP an opportunity to comment on the Undertaking and its effects on historic properties; that the Corps shall take into account the effects of the Undertaking on historic properties; and that the Corps has satisfied its responsibilities under Section 106 of the National Historic Preservation Act and applicable implementing regulations for all aspects of the Undertaking.

SIGNATORIES:

U.S. ARMY CORPS OF ENGINEERS, SACRAMENTO DISTRICT

BY: ______________________________ DATE: _______________

Michael J. Farrell, Colonel, U.S. Army Corps of Engineers, District Commander

CALIFORNIA STATE HISTORIC PRESERVATION OFFICE

BY: ______________________________ DATE: _______________

Carol Roland-Nawi, Ph.D., State Historic Preservation Officer

CONCURRING PARTIES:

STATE OF CALIFORNIA DEPARTMENT OF WATER RESOURCES

BY: ______________________________ DATE: _______________

TITLE:

THE SAN JOAQUIN AREA FLOOD CONTROL AGENCY

BY: ______________________________ DATE: ______________

TITLE:

BUENA VISTA RANCHERIA OF ME-WUK INDIANS

BY: ______________________________ DATE: ______________

TITLE:

CALIFORNIA VALLEY MIWOK TRIBE

BY: ______________________________ DATE: ______________

TITLE:

IONE BAND OF MIWOK INDIANS

BY: ______________________________ DATE: ______________

TITLE:

NOTOTOMNE/NORTHERN VALLEY YOKUTS

BY: ______________________________ DATE: ______________

TITLE:

WILTON RANCHERIA

BY: ______________________________ DATE: ______________

TITLE:

Appendix 1. Area of Potential Effects

Appendix 2

4. Standards and Guidelines for Research Designs

Research designs prepared for this Undertaking shall specify, at a minimum:

- The property, or properties, or portions of properties where data recovery is to be carried out;
- Any property, or properties or portions of properties that will be destroyed with data recovery;
- The research questions to be addressed through the data recovery, with an explanation of their relevance and importance;
- The field methods to be used, with an explanation of their relevance to the research questions;
- The methods to be used in analysis, data management, and dissemination of data, including a schedule;
- The proposed disposition of recovered materials and records;
- Proposed methods by which the parties to the Programmatic Agreement will be kept informed of the work and afforded the opportunity to participate; and
- A proposed schedule for the submission of progress reports to the California State Historic Preservation Officer.

Appendix 3

Historic Property Treatment Plans (HPTP) shall address:

- The historic properties or portions of historic properties where treatment will be implemented;
- Any historic properties or portions of historic properties that will be destroyed or altered without treatment;
- If the property or properties are eligible under criteria (A), (B), or (C), a mitigation plan other than data recovery may be considered. These may include, but are not limited to HABS/HAER recordation, oral history, historic markers, exhibits, interpretive brochures or publications.
- If the property or properties are eligible under criterion (D), a research design including the research questions and goals that the data recovery on a property could inform, an

explanation of the theoretical and substantive relevance and importance of the proposed research, and specifically how the proposed actions will inform those questions and goals;

- The field and analysis methods to be used, with an explanation of their relevance to the goals of the mitigation;

- The methods to be used in data management and dissemination of data, including a schedule;

- The proposed disposition of recovered materials and records;

- Proposed methods for disseminating results of work to cultural resources professionals and separately to the interested public;

- Proposed methods by which interested Native American Tribes and individuals, local governments, and other interested persons will be kept informed about implementation of the HPTP and afforded an opportunity to comment;

- A proposed schedule for submission of progress reports to the Corps, SHPO, and the Council;

- Methods and procedures for the recovery, analysis, treatment, and disposition of human remains, associated grave goods, and objects of cultural patrimony that reflect any concerns and/or conditions identified as a result of consultations between the Corps and any affected Native American Group (see Stipulation V);

- Qualifications of consultants employed to undertake the implementation of the HPTP, will meet, at minimum, those standards described in Stipulation I.

Avoidance of adverse effects on historic properties is the preferred treatment approach. The HPTP will discuss and justify the chosen approaches to the treatment of project historic properties and those treatment options considered, but rejected. If preservation of part or all of any historic properties is proposed, the treatment plan will include discussion of the following:

1. Description of the area or portions of the historic properties to be preserved in-place, and an explanation of why those areas or portions of sites were chosen;
2. Explanation of how the historic properties will be preserved in-place, including both legal and physical mechanism for such preservation;
3. A plan for monitoring and assessing the effectiveness of mechanisms to preserve the historic properties; and
4. A plan for minimizing or mitigating future adverse effects on the historic properties, if preservation in-place mechanisms prove to be ineffective.

CALIFORNIA DEPARTMENT OF
FISH and WILDLIFE **RareFind**

Query Summary:
Taxonomic Group IS (Fish OR Amphibians OR Reptiles OR Birds OR Mammals OR Mollusks OR Arachnids OR Crustaceans OR Insects OR Ferns OR Gymnosperms OR Monocots OR Dicots OR Lichens OR Bryophytes)
AND Quad IS (Lodi South (3812113) OR Waterloo (3812112) OR Stockton West (3712183) OR Stockton East (3712182) OR Lathrop (3712173) OR Manteca (3712172))

[Print] [Close]

CNDDB Element Query Results

Scientific Name	Common Name	Taxonomic Group	Element Code	Total Occs	Returned Occs	Federal Status	State Status	Global Rank	State Rank	CA Rare Plant Rank	Other Status	Habitats
Agelaius tricolor	tricolored blackbird	Birds	ABPBXB0020	429	4	None	None	G2G3	S1S2	null	ABC_WLBCC-Watch List of Birds of Conservation Concern | BLM_S-Sensitive | CDFW_SSC-Species of Special Concern | IUCN_EN-Endangered | USFWS_BCC-Birds of Conservation Concern	Freshwater marsh | Marsh & swamp | Swamp | Wetland
Ambystoma californiense	California tiger salamander	Amphibians	AAAAA01180	1095	2	Threatened	Threatened	G2G3	S2S3	null	CDFW_SSC-Species of Special Concern | IUCN_VU-Vulnerable	Cismontane woodland | Meadow & seep | Riparian woodland | Valley & foothill grassland | Vernal pool | Wetland
Astragalus tener var. tener	alkali milk-vetch	Dicots	PDFAB0F8R1	65	1	None	None	G2T2	S2	1B.2	null	Alkali playa | Valley & foothill grassland | Vernal pool | Wetland
Athene cunicularia	burrowing owl	Birds	ABNSB10010	1858	28	None	None	G4	S3	null	BLM_S-Sensitive | CDFW_SSC-Species of Special Concern | IUCN_LC-Least Concern | USFWS_BCC-Birds of Conservation Concern	Coastal prairie | Coastal scrub | Great Basin grassland | Great Basin scrub | Mojavean desert scrub | Sonoran desert scrub | Valley & foothill grassland
Atriplex cordulata var. cordulata	heartscale	Dicots	PDCHE040B0	68	1	None	None	G3T2	S2	1B.2	BLM_S-Sensitive	Chenopod scrub | Meadow & seep | Valley & foothill grassland
Atriplex joaquinana	San Joaquin spearscale	Dicots	PDCHE041F3	109	1	None	None	G2	S2	1B.2	BLM_S-Sensitive | SB_RSABG-Rancho Santa Ana Botanic Garden	Alkali playa | Chenopod scrub | Meadow & seep | Valley & foothill grassland
Blepharizonia plumosa	big tarplant	Dicots	PDAST1C011	48	1	None	None	G2	S2	1B.1	SB_RSABG-Rancho Santa Ana Botanic Garden	Valley & foothill grassland
		Crustaceans	ICBRA03150	102	2	None	None	G2	S2	null	null	

Branchinecta mesovallensis	midvalley fairy shrimp											Vernal pool | Wetland
Brasenia schreberi	watershield	Dicots	PDCAB01010	33	1	None	None	G5	S2	2B.3	null	Marsh & swamp | Wetland
Buteo swainsoni	Swainson's hawk	Birds	ABNKC19070	2394	154	None	Threatened	G5	S3	null	ABC_WLBCC-Watch List of Birds of Conservation Concern | BLM_S-Sensitive | IUCN_LC-Least Concern | USFWS_BCC-Birds of Conservation Concern	Great Basin grassland | Riparian forest | Riparian woodland | Valley & foothill grassland
California macrophylla	round-leaved filaree	Dicots	PDGER01070	155	1	None	None	G2	S2	1B.1	BLM_S-Sensitive | SB_RSABG-Rancho Santa Ana Botanic Garden | SB_SBBG-Santa Barbara Botanic Garden	Cismontane woodland | Valley & foothill grassland
Chloropyron palmatum	palmate-bracted salty bird's-beak	Dicots	PDSCR0J0J0	26	1	Endangered	Endangered	G1	S1	1B.1	SB_RSABG-Rancho Santa Ana Botanic Garden	Chenopod scrub | Meadow & seep | Valley & foothill grassland | Wetland
Cirsium crassicaule	slough thistle	Dicots	PDAST2E0U0	18	1	None	None	G2	S2	1B.1	BLM_S-Sensitive	Chenopod scrub | Freshwater marsh | Marsh & swamp | Riparian scrub | Wetland
Delphinium recurvatum	recurved larkspur	Dicots	PDRAN0B1J0	96	1	None	None	G3	S3	1B.2	BLM_S-Sensitive	Chenopod scrub | Cismontane woodland | Valley & foothill grassland
Desmocerus californicus dimorphus	valley elderberry longhorn beetle	Insects	IICOL48011	204	3	Threatened	None	G3T2	S2	null	null	Riparian scrub
Elanus leucurus	white-tailed kite	Birds	ABNKC06010	158	1	None	None	G5	S3	null	BLM_S-Sensitive | CDFW_FP-Fully Protected | IUCN_LC-Least Concern	Cismontane woodland | Marsh & swamp | Riparian woodland | Valley & foothill grassland | Wetland
Eryngium racemosum	Delta button-celery	Dicots	PDAPI0Z0S0	26	1	None	Endangered	G1Q	S1	1B.1	null	Riparian scrub | Wetland
Hibiscus lasiocarpos var. occidentalis	woolly rose-mallow	Dicots	PDMAL0H0R3	173	1	None	None	G5T2	S2	1B.2	SB_RSABG-Rancho Santa Ana Botanic Garden	Freshwater marsh | Marsh & swamp | Wetland
Hypomesus transpacificus	Delta smelt	Fish	AFCHB01040	27	1	Threatened	Endangered	G1	S1	null	AFS_TH-Threatened | IUCN_EN-Endangered	Aquatic | Estuary
Lathyrus jepsonii var. jepsonii	Delta tule pea	Dicots	PDFAB250D2	131	1	None	None	G5T2	S2	1B.2	SB_BerrySB-Berry Seed Bank | SB_RSABG-Rancho Santa Ana Botanic Garden	Freshwater marsh | Marsh & swamp | Wetland
Lepidurus packardi		Crustaceans	ICBRA10010	272	1	Endangered	None	G3	S2S3	null	IUCN_EN-Endangered	Valley & foothill grassland |

	vernal pool tadpole shrimp											Vernal pool | Wetland
Lilaeopsis masonii	Mason's lilaeopsis	Dicots	PDAPI19030	197	2	None	Rare	G2	S2	1B.1	null	Freshwater marsh | Marsh & swamp | Riparian scrub | Wetland
Linderiella occidentalis	California linderiella	Crustaceans	ICBRA06010	416	1	None	None	G2G3	S2S3	null	IUCN_NT-Near Threatened	Vernal pool
Lytta moesta	moestan blister beetle	Insects	IICOL4C020	12	1	None	None	G2	S2	null	null	Valley & foothill grassland
Melospiza melodia	song sparrow ("Modesto" population)	Birds	ABPBXA3010	92	2	None	None	G5	S3?	null	CDFW_SSC-Species of Special Concern	null
Oncorhynchus mykiss irideus	steelhead - Central Valley DPS	Fish	AFCHA0209K	31	3	Threatened	None	G5T2	S2	null	AFS_TH-Threatened	Aquatic | Sacramento/San Joaquin flowing waters
Sagittaria sanfordii	Sanford's arrowhead	Monocots	PMALI040Q0	93	2	None	None	G3	S3	1B.2	BLM_S-Sensitive	Marsh & swamp | Wetland
Spirinchus thaleichthys	longfin smelt	Fish	AFCHB03010	45	2	Candidate	Threatened	G5	S1	null	CDFW_SSC-Species of Special Concern	Aquatic | Estuary
Sylvilagus bachmani riparius	riparian brush rabbit	Mammals	AMAEB01021	16	11	Endangered	Endangered	G5T1	S1	null	null	Riparian forest
Symphyotrichum lentum	Suisun Marsh aster	Dicots	PDASTE8470	173	2	None	None	G2	S2	1B.2	null	Brackish marsh | Freshwater marsh | Marsh & swamp | Wetland
Thamnophis gigas	giant garter snake	Reptiles	ARADB36150	271	2	Threatened	Threatened	G2	S2	null	IUCN_VU-Vulnerable	Marsh & swamp | Riparian scrub | Wetland
Trichocoronis wrightii var. wrightii	Wright's trichocoronis	Dicots	PDAST9F031	9	1	None	None	G4T3	S1	2B.1	null	Marsh & swamp | Meadow & seep | Riparian forest | Vernal pool | Wetland
Trifolium hydrophilum	saline clover	Dicots	PDFAB400R5	49	1	None	None	G2	S2	1B.2	null	Marsh & swamp | Valley & foothill grassland | Vernal pool | Wetland
Vireo bellii pusillus	least Bell's vireo	Birds	ABPBW01114	467	1	Endangered	Endangered	G5T2	S2	null	ABC_WLBCC-Watch List of Birds of Conservation Concern | IUCN_NT-Near Threatened	Riparian forest | Riparian scrub | Riparian woodland
Xanthocephalus xanthocephalus	yellow-headed blackbird	Birds	ABPBXB3010	11	1	None	None	G5	S3	null	CDFW_SSC-Species of Special Concern | IUCN_LC-Least Concern	Marsh & swamp | Wetland

APPENDIX A-4

SECTION 404(b)(1) CLEAN WATER ACT EVALUATION

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

DRAFT
Section 404(b) (1) Clean Water Act Compliance Evaluation
Lower San Joaquin River Feasibility Study

I. Introduction

This document constitutes the Statement of Findings, and review and compliance determination according to the Section 404(b)(1) guidelines for the proposed project described in the Draft FR/EIS/EIR issued by the Sacramento District. This analysis has been prepared in accordance with 40 CFR Part 230- Section 404(b)(1) guidelines and USACE Planning Guidance Notebook, ER 1105-2-100.

The Clean Water Act sets national goals and policies to eliminate the discharge of water pollutants into navigable waters. Any discharge of dredged or fill material into Waters of the United States by the Corps requires a written evaluation that demonstrates that a proposed action complies with the guidelines published at 40 CFR Part 230. These guidelines, referred to as the Section 404(b)(1) Guidelines or "Guidelines," are the substantive criteria used in evaluating discharges of dredged or fill material under Section 404 of the Clean Water Act.

Fundamental to the Guidelines is the precept that "dredged or fill material should not be discharged into the aquatic ecosystem, unless it can be demonstrated such a discharge will not have an unacceptable adverse impact either individually or in combination with known and/or probable impacts of other activities affecting the ecosystems of concern."

The procedures for documenting compliance with the Guidelines include the following:

- Examining practicable alternatives to the proposed discharge that might have fewer adverse environmental impacts, including not discharging into a water of the U.S. or discharging into an alternative aquatic site

- Evaluating the potential short- and long-term effects, including cumulative effects, of a proposed discharge of dredged or fill material on the physical, chemical, and biological components of the aquatic environment.

- Identifying appropriate and practicable measures to mitigate the unavoidable adverse environmental impacts of the proposed discharge

- Making and documenting the Findings of Compliance required by §230.12 of the Guidelines.

This Clean Water Act, Section 404(b)(1) evaluation of compliance with the Guidelines is not intended to be a “stand alone” document; it relies heavily on information provided in the draft integrated Feasibility Report and joint Environmental Impact Statement/ Environmental Impact Report (FR/EIS/EIR) to which it is attached.

II. Project Description

a. Proposed Project

The Lower San Joaquin River Project (LSJR project) is a cooperative effort by the U.S. Army Corps of Engineers (USACE) and non-federal sponsors, the Central Valley Flood Protection Board and the San Joaquin Area Flood Control Association. USACE has completed a Draft integrated Feasibility Report and joint Environmental Impact Statement/ Environmental Impact Report (draft FR/EIS/EIR), dated February 2015. The Draft FR/EIS/EIR will be referenced throughout the document to describe the existing conditions near the project site, as well as some potential impacts of the proposed project and the other alternatives. Information on alternatives is taken from Chapters 3 and 4 of the Draft FR/EIS/EIR.

The primary and permanent structures consist of roughly 23.6 miles of improved levee, a segment of floodwall, and a segment of new levee surrounding the City of Stockton and two in-water closure structures. Staging areas on the landside of the levees would be cleared for construction use and temporary concrete batch plants would be constructed on the landside of existing levees as necessary to facilitate the construction of slurry walls, flood gates, and flood wall along levee reaches. Along Calaveras River, where waterside earthen benches are present, staging may also occur on the waterside on the waterside of the levee.

The proposed project would require discharge of dredged or fill material into Waters of the United States under Section 404 of the Clean Water Act and could include the following proposed elements:

Levee Cut-off Walls, Slope Reshaping, and Levee Height Fixes – These elements are proposed to address seepage and slope stability concerns and would be applied to nearly all of the 23.6 miles of levees around North and Central Stockton. Construction activities would cause a temporary disturbance to provide space to construct the footing for the floodwall. Upon completion of the levee slopes and easement areas would be seeded with native herbaceous plant species.

Floodwall – A floodwall is proposed on Dad’s Point at the mouth of Smith Canal. The floodwall would be constructed of sheetpiles. Construction activities would cause a temporary disturbance to provide space to construct the footing for the floodwall. Upon completion of the floodwall the waterside slopes would be seeded with native herbaceous species.

Erosion Protection - To reduce erosion concerns, bank protection would be placed on the landside of levee slopes, where the levees are at risk from storm surges originating from the Delta (West).

Seismic Remediation – This project element would improve seismic stability to the Delta front levees of North Stockton that are frequently loaded (due to slough water surface elevations that are tidally influenced) and that are also subject to potentially significant deformations due to a seismic event. The seismic (deep soil mixing) remediation measure would involve installation of a grid of drilled soil-cement mixed columns aligned longitudinally with, and transverse to, the alignment of the levee extending beyond the levee prism. This measure would minimize significant deformation of the levee during a seismic event.

The seismic remediation would involve degrading approximately the top half of the levee and placing the degraded material landward as shown in Figure 4-5 of the draft FR/EIS/EIR. Prior to construction, the construction area would be cleared and grubbed. The material obtained from degrading the levee would extend up to 60 feet beyond the existing levee and would be compacted such that the material forms an extension to the existing levee. The crest of the levee would then be reconstructed with suitable material to comply with the USACE levee design criteria. A determination may be made during the future design that all of the degraded material may not be necessary to extend the levee to the proposed toe shown in Figure 4-5 of the draft FR/EIS/EIR. The proposed toe could be located along an imaginary line extending from the landward face of the proposed levee to existing grade. During the current feasibility planning the maximum extent of the reconstruction berm is shown in order to show the maximum impacts which could occur.

Deep soil mixing augers would be used to construct a continuous grouping of cells spaced equally in both the longitudinal and transverse direction to the levee alignment as shown in the plan view in Figure 4-5. The deep soil mixing is a seismic strengthening feature meant to keep the levee from liquefying during seismic activity. After construction is completed, the levee crest would then be topped with a 6-inch aggregate road, and slopes would be hydroseeded for erosion control. This degrading and reconstruction effort would occur along 3 miles of Fourteenmile Slough and Tenmile Slough.

Closure Structures on Smith Canal and on Fourteenmile Slough – Two gates would be constructed in the North and Central Stockton area. One would be on Fourteenmile Slough and one would be on Smith Canal. These gates are discussed in the draft FR/EIS/EIR in Section 5.7, Wetlands and Other Waters of the United States. Construction would require dredging or draglining, construction of a temporary cofferdam, in-water excavation, and placement of some structural features in the wet. The “wing” structures supporting the operable gates would permanently block a portion of each of these waterways. The operable gates would be about 50 feet wide and would be exercised briefly (closed and immediately opened) once or twice a year. They would close to reduce flood risk about every three years and remain closed for a day or two. One or both of these gates could also be closed as an emergency response measure if there is a levee failure eastward of the levees. The new permanent closure structure would directly affect about 0.5 acre of open water in Fourteenmile Slough and about 0.5 acre in Smith Canal. Construction would directly impact an additional 1 acre in Fourteenmile Slough and 3 acres in Smith Canal. To enable construction of a closure structure, a temporary staging area with a batch plant and graving site would be constructed adjacent to Smith Canal and adjacent to Fourteenmile Slough.

b. Location

Location information is taken from Section 1.3, Project Location and Study Area of the Draft FR/EIS/EIR.

The study area for the LSJRFS is located along the lower (northern) portion of the San Joaquin River system in the Central Valley of California (Figure 1). The San Joaquin River originates on the western slope of the Sierra Nevada and emerges from the foothills at Friant Dam (Figure 2). The river flows west to the Central Valley, where it is joined by the Fresno, Chowchilla, Merced, Tuolumne, Stanislaus and Calaveras Rivers, and smaller tributaries as it flows north to the Sacramento-San Joaquin Delta.

This proposed project area includes the flood risk management system (primarily levees) and the adjacent waterways and lands in the North and Central Stockton area. Rivers, streams, and sloughs in the project area include the San Joaquin River, Stockton Deep Water Ship Channel, French Camp Slough, Duck Creek, Lower Calaveras River, Tenmile Slough, Fivemile Slough, Fourteenmile Slough, and Mosher Creek.

c. Purpose and Need

The overall purpose of the project is to reduce flood risk to urban and urbanizing parts of the study area, including the City of Stockton. Reducing flood risk would reduce the potential for loss of life and damage to property in from flooding. The Federal objective of water resources planning is to contribute to national economic development (NEDP consistent with protecting the Nation’s environment, in accordance with national environmental statutes, applicable executive orders, and other Federal planning requirements. The Non-Federal Partners’ objective is to meet the requirements of California Senate Bill (SB) 5 of 2007, the Central Valley Flood Improvement Act, to achieve a 200-year level of protection for the urban and urbanizing areas within the Study Area. These areas have experienced multiple flooding events since records have been maintained. The existing levee system within the study area protects over 71,000 acres of mixed-use land with a current population estimated at 264,000 residents and an estimated $21 billion in damageable property.

d. Authority

The general authority for flood control investigations in the San Joaquin River Basin arises under the Flood Control Act of 1936 (Public Law [PL] 74-738), Sections 2 and 6 and amended by the Flood Control Act of 1938 (PL 75-761). The Flood Control Act of 1936, Section 6 explicitly permits further reports to be authorized by congressional resolutions. Further studies of this river system were directed in the 8 May 1964 resolution adopted by the Committee on Public Works of the House of Representatives.

e. Alternatives [40 CFR 230.10]. Unless otherwise noted, the information is from the February 2015 Draft FR/EIS/EIR.

(1) Alternative 1 - No action

The No Action Alternative serves as a benchmark against which the effects and benefits of the action alternatives are evaluated. The No Action Alternative assumes that current conditions and operation and maintenance practices would be expected to continue to occur in the foreseeable future if the project were not implemented, based on current plans and consistent with available infrastructure and community services. The No Action alternative would have no impacts to wetlands or other waters of the United States, however, this would not achieve improved flood risk management for the City of Stockton and enhanced public safety would not be realized. This alternative is not practicable, as it would not meet the purpose and need of the proposed project.

(2) Other project designs:

Alternative 7a, North and Central Stockton, Delta Front, Lower Calaveras River, and San Joaquin River Levee Improvements excluding RD 17. This alternative would implement levee improvements around North and Central Stockton and two closure structures; one on Fourteenmile Slough and one on Smith Canal. The alternative would combine the levee improvement measures of cutoff wall, deep soil mixing (seismic), and levee geometry improvements. In addition to the levee improvements, this alternative would address projected sea level change by including raises in levee height where needed.

This alternative is considered practicable and will be retained. An evaluation of the impacts of Alternative 7a will be discussed throughout this document in order to determine if it is the least environmentally damaging practicable alternative (LEDPA).

Alternative 7b, North and Central Stockton, Delta Front, Lower Calaveras River, and San Joaquin River Levee Improvements including RD 17.

This alternative would implement the same levee improvements and closure structures as Alternative 7a, but this alternative would also implement levee improvements in RD17, including about 2.2 miles of new levees at the secondary levee at the Old River flow split and a tie-back levee. The new levees would also include a cutoff wall to address potential seepage issues.

This alternative is not considered practicable because it is not consistent with USACE water resources policies. Therefore Alternative 7b will not be retained in this analysis.

Alternative 8a, North and Central Stockton, Delta Front, Lower Calaveras River, San Joaquin River, and Stockton Diverting Canal Levee Improvements excluding RD 17.

This alternative would implement levee improvements around North and Central Stockton and two closure structures; one on Fourteenmile Slough and one on Smith Canal. The alternative would combine the levee improvement measures of cutoff wall, deep soil mixing (seismic), and levee geometry improvements. In addition to the levee improvements, this alternative would address projected sea level change by including raises in levee height where needed. In addition to the levee improvements, this alternative would address projected sea level change by including raises in levee height where needed. This alternative differs from Alternative 7a in that it includes additional levee improvements (cutoff walls and slope re-shaping) along Lower Calaveras River and along the Stockton Diverting Canal.

This alternative is considered practicable and will be retained. An evaluation of the impacts of Alternative 8a will be discussed throughout this document in order to determine if it is the least environmentally damaging practicable alternative (LEDPA).

Alternative 8b, North and Central Stockton, Delta Front, Lower Calaveras River, San Joaquin River, and Stockton Diverting Canal Levee Improvements including RD 17.

This alternative would implement levee improvements around North Stockton, Central Stockton, and RD17 and two closure structures; one on Fourteenmile Slough and one on Smith Canal. The alternative would combine the levee improvement measures of cutoff wall, deep soil mixing (seismic), and levee geometry improvements. In addition to the levee improvements, this alternative would address projected sea level change by including raises in levee height where needed. This alternative differs from Alternative 8a in that it includes levee improvements and a new levee tie back in RD17. It differs from Alternative 7b in that it includes additional levee improvements (cutoff walls and slope re-shaping) along Lower Calaveras River and along the Stockton Diverting Canal.

This alternative is not considered practicable because it is not consistent with USACE water resources policies. Therefore Alternative 8b will not be retained in this analysis.

Alternative 9a, North and Central Stockton, Delta Front, Lower Calaveras River, San Joaquin River Levee Improvements and Mormon Channel Bypass excluding RD 17.

This alternative would implement levee improvements in North and Central Stockton and would construct a diversion structure on the Stockton Diverting Canal and a flood bypass through the Old Mormon Channel. The alternative would combine the levee improvement measures of cutoff wall, deep soil mixing (seismic), and levee geometry improvements. In addition to the levee improvements, this alternative would address projected sea level change by including raises in levee height where needed. The diversion control structure at the Stockton Diverting Canal would consist of pipe culverts with gates to control releases to a maximum flow of approximately 1,200 cfs about every two years. Alternative 9a differs from Alternative 7a only in the flood bypass and associated Stockton Diverting Canal structure elements.

This alternative is considered practicable and will be retained. An evaluation of the impacts of Alternative 9a will be discussed throughout this document in order to determine if it is the least environmentally damaging practicable alternative (LEDPA).

Alternative 9b, North and Central Stockton, Delta Front, Lower Calaveras River, San Joaquin River Levee Improvements and Mormon Channel Bypass including RD 17.

This alternative would implement levee improvements in North and Central Stockton and would construct a diversion structure on the Stockton Diverting Canal and a flood bypass through the Old Mormon Channel. Alternative 9a would also implement levee improvements and new levee segments in RD17. The alternative would combine the levee improvement measures of cutoff wall, deep soil mixing (seismic), and levee geometry improvements. In addition to the levee improvements, this alternative would address projected sea level change by including raises in levee height where needed. The diversion control structure at the Stockton Diverting Canal would consist of pipe culverts with gates to control releases to a maximum flow of approximately 1,200 cfs about every two years. Alternative 9b differs from Alternative 9a only in the inclusion of levee improvements and new levee segments in RD17. Alternative 9b differs from Alternative 7b only in the inclusion of a flood bypass and associated Stockton Diverting Canal structure elements.

This alternative is not considered practicable because it is not consistent with USACE water resources policies. Therefore Alternative 9b will not be retained in this analysis.

f. General Description of Dredged or Fill Material

For each of the action alternatives (Alternatives 7a, 8a, and 9a) the following project elements would require dredging and/or placement of fill waters of the United States:

- In-water closure structure on Fourteenmile Slough
    - 0.5 acres permanent impacts
    - 1 acre temporary construction impacts
- In-water closure structure on Smith Canal
    - 0.5 acres permanent impact
    - 3 acres temporary construction impacts
- Levee slope reshaping
- Seepage berms
    - Seepage berms and levee slope reshaping together could impact up to 33 miles of toe drains and ditches
- Vegetation clearing to establish USACE Vegetation ETL “vegetation free zones”

(1) General Characteristics of Material

Fill into waters of the United States is required for the purpose of 1) reshaping levee slopes and repairing levee heights, and 2) constructing two closure structures (flood gates). Materials for levee slope and height repairs would be suitable soils acquired from within 25 miles of the project area. Fill materials for bank protection, seepage berms, and adjacent levees would consist of large stone riprap to armor the waterside slope. Construction of Closure Structures would require excavation and dredging of fines, and the placement of the concrete and sheet pile for the control structure. The substrate is mostly fine sand and silt. The proposed fill for the alternatives would come from on-site construction or imported fill material. The No Action Alternative would result in no changes.

(2) Quantity of Material

An unknown quantity of material would be dredged for the construction of the closure structures and removed to an approved disposal areas. An unknown quantity of material would be placed into existing landside toe drains and ditches to construct seepage berms and levee slope reshaping under all action alternatives.

(3) Source of Material

Potential sources for borrow material include the existing levees and suitable lands within 25 miles of the project area. Potential locations for borrow would be based on current land use patterns, soil types from U.S. Soil Conservation Service (SCS), and USACE’s criteria for material specifications. Borrow sites would be lands that are the least environmentally damaging and would be obtained from willing sellers.

Any riprap required to protect the closure structures would be imported from a licensed, permitted facility that meets all Federal and State standards and requirements. Concrete material for the sheet pile walls and flood walls would be imported from a licensed, permitted facility or made by the on-site batch plant. The material would be transported along existing roadways and construction access roads.

g. Description of the Proposed Discharge Site

(1) Location

The location of the discharge sites would be at the locations of the closure structures in Fourteenmile Slough and Smith Canal. Materials dredged to construct the closure structures would be disposed of at an approved disposal site in the vicinity

(2) Size

Construction activities associated with Alternatives 7a, 8a, and 9a would result in the loss of Waters of the United States, including wetlands, as well as upland habitat, vegetation, and the disruption of wildlife movement corridor (Table 1). The project is located along the levees and waterways surrounding North and Central Stockton. Materials would be placed into Fourteenmile Slough, Tenmile Slough, and Smith Canal. Materials would also be placed into landside levee toe drains and irrigation/drainage ditches within the project footprint. These ditches and drains would be relocated and restored on site. A wetland delineation has not been completed but wetlands and other waters are assumed to be jurisdictional under Section 404 of the Clean Water Act.

Table 1: Impacts to Waters of The United States (Alternatives 7a, 8a, 9a) [1]

Location	Feature	Habitat Type	Total Permanent Impacts	Total Temporary Impacts
Fourteenmile Slough	Closure Structure	Tidally influenced estuary slough	0.5 acres	1.0 acre
Smith Canal	Closure Structure	Tidally Influenced riverine canal	0.5 acres	3 acres
Landside toe drains and ditches	Seepage berms, levee height raises, levee slope reshaping	Open water with a freshwater marsh fringe and/or riparian shrub scrub in some places.	Up to 33 miles	Toe drains and ditches would be reestablished landward of the construction

				footprint.
TOTAL IMPACT AREA			1.0 acres	4 acres

[1] In addition to the impacts shown in this table, Alternative 9a would affect the length of Old Mormon Channel by constructing a flood bypass from the Stockton Diverting Canal through Old Mormon Channel to the Stockton Deep Water Ship Channel. Restoring flood flows to Old Mormon Channel would be expected to off-set any temporary adverse construction impacts.

Alternatives 7a, 8a, and 9a would encompass the same disposal sites. However, Alternative 9a could generate a larger amount disposal material due to excavation to construct a flood bypass within Old Mormon Channel.

The No Action Alternative would have no have impacts to disposal sites.

(3) Type of Site

The types of disposal sites are two tidally influenced sloughs, one tidally influenced canal, landside toe drains and ditches, and previously disturbed designated dredge disposal sites.

(4) Type of Habitat

The following habitat types were identified at and around the project area. This discussion is broad and focuses on all habitat types, not just those that are potentially jurisdictional. The study area consists of levees plus a 15 foot waterside easement and a 20 foot landside easement. Habitat types recorded in the study area are described in Section 5.9 of the draft FR/EIS/EIR.

The Lower San Joaquin River project area supports waters of the United States, including rivers, estuarine sloughs, and wetlands. The wetlands and other waters of the United States in the project area are highly altered as a result of flood risk management projects, reclamation for agriculture and urbanization, and navigation projects. These projects have resulted in general straightening and simplification of river, stream, and slough structure.

The National Wetland Inventory (NWI) indicates several wetlands within and adjacent to the riparian zone of the San Joaquin River and its tributaries. However, NWI maps do not show wetlands as present in the footprint of proposed new levees

Perennial Drainages

The San Joaquin River, lower Calavaras River, French Camp Slough, Duck Slough, Stockton Deepwater Ship Channel, Stockton Diverting Canal, Tenmile Slough, Fourteenmile Slough, Fivemile Slough, Smith Canal, Burns

Cutoff, Mosher Slough/Creek, Paradise Cut, Old River North, Walthall Slough, and Mormon Slough are the perennial drainages in the project area. The San Joaquin River and the lower reaches of its tributaries in the project area, the Stockton Deepwater Chip Channel, and the sloughs around north Stockton are tidally influenced.

Before construction of the Stockton Diverting Canal, Old Mormon Channel was perennial in most years. Today, the channel received local stormwater runoff and intermittently contains water in portions of the channel.

Perennial to Intermittent Drainages

Landside levee toe drains are present throughout the project area. Agricultural canals and ditches are present in agricultural lands outside urban areas. In the project area, most of these agricultural canals and ditches are located on Shima Tract, Wright Tract, and in RD 17. Levee toe drains and agricultural ditches and may contain water seasonally or year-round.

Ponds

Small ponds are located eastward of the San Joaquin River levee in RD17. Manmade ponds exist in North Stockton and in the northern part of RD 17 but are part of residential developments and will not be affected by this project and are, therefore, not treated in this impact analysis.

Emergent Wetland

Narrow bands of emergent marsh are present along some portions of the San Joaquin River, its tributaries, and along the sloughs in the vicinity of north Stockton. Greater expanses are present in areas that have a waterside bench in the canal such as the tip of RD17 that joins French Camp Slough. Some depressions that exist along the lower levees and adjacent to the waterside or landside of the levees contain wetland attributes.

Toe drains, and agricultural and roadside ditches are routinely maintained to maintain flow capacity for flood risk management or agricultural purposes and, therefore, are frequently cleared of vegetation. Nevertheless, wetland vegetation is sporadically and intermittently present in and along these waterways. Toe drains and agricultural ditches are dominated by a mix of native and nonnative aquatic and semi-aquatic plant species such as curly dock, African pricklegrass, floating water primrose, willow weed, annual beard grass and nutsedge (AECOM 2011).

The Draft EIS/EIR for the RD17 Early Implementation Project (AECOM 2011) documents the presence of freshwater marsh in a depression on the landside of the levee between Howard Road to the north and a dirt farm road on the south. Vegetation in the marsh is reported as being dominated by narrow-leaved cattail with Fremont cottonwood and red willow trees growing on the perimeter. The draft EIS/EIR also documents a limited amount of freshwater marsh around the edges of a constructed pond that is located on a large private estate and equestrian center located east levee in RD17. A second area of freshwater marsh is located just in RD17 in an area of backwater on the San Joaquin River.

Intertidal Areas

Vegetated rocky intertidal areas are present in Fourteenmile Slough.

Channel Islands

These unique islands are present in the main channels in Fourteenmile Slough and in the Lower Calaveras River. Wetland vegetation is likely to be present around the edges of these islands.

Riparian Communities

In general, riparian communities are among the richest community types, in terms of structural and biotic diversity, of any plant community found in California. Riparian vegetation provided important ecological functions, including: wildlife habitat; migratory corridor for wildlife; filters out pollutants and shades waterways, thereby improving water quality; provides connectivity between waterways and nearby uplands; provision of biomass (nutrients, insects, large woody debris, etc.) to adjacent waterways; and, in some situations, reduces the severity of floods by stabilizing riverbanks. Riparian forests and woodlands –even remnant patches— are important wildlife resources because they continue to be used by a large variety of wildlife species and because of their regional and statewide scarcity.

Shaded Riverine Aquatic (SRA) Habitat

SRA habitat is the nearshore aquatic zone composed of instream woody material providing in-water cover and shoreline trees and shrubs providing overhead canopy cover. Overhanging trees and shrubs provide shade which is an element of SRA cover important to the survival of many aquatic organisms, including fish. Overhanging vegetation moderates water temperatures, which is an important factor for various life stages of native fish species. The vegetation provides food and habitat for both terrestrial and aquatic invertebrates, which in turn serve as food for several fish species. Aquatic vegetation, or in-water cover, provides a diversity of microhabitats which allows for high species diversity, abundance, and a food source for instream invertebrates, which in turn are eaten by several native fish species. Thus, a broad food base and extensive cover and habitat niches are supported by in-water cover. These values in turn create high fish diversity and abundance (USFWS 1992). Additional discussion of SRA is provided in Section 4.11, Fisheries.

Riparian Woodland

Riparian woodlands in the project area include cottonwood riparian woodland, valley oak riparian woodland, walnut riparian woodland, and riparian scrub. Riparian habitats are considered to be among the most productive wildlife habitats in California and typically support the most diverse wildlife habitats. In addition to providing important nesting and foraging habitat, riparian habitats function as wildlife movement corridors.

Great Valley Cottonwood Riparian Forest

Larger remnant patches of Great Valley cottonwood riparian forest located within the project area are dominated by large Fremont cottonwood trees and Goodding's willow. Most of the otherwise linear or smaller patchy areas of this community lack Fremont cottonwood and are represented by Gooding's willow, red willow, arroyo willow, narrow leaved-willow, and scattered valley oak, Oregon ash, and buttonbush. Native ground cover species, mainly found in the larger remnant patches of riparian forest, include California blackberry and wild rose. Common nonnative understory species found in most elements include Himalayan blackberry and tree tobacco. Most of the Great Valley cottonwood riparian forest community could also be characterized as Great Valley riparian scrub, which does not include Fremont cottonwood and is characterized by a shorter canopy and more uniform structure; however, this habitat is part of the Great Valley cottonwood riparian forest that was extensive and connected along this entire reach of the San Joaquin River, and this document therefore describes all riparian habitat as such.

Great Valley Oak Riparian Forest

Great Valley oak riparian forest is also located within the project area, occurring only on the landside of the levees. Two significant oak groves of very large, healthy valley oak trees are present on the landside in RD17 and account for the majority of the Great Valley oak riparian forest; although several groups of smaller valley oak trees and individual valley oak trees scattered along the landside and also contribute to this community. Although not measured, several of the largest trees in these landside oak groves present are close to 100 inches dbh, which is a size that indicates they are possibly several hundred years old (Bartolome 1997, cited in AECOM, 2011).

Herbaceous Community

Nonnative Annual Grasslands

Nonnative annual grassland occurs throughout the project area on levee slopes, along roadsides, and in undeveloped parcels. These areas are dominated by nonnative annual grasses and nonnative ruderal vegetation and may support stands of noxious species. Ruderal vegetation and grassland generally occurs in disturbed areas, such as levee slopes and edges of agricultural fields and roads. Areas of pasture associated with residences are primarily annual grasses that are grazed by horses and were mapped as nonnative annual grassland. The annual grasslands in the project area contain a relatively large proportion of ruderal species, likely because of substantial disturbance from human activities.

Nonnative annual grassland is dominated by naturalized annual grasses with intermixed perennial and annual forbs. Grasses commonly observed in the project area are foxtail barley, ripgut brome, Italian ryegrass, and soft chess. Other grasses are wild oats, Bermuda grass, and rattail fescue. Forbs commonly observed in annual grasslands in the project area are yellow star-thistle, prickly lettuce, bristly ox-tongue, sweet fennel, Italian thistle, horseweed, black mustard, fireweed, broad-leaf pepper grass, common sunflower, pigweed, cheeseweed, bindweed, and telegraph weed. The annual grasslands in the project area contain a relatively large proportion of ruderal species, likely because of substantial disturbance from human activities. Elderberry shrubs occur in several areas of nonnative annual grassland.

Ruderal vegetation is characterized by nonnative weedy and sometimes invasive vegetation and nonnative annual grasses. Common weed species include yellow star-thistle, black mustard, shortpod mustard, Italian thistle, milk thistle, and Himalayan blackberry; common grass species include ripgut brome, foxtail barley, Bermuda grass, and Johnsongrass. The levee slopes are dominated by ruderal vegetation. Large open areas in RD 17 are composed primarily of ruderal vegetation as are some smaller open areas that border roads, parking lots, and agricultural land, and Old Mormon Channel.

Agricultural Communities

In the project area, agricultural lands include row and field crops, fallow and disked agricultural fields, orchards, and vineyards. General farming practices result in monotypic stands of vegetation for the growing season and bare ground in the fall and winter. Irrigation ditches are a part of most of the agricultural fields in the project area.

Cropland occurs in RD17, Shima Tract, Wright Tract, northeast of the Stockton Diverting Canal, and along the upper reaches of the Calaveras River. Ruderal species grow along the edges of fields and irrigation ditches, some of which contain water and associated aquatic plants.

Developed Lands

Developed lands in the project area include areas in levee roads, railways, roads, buildings, and landscaped areas as well as barren areas that have been disturbed and are not vegetated. Developed areas consist of residential areas; parks; boat launching facilities; boat docks; and ranch houses and related facilities. Vegetation in residential areas and parks consists of turf grasses, landscape trees, and occasional valley oak trees. Ranch lands often contain, a variety of landscape trees and shrubs, and occasional native trees including valley oak trees. In north and central Stockton, most of the areas landside levees in the project area are "developed." This is also true of lands in the northern portion of RD17 (Weston Ranch) and in the southern RD17 near Lathrop and Manteca.

(5) Timing and Duration of Discharge

Full project construction would occur over twelve years. Fill of landside toe drains and ditches would occur at the time that each levee segment is constructed. These toe drains and ditches would be reestablished further landward at the time that each levee segment is constructed. Construction of each closure structure is expected to take two construction seasons.

h. <u>Description of Disposal Method</u>

The descriptions of the disposal methods within the proposed project area are excerpted below from the Draft FR/EIS/EIR.

Construction of the closure structures would take place from a barge and/or from heavy equipment on the top of the levee. Construction would disturb the aquatic environment, including nearshore marsh habitat, and would require removal of vegetation on and adjacent to the levee. Material dredged removed for the closure structures would be used in construction of other project features (floodwall, levees) where feasible. The remainder of the materials would be hauled off site and disposed of at a designated disposal site. Conservation and compensation plantings at the water's edge would be accomplished from a barge using a "stinger." These plantings may be placed into existing rip rap or riverine soils.

Alternatives 7a, 8a, and 9a would utilize similar disposal methods. However, Alternative 9a would also include excavation within the Old Mormon Channel in order to establish a flood bypass. The No Action Alternative would not require the disposal of materials.

II. Factual Determinations

a. Physical Substrate Determinations (Sections 230.11 (a) and 230.20)

(1) Comparison of Existing Substrate and Fill

The description of the current substrate within the proposed project area is taken from Sections 5.1 and 5.3 of the Draft FR/EIS/EIR. The existing levee system is located on deposits consisting of Holocene alluvium and Holocene basin deposits, as well as late Pleistocene alluvial fan and terrace deposits of the Modesto and Riverbank Formations. These Quaternary deposits are variably dissected and overlain by younger Quaternary (Historical) deposits consisting of channel, floodplain, and artificial fill (levees and spoils from dredging). Some rocky substrate is present within Fourteenmile Slough in the vicinity of the proposed closure structure.

Soils in the project area range from highly sandy to dominantly fine, with fine to extremely coarse gradations. Erosion and expansion potentials are low to moderate for the soil series. Severe erosion is not generally a concern due to the relatively level terrain; however, wind can erode exposed and recently disturbed soils. Expansive soils contain a higher content of clay and expand and shrink, depending on water content. Subsidence can occur locally as a result of seasonal changes in soil moisture content. Substantial groundwater-related subsidence has occurred throughout the San Joaquin Valley as drainage of lowlands has resulted in the decomposition of organic components in the soils.

Fill material used during project construction would come from borrow material excavated from the within 25-mile radius of the project area and from existing on-site levee materials removed to make the proposed levee improvements.

(2) Changes to Disposal Area Elevation

The description of changes to the disposal sites within the proposed project area are taken from Chapter 4 of the Draft FR/EIS/EIR. Alternatives 7a, 8a, and 9a all involve placement of permanent materials into Fourteenmile Slough and Smith Canal in order to construct closure structures. They all also include placement of fill into Fourteenmile Slough and Tenmile Slough to construct an in-water work platform for construction of seismic remediation of adjacent levees.

Alternatives 7a, 8a, and 9a primarily call for landside levee fixes that do not change in-channel geometry or characteristics; therefore, the hydraulics of the system would not change. The hydraulic analysis completed for this study considered the impacts of the two closure structures (on Fourteenmile Slough and Smith Canal). Additional work is expected to reduce the area of impact and minimize affect to water surface elevation, except where the objective is to reduce flood risk by operating the closure structure gates when the water surface elevation reaches 8 feet. With the mitigation measures proposed to avoid and minimize impacts, the impacts of the proposed project on elevation would be minimal. The closure structures would extend from the in-water substrate to several feet above the water surface.

The closure structures were analyzed with a hydraulic model. The closure structures would operate (close) when the water surface elevation of the adjacent waters reach 8 feet in elevation. The purpose of these structures is to reduce hydraulic pressure on levees surrounding the City of Stockton by taking the peak off of flood flows about every three years. Under Alternative 9a, Old Mormon Channel would be excavated in specific locations to assure passage of 1,200 cfs. The no action/no project alternative would not modify the substrate elevation or bottom contours.

(3) Migration of Fill

The description of materials and placement are taken from Chapter 4 of the Draft FR/EIS/EIR.

Levee improvements around North and Central Stockton, including cutoff wall construction, levee height fixes, levee raises, slope reshaping, and seismic remediation would require ground disturbing activities that would potentially cause erosion and soil disturbance, subsequently resulting in sediment transport and delivery to aquatic habitats. An increase in sedimentation and turbidity could occur in adjacent water bodies during earth moving activities and could be considered significant. These indirect effects would be reduced to less than significant with the implementation of BMPs discussed in Water Quality (Section 3.5).

Alternatives 7a, 8a, and 9a would produce similar impacts on erosion and accretion patterns that would be minimized with the use of BMP’s.

The no action alternative would not result in any change to erosion and accretion patterns.

(4) Duration and Extent of Substrate Change

Alternatives 7a, 8a, and 9a would cause similar impacts to substrate. The proposed action would result in the removal of some native substrate. During project design, additional opportunities to reduce impacts will be evaluated. Alternative 9a would cause additional impacts due to the construction of the flood bypass through Old Mormon Channel.

The no action/no project alternative would not modify the substrate.

(5) Changes to Environmental Quality and Value

Alternatives 7a, 8a, and 9a would each require dredging for the two closure structures. Disposal sites selected would be previously disturbed areas that are designated disposal areas. Placement of material at these locations would be consistent with current land use. Additional information on vegetation, wildlife, and fisheries is found in Chapter 5 of the Draft FR/EIS/EIR. Materials excavated from Old Mormon Channel under Alternative 9a would be disposed at approved locations on land. The no action alternative would not modify the environmental quality and value.

(6) Actions to Minimize Impacts

Construction would have minor, short-term impacts. Constructed features (closure structures) would permanently alter the affected waterways. Best management practices, like use of silt fences to reduce unintended soil movement and turbidity, would be implemented to avoid impacts. Potential impacts would be further minimized through design and operational refinements to the extent feasible. Compensatory mitigation would off-set any remaining impacts. Additional information on mitigation measures, including BMPs is in Sections 5.5 and 5.7 of the Draft FR/EIS/EIR.

b. Water Circulation, Fluctuation, and Salinity Determinations

(1) Alteration of Current Patterns and Water Circulation

The operation of the closure structures under Alternatives 7a, 8a, and 9a and the resultant change in stages in the waterways East of the closure structures has been analyzed with a hydraulic model to achieve the intended risk reduction for the City of Stockton. The stages and tidal prism West of the closure structures would not change; it is assumed when the closure structures are operating, the stages in the waterways to the East of the structures would remain at a non-damaging stage of 8 feet (NAVD88). The operation of the two closure structures will be further refined during the next project phase. The gate operation of the closure structure could be dependent on a number of conditions within the project area.

The no action/no project alternative assumes no action would be taken. In the no action scenario, currents, circulation and drainage patterns of system would remain unchanged.

(2) Interference with Water Level Fluctuation

Because the San Joaquin River system is regulated by upstream dams which allow a specific amount of water to be released into systems, the practicable build alternatives and the no action/no project alternative would not change water level fluctuation patterns.

(3) Salinity Gradients Alteration

Salinity gradients would not be affected.

(4) Effects on Water Quality

The description of the current water quality condition of surface waters in the project area is taken from Section 5.5 of the Draft FR/EIS/EIR.

The latest version of the Section 303(d) list for California issued by the SWRCB (approved October 26, 2006) identifies impaired status for waterways in the eastern Delta, including the upper San Joaquin River. Potential source of pollution for all of the listed constituents in the basin include agriculture, urban runoff/storm sewers, resource extraction, and unknown sources. The eastern Delta, including the upper San Joaquin River, is on the Section 303(d) list for impairment for boron, chlorpyrifos, diazinon, dichlorodiphenyltrichloroethane (DDT), electrical conductivity (EC), unknown toxicity, Group A pesticides, exotic species, and mercury. Downstream of RD17, the Stockton Deepwater Ship Channel is being addressed by a Total Maximum Daily Load (TMDL) for dissolved oxygen and is no longer on the Section 303(d) list. TMDLs have been initiated for organophosphorous pesticides (i.e., diazinon and chlorpyrifos), salinity and boron, and selenium in the upper San Joaquin River watershed and for total dissolved solids (TDS) and mercury in Delta channels, TMDLs for the other listed pollutants are scheduled to be developed at various times over the next 10 years in accordance with the priorities contained in the Section 303(d) list.

(a) Water Chemistry

Project activities involving concrete and concrete wash water have the potential to affect pH, turbidity, and hexavalent chromium in receiving waters. Concrete wash water tends to have relatively high pH (between 10 and 14). Approved BMPs for managing concrete wash water include curing / air drying, off hauling for treatment, and active treatment onsite using carbon dioxide or a stronger acid such as sulfuric or acid. Hexavalent chromium is present in Portland Cement Concrete (PCC) and PCC grindings. Active treatment systems (ATS) targeting pH and turbidity may not remove hexavalent chromium, unless they are augmented with ferrous sulfate or some other chemical agent to reduce hexavalent chromium to trivalent chromium.

Mitigation measures proposed for pH and turbidity would be development and implementation of an approved Stormwater Pollution Prevention Plan (SWPPP), including an ATS if needed to attain water quality objectives. To mitigate for hexavalent chromium risks, the ATS plan would include monitoring and treatment measures to attain no significant increase of hexavalent chromium in receiving waters.

(b) Salinity

The project would not change salinity levels.

(c) Clarity

Dredging and placement of fill materials would temporarily reduce clarity due to an increase in total suspended solids within the project area. Clarity is not expected to be substantially affected outside the immediate project area. However, the reduction of clarity caused by construction activities would be short in duration and would return to pre-construction levels upon project completion.

(d) Color

Dredging and placement of fill materials would temporarily induce a color change due to an increase in turbidity. However, conditions would return to pre-construction levels upon completion of the project.

(e) Odor

The project would not affect odor.

(f) Taste

The project would not affect taste.

(g) Dissolved Gas Levels

The proposed project would have temporary impacts on dissolved gas levels within the project vicinity. Development and implementation of an approved SWPPP would avoid significant negative effects.

(h) Temperature

Construction activities have the potential to create substantial turbidity, thus affecting water temperature. Proposed mitigation measures, specifically, conducting work during low flow periods and installing sediment barriers to reduce sediment from entering waterways would be required to control turbidity and the mobilization of pollutants that may be present in sediments. Removal of trees and shrubs that overhang the waterways could increase water temperature in the immediate vicinity.

(i) Nutrients

Release of suspended sediments from project activities could potentially cause turbidity thresholds to be exceeded. This could concurrently cause thresholds for metals and nutrients to be exceeded. Turbidity would be controlled outside the working area using a combination of BMPS as appropriate. Development and implementation of an approved SWPPP would also prevent release of excess nutrients.

(j) Eutrophication

The project is not expected to contribute excess nutrients into the stream or promote excessive plant growth due to BMPs and design and operational refinements.

(5) Changes to Environmental Quality and Value

Alternatives 7a, 8a, and 9a could impact the water quality during construction from earth moving operations, storage and handling of construction materials on site and the operation and maintenance of construction equipment on-site. Construction and associated materials, including solvents, paints, waste materials and fuels associated with operation and maintenance of construction equipment present on-site could introduce hazardous or toxic materials and silt and debris into surrounding waters, resulting in degradation of the water quality. Although there is risk of substantial effects to water quality during project construction, these effects would be short term and localized within the project area. Effective compliance with BMPs, containment plans, and CVRWQCB water quality thresholds is expected to lower risk of changes to environmental quality and value.

Construction of the Fourteenmile Slough and Smith Canal closure structures would significantly affect water quality in adjacent waterways. Construction of the closure structures would require construction of coffer dams, dewatering the areas enclosed by the coffer dams, excavation of within the enclosed area in order to construct the closure structures. These activities could cause sediment runoff into the adjacent waterways.

(6) Actions to Minimize Impacts

Construction and excavation would be timed with low water levels when possible to minimize impacts. The impacts to water quality due to construction activities would be minimized by compliance with thresholds of the Section 401 Water Quality Certification, issued by the Central Valley Regional Water Quality Control Board (CVRWQCB).

In addition, proposed mitigation measures would reduce the potential impacts of the proposed project on water quality. These mitigation measures are located in the Water Quality Section (5.5) of the Draft FR/EIS/EIR.

The contractor would be required to produce compliance plans and implement the proposed mitigation measures during project construction; therefore, impacts to the water quality from project construction are expected to be minimal.

c. Suspended Particulate/Turbidity Determinations

(1) Alteration of Suspended Particulate Type and Concentration

During construction, risk is present for increased levels of turbidity as soils are exposed during rain events. In addition, the dredging of material and placement of fill materials could result in releases of suspended sediments and increased turbidity into the water. Exposed material could be eroded by wave action or storm runoff. The use of best management practices (BMP's), such as utilizing erosion control devices (silt fencing) within the project area, and side slope stabilization of exposed fills would minimize increases in suspended sediments or turbidity associated with the proposed project. Additional information on water quality is found in Section 5.5 of the Draft FR/EIS/EIR.

The no action/no project alternative would result in the project not being completed, which would result in no impacts to suspended sediment and turbidity.

(2) Particulate Plumes Associated with Discharge

Earthwork would be performed during low flow periods to minimize particulate plumes. However, particulate plumes could occur from the placement of fill materials but are expected to be contained. Plumes would dissipate after construction activity is completed.

(3) Changes to Environmental Quality and Value

Particulate plumes resulting from any construction activity under Alternatives 7a, 8a, and 9a would not persist after project completion. Particulates suspended within the disposal area are not expected to differ in type from particulates currently within the project area.

There could also be long term effects to water quality as the closure structures begin to deteriorate over time. Increased turbidity and metal contamination in the water column as iron or other metals in the closure structures corrode would also impact water quality. In addition, maintenance activities would disturb the channel bottom during repairs.

(4) *Actions to Minimize Impacts*

Effects would be minimized by performing work during low water level periods when possible. A Stormwater Pollution Protection Plan (SWPPP) would be prepared for project construction, which would describe and identify BMPs that would minimize impacts during on-site and off-site construction activities. As a result of contractor compliance with the CVRWQCB certification, consistent water quality monitoring, and mitigation measures listed in Section 5.5 of the Draft FR/EIS/EIR, increases in sedimentation and turbidity are expected to be minimized and temporary.

d. Contaminant Determinations

Construction activities for Alternatives 7a, 8a, and 9a would involve the use of hazardous materials such as fuels and lubricants to operate construction equipment and vehicles such as excavators, compactors, haul trucks, and loaders. Bentonite (a non- hazardous material) would be transported to sites where slurry cutoff wall construction would occur.

Construction of closure structures in Fourteenmile Slough and Smith Canal could result in the release of different types of contaminants that exist in the soil into the environment, significantly affecting water quality. These contaminants include pesticides, fertilizers, organic litter, and debris containing hazardous substances. In addition, contaminated dredge material could be exposed during excavation of the Fourteenmile Slough and Smith Canal for placement of the closure structures.

Alternatives 7a, 8a, and 9a involve the use of borrow material. In order to ensure that there are no contaminants within the proposed borrow or fill material, BMPs listed in the Water Quality Section (Section 5.5) of the Draft FR/EIS/EIR would be implemented. Provided these mitigation measures are implemented by the contractor, there would be minimal impacts to aquatic resources from contaminants. The no action alternative would result in no impacts from potential contaminants.

e. Aquatic Ecosystem and Organism Determinations

(1) Effects on Plankton

Plankton are drifting organisms that inhabit the pelagic zone of oceans, seas, or bodies of fresh water. Construction of the project would be temporary and short term and would include temporary displacement due to in-water construction and decreased plankton density due to increased turbidity. With implementation of mitigation measures and BMPS, the effects would be temporary and not significant.

(2) Effects on Benthos

Benthic organisms are found in the benthic zone which is the ecological region at the lowest level of a body of water such as an ocean or a lake, including the sediment surface and some sub-surface layers. Native benthic species could be affected by the dredging and excavation required to construct the closure structures. Dredging would result in the complete removal of benthic organisms from the control structure site.

(3) Effects on Nekton

Nekton are actively swimming aquatic organisms that range in size and complexity from plankton to marine mammals. Descriptions of fish and other aquatic resources below are from Sections 5.11 and 5.12 of the Draft FR/EIS/EIR.

Native fish present in the Lower San Joaquin River study area can be separated into anadromous species and resident species. Native anadromous species include four runs of Chinook salmon, steelhead trout, and green sturgeon. All of these anadromous species are expected to use habitat in parts of the study area. Native resident species include but are not limited to pikeminnow (*Ptychocheilus grandis*), Sacramento splittail (*Pogonichthys macrolepidotus*), Sacramento sucker (*Catostomus occidentalis*), hardhead (*Mylopharodon conocephalus*), San Joaquin roach (*Lavinia symmetricus*), and steelhead/rainbow trout (*O. mykiss*) and can be found throughout the study area in various aquatic habitats. Additional native and nonnative fish species potentially present in the study area can be seen in Table 5.11-1 of the Draft FR/EIS/EIR.

Project construction may disturb soils and the nearshore environment, leading to increases in sediment in the nearshore aquatic habitat. This in turn may increase sedimentation (i.e., deposition of sediment on the substrate), suspended sediments, and turbidity. Increases in suspended solids and turbidity will generally be short-term in nature and not result in a substantial reduction in population abundance, movement, and distribution.

Due to the common footprints of the action alternatives, the impacts to fish and other aquatic organisms would be similar as for the proposed project.

The no-action alternative would result in no losses of habitat for fish and other aquatic organisms.

(4) Effects on Aquatic Food Web

Description of ecological effects is taken from Sections 5.11 and 5.19 of the Draft FR/EIS/EIR.

Under Alternatives 7a, 8a, and 9a, levee improvements, construction of a floodwall at Smith Canal, and construction of the two closure structures would produce vibration from construction equipment would most likely disturb the native resident fish by increasing noise, water turbulence, and turbidity, causing them to move away from the area of placement. For some pelagic native juvenile species utilizing the near shore habitat for cover, moving away from that cover could put them at a slight increased risk of predation. Other measures for the San Joaquin River levees, including cutoff wall construction, levee height and slope reshaping, would be constructed outside of the natural river channel with no direct significant effects to native fish species.

Additional indirect effects from the permanent closure structures on Fourteenmile Slough and Smith Canal could have potentially significant effects. During non-operational conditions overwater and in-water structures can alter underwater light conditions and provide potentially favorable holding conditions for adult fish, including species that prey on juvenile fishes. Permanent shading from the installation of piles and other structures could increase the number of predatory fish (e.g., striped bass, largemouth bass) holding in the study area and their ability to prey on resident native fish species.

Implementation of BMP's and other mitigation measures proposed (Section 5.11) would result in minimal impacts on fish and aquatic wildlife habitat outside the immediate work area. The no-action alternative would result in no effect to fish and other aquatic organisms.

(5) Effects on Special Aquatic Sites

(a) Sanctuaries and Refuges

No sanctuaries and refuges are within the project area.

(b) Wetlands

Seasonal and permanent wetlands likely occur along portions of all of the waterways that would be affected by the project. During the next project phase a qualified wetlands biologist will identify and evaluate all wetlands potentially affected by the project.

(c) Mud Flats

No mud flats are within the project area.

(d) Vegetated Shallows

No vegetated shallows are within the project area.

(e) Coral Reefs

No coral reefs are within the project area.

(f) Riffle and Pool Complexes

No riffle and pool complexes are within the project area.

(6) Threatened and Endangered Species

Implementation of Alternatives 7a, 8a, and 9a could result in direct effects to the listed valley elderberry longhorn beetle (VELB) if elderberry shrubs are incidentally damaged by construction personnel or equipment. Impacts may also occur if elderberry shrubs need to be transplanted because they are located in areas that cannot be avoided by construction activities. Potential impacts due to damage or transplantation include direct mortality of beetles and/or disruption of their lifecycle.

The potential to affect giant garter snake and its habitat exists in the Stockton Diverting Canal. Alternative 8a would include levee improvements on the Stockton Diverting Canal. These improvements are not expected to impact waters of the United States, including wetlands. Construction activities would temporarily affect potential upland habitat. The canal provides low to moderate food, cover, and water values for GGS.

Special-status birds protected under the Migratory Bird Treaty Act (MBTA) including Swainson’s hawk and tricolored blackbird have potential to nest in or adjacent to the study area based on reported occurrences within a 1-mile radius.

In the study area, burrowing owls could nest in areas with non-native grasslands intermixed with barren ground and in unvegetated areas at farmland areas having berms or levees nearby. Construction activities, including grading and clearing activities within and adjacent to these lands cover types, could result in nesting failure, death of nestlings, or loss of eggs.

Construction activities such as tree removal and trimming or construction noise could result in significant impacts on roosting hoary, Western red, and pallid bats, including the destruction of active roosts, the loss of individuals, or roost failure and the disruption of the wildlife movement corridor. In addition, nighttime construction activities, if needed, could disturb bats emerging from nearby roosts resulting in the disruption of foraging activities.

Direct and indirect significant effects to Chinook salmon, Central Valley steelhead, green sturgeon, and delta smelt due to loss of SRA and riparian habitat from construction of project features and clearing to establish the USACE Levee Vegetation ETL vegetation free zones. Long-term effects on fish habitat include loss of aquatic vegetation and SRA cover. Water quality effects, such as impacts from fuel leaks or contaminants, are detailed in the water quality analysis (Section 5.5).

Alternative 9a has the same project footprint as Alternative 7a, except that it includes construction of a diversion structure in the Stockton Diverting Canal levee, utility relocations, and excavation in Old Mormon Channel in order to divert 1,200 cfs of flood flows into Old Mormon Channel about every two years. This may result in changes to fish migration. Renewed floodflows may also improve wetland habitat and water quality in portions of Old Mormon Channel and in the Stockton Deep Water Ship Channel.

All terms and conditions of Incidental Take Statements accompanying Biological Opinions issued by the USFWS and NMFS will be fully implemented, as appropriate.

The no action alternative would not result in direct impacts to endangered and/or threatened species.

(7) Other Wildlife

Alternatives 7a, 8a, and 9a would have short-term and long-term effects on resident mammals, birds, reptiles, and amphibians. Noise from construction equipment and increased human presence could temporarily displace some wildlife, and temporary alteration of riparian and aquatic habitat would occur. Removal of trees and shrubs would eliminate habitat and interrupt movement corridors.

To ensure that there would be no effect to migratory birds, preconstruction surveys would be conducted, if needed, in and around the project area. If any migratory birds are found, a protective buffer would be delineated, and USFWS and CDFG would be consulted for further actions. Recommendations proposed by the USFWS in their June 24, 2014, Fish and Wildlife Coordination Act Report and USACE responses are provided below:

USFWS Recommendation 1: Resolve uncertainties and information gaps in the study, as follows:

a) Determine vegetation impacts and future allowances in all project locations with certainty, prior to construction;
b) Clarify the expected future habitat types, and locations, for the Mormon Channel bypass;
c) Conduct ground-level assessment of vegetation losses, including but not limited to cover typing, species, height, diameter, substrate, and inundation frequency; and a habitat evaluation procedures study if deemed appropriate by the Service;
d) Develop and propose mitigation to offset habitat losses, using the guidance provided in this report (see Discussion, above), with locations and quantities of all mitigation plantings, and plans for monitoring;
e) Complete assessment of impacts for all alternatives; and
f) Identify staging and borrow areas.

Response: Concur in part. As part of USACE Planning Modernization, some of the specific information previously developed during the feasibility phase of a project is either not developed or is developed during later project phases only for the preferred alternative (TSP). The simplifying assumptions and analytical methods that were used to quantify impacts are likely to overestimate actual environmental impacts to fish and wildlife habitat. However, the level of information developed at this feasibility stage is sufficient to discern the relative differences in the impacts between alternatives to inform the decision making process and satisfy NEPA and CEQA requirements.

a) Concur. Prior to construction, vegetation impacts and future approved vegetation allowances would specifically determined for all project locations. This would include on site vegetation surveys.

b) Concur in part. Alternatives 9a and 9b include restoring floodflows to the Old Mormon Channel (new flood bypass). The Lower San Joaquin River Project would not include ecosystem restoration. However, portions of the channel may be suitable for inclusion in the mitigation plan. SJAFCA may have an interest in restoring habitat within the flood bypass as a separate project.

c) Concur. During PED, field surveys would be completed to specifically assess vegetation losses. The scope of these surveys would be coordinated with

USFWS, NMFS, and CDFW. If appropriate, a habitat evaluation procedure study would also be completed.

d) Concur. Mitigation that would avoid, minimize, rectify or compensate for potential adverse impacts that have been identified in this draft report. A full mitigation and monitoring plan related to habitat elements will be developed for the recommended plan (preferred alternative). The plan will be coordinated with USFWS, NMFS, and CDFW, and will be included as an appendix to the final report.

e) Concur. Chapter 5 of this draft report includes a complete assessment of impacts for all alternatives.

f) Concur. Chapter 4, Section 4.1.4, generally describes staging and borrow areas needed to implement the alternatives included in the final array of alternatives. Staging and borrow areas would be specifically identified and evaluated during PED.

USFWS Recommendation 2: Develop a setback levee alternative for alternatives which include the RD 17 work element;

Response: Concur, in part. Setback levee measures were considered during the plan formulation process. One modest setback is included in RD 17 in all of the "b" alternatives. The costs vs benefits of constructing an extensive setback levee caused these measures to be screened out of more detailed analysis during the plan formulation process. For this reason, extensive setback levees are not part of any of the final array of alternatives.

USFWS Recommendation 3: Initiate section 7 consultation with the Service on the effects of project construction, operation, and maintenance, on federally-listed species;

Response: Concur. As part of this Feasibility Study, USACE will request to initiate Section 7 consultation with the Service on the potential effects of project construction, operation, and maintenance, on federally-listed species.

USFWS Recommendation 4: Conduct appropriate consultation with the CDFW on effects to State-listed species, and with NMFS, for effects to anadromous fisheries under their jurisdiction.

Response: Concur. SJAFCA and CVFPB as CEQA lead agencies will consult with CDFW, as appropriate, on potential project effects to State-listed species. As part of this Feasibility Study, USACE will request to initiate Section 7 consultation with NMFS on the potential effects of project on federally-listed species.

USFWS Recommendation 5: Develop enhancement and restoration opportunities for incorporation to the maximum extent possible into the preferred alternative for the project.

Response: Concur. Opportunities for restoration were considered during the plan formulation process, however, opportunities to incorporate ecosystem restoration into the preferred alternative (Tentatively Selected Plan) are severely constrained due to the proximity of the levee system to both the waterways and the highly urbanized Stockton area. Therefore, restoration actions are not included in the proposed action.

The no action alternative would not directly impact endangered and/or hreatened species.

(8) Actions to Minimize Impacts

Many mitigation measures to avoid and minimize impacts to the aquatic environment, as well as, compensatory mitigation measures in order to compensate for unavoidable impacts are proposed. Mitigation measures are listed in Sections 5.5, 5.10, 5.11, and 5.12 of the Draft FR/EIS/EIR.

f. Proposed Disposal Site Determinations

(1) Mixing Zone Size Determination

Not applicable.

(2) Determination of Compliance with Applicable Water Quality Standards

The fill material would not violate Environmental Protection Agency or State water quality standards or violate the primary drinking water standards of the Safe Drinking Water Act (42 USC 300f - 300j). Project design, compliance with State water quality thresholds and standard construction and erosion practices would preclude the introduction of substances into surrounding waters.

The proposed project would not affect existing or potential water supplies, nor would the other alternatives, including the no-action alternative.

(3) Potential Effects on Human Use Characteristics

a) Municipal and Private Water Supplies

The fill material would not violate Environmental Protection Agency or State water quality standards or violate the primary drinking water standards of the Safe Drinking Water Act (42 USC 300f – 300j).

Project design, compliance with State water quality thresholds and standard construction and erosion practices would preclude the introduction of substances into surrounding waters. Materials removed for disposal off-site would be disposed of in an appropriate landfill or other upland area.

b) Recreation and Commercial Fisheries

The study area is heavily used for recreational fishing. A description of these game fish is provided in the FR/EIS/EIR Fisheries, Section 5.11.

Temporary disruption of these activities would occur during construction when the levee crown and adjacent construction and staging areas are closed to public access. Even if the recreation areas themselves are not closed, proximity to construction equipment and activities may degrade recreational experiences. However, this effect is temporary and there are alternative locations for these types of recreation activities in the city.

Alternatives 7a, 8a, and 9a would result in similar impacts to recreational fisheries. The no-action alternative would result in no impacts to recreational fisheries.

c) Water-related recreation

In addition to recreational fishing, the study area is used for picnicking, walking and boating.

All action alternatives (Alternatives 7a, 8a, and 9a) are similar in their potential impacts to recreation. All alternatives include construction of in-water gated closure structures in Fourteenmile Slough and Smith Canal. These could temporarily disrupt recreational boating and personal watercraft use. Temporary disruption of recreational boating would result from the presence of construction vehicles, equipment, and personnel in and adjacent to the Smith Canal, Fourteenmile Slough, and Tenmile Slough, as well as temporary construction effects on water quality (i.e., increased turbidity from suspended materials) in the canal, sloughs, and in the Stockton Deepwater Ship Channel near Smith Canal.

The boat launch, just inside Smith Canal, provides a vehicle-accessible boat ramp. Temporary closure of the boat launch facility during construction of the closure structure at Smith Canal and the floodwall on Dad’s Point would affect recreational boaters as well as general passive recreation at Dad’s Point. Coordination with the City of Stockton and the facility manager would occur prior to closing the facility to any recreational vehicle and reducing access to recreational boating and other recreational opportunities in the project vicinity. Implementation of the avoidance, minimization, and other mitigation measures would reduce impacts to less than significant.

The impacts on recreation for Alternative 8a would be the same as those for Alternative 7a, with the addition of impacts associated with the levee improvements along additional portions of Lower Calaveras river and the Stockton Diverting Canal. Impacts on recreation for Alternative 9a would be the same as those for Alternative 7a except that there would be additional impacts associated with construction of the diversion structure on the Stockton Diverting Canal and construction of a flood bypass through Old Mormon Channel.

The no-action alternative would result in no impacts to other water related recreation.

d) Aesthetics

Construction activities under Alternatives 7a, 8a, and 9a would introduce considerable heavy equipment and associated vehicles, including dozers, graders, cranes, scrapers, and trucks into the views of adjacent residents, recreationists, motorists, and businesses. The equipment would be visible throughout the construction season. Presence of the equipment would temporarily degrade the visual quality of the study area. The construction impacts on aesthetics would be temporary, and would primarily affect local residents or recreationists in the immediate vicinity.

Construction has the potential to substantially degrade the existing visual character or quality of the levee reaches and surroundings for viewer groups for two other reasons: 1) a new levee embankment or flood structure (e.g., flood wall, adjacent levee raise, setback levee) would be present, and 2) construction would require the removal of all vegetation the levee surfaces where improvements are to be made and all woody vegetation from the all levee surfaces and fifteen feet water-ward of the levee toe and ten to twenty feet landward of the levee toe. This would degrade the visual character of the area and obstruct views. For example, the flood wall constructed on Dad’s Point at the mouth of Smith Canal could obstruct views of the Stockton Deepwater Ship Channel and the Port of Stockton and change the quality of the visual character of these areas.

The impacts on recreation for Alternative 8a would be the same as those for Alternative 7a, with the addition of impacts associated with the levee improvements along additional portions of Lower Calaveras river and the Stockton Diverting Canal. Impacts on recreation for Alternative 9a would be the same as those for Alternative 7a except that there would be additional impacts associated with construction of the diversion structure on the Stockton Diverting Canal and construction of a flood bypass through Old Mormon Channel.

The no-action alternative would not alter the aesthetics and therefore would have no impacts.

e) Parks, National and Historic Monuments, National Seashores, Wilderness Areas, Research Sites, and Similar Preserves.

Not applicable.

g. Determination of Cumulative Effects on the Aquatic Ecosystem

Effects of the proposed action include reductions in nearshore aquatic and riparian habitat that is used by aquatic and terrestrial species.

Public and private in-water gates exist throughout the San Francisco Estuary. They are designed to manage water quality and to reduce flood risk.

A number of other commercial and private activities, including recreation, as well as urban and rural development, could potentially affect listed species in the San Joaquin River basin. Levee maintenance activities by state agencies and local reclamation districts are likely to continue, although any effects on listed species will be addressed through Section 10 or Section 7 (in cases where a federal permit is required) of the ESA. Ongoing non-federal activities that affect listed salmonids, green sturgeon, delta smelt, valley elderberry longhorn beetle, giant garter snake and their habitat, will likely continue in the short-term, at intensities similar to those of recent years.

Potential cumulative effects on fish may include any continuing or future non-federal diversions of water that may entrain adult or larval fish or that may incrementally decrease outflows, thus changing the position of habitat for these species. Water diversions through intakes serving numerous small, private agricultural lands and duck clubs in the San Francisco Estuary and upstream of the estuary contribute to these cumulative effects. These diversions also include municipal and industrial uses and power production. The introduction of exotic species may also occur under numerous circumstances. Exotic species can displace native species that provide food for larval fish. Beneficial impacts on fish accrue from the Federal, state and local efforts to restore fisheries habitat in the upper San Joaquin River watershed, and remove fish passage barriers along the Lower Calaveras River and Mormon Channel. Reintroduction of Spring-run Chinook salmon may restore this fishery to the San Joaquin River system.

Potential cumulative effects on all species discussed above could include: wave action in the channels and sloughs caused by boats that may degrade riparian and wetland habitat and erode banks; dumping of domestic and industrial garbage; land uses that result in increased discharges of pesticides, herbicides, oil, and other contaminants; and conversion of riparian areas for urban development. In addition, routine vegetation clearing and mowing associated with agricultural practices may affect or remove habitat for the valley elderberry longhorn beetle and giant garter snake.

h. Determination of Secondary Effects on the Aquatic Ecosystem

Under Alternatives 7a, 8a, and 9a all trees and shrubs would be removed from the levee crown and slopes, and from within fifteen feet water-ward of the levee toe and from within twenty feet of the landside levee toe. Vegetation would be removed in order to construct the levee improvements and to establish a Vegetation ETL-compliant no vegetation zone and landside operations, maintenance, and emergency access corridor. At the end of each construction season, disturbed area would be seeded with native herbaceous plants. Compensatory mitigation would be accomplished through a combination of on-site plantings where feasible, mitigation bank credits, and off-site plantings.

Risk exists for the unintentional placement of dredge and/or fill outside of the proposed project area. Unintentional placement could result in additional adverse impacts to water quality, aquatic and other wildlife habitat, recreation, aesthetics and air quality. To reduce the risk of such impacts, contract specifications would require the contractor to mark the project boundaries, and that the contractor install erosion control (i.e. silt fencing, silt curtains) where possible within any standing waters.

III. Findings of Compliance or Non-Compliance with the Restrictions on Discharge

(1) No significant adaptations of the guidelines were made relative to this evaluation.

(2) No practicable alternative exists which meets the study objectives that does not involve discharge of fill into waters of the United States.

(3) The discharges of fill materials would not cause or contribute to, after consideration of disposal site dilution and dispersion, violation of any applicable State water quality standards for waters. The discharge operations would not violate the Toxic Effluent Standards of Section 307 of the Clean Water Act.

(4) The placement of fill materials would not result in significant adverse effects on human health and welfare, including municipal and private water supplies; recreational and commercial fishing; fish, shellfish, and wildlife populations and habitat, and special aquatic sites. The life stages of aquatic species and other wildlife would not be adversely affected in the San Joaquin River system.

Temporary inhibition of life stages would occur within a localized project area. Significant adverse effects on aquatic ecosystem diversity, productivity and stability, and recreational, aesthetic, and economic values would not occur.

(5) The placement of fill materials in the project area(s) would not jeopardize the continued existence of any species listed as threatened or endangered or result in the likelihood of destruction or adverse modification of any critical habitat as specified by the Endangered Species Act of 1973.

(6) Appropriate steps to minimize potential adverse effects of the discharge on aquatic systems will be implemented.

(7) On the basis of the guidelines the proposed disposal site for the discharge of dredged material is specified as complying with the requirements of the guidelines with the inclusion of appropriate and practicable conditions to minimize pollution or adverse effects to the aquatic ecosystem.

APPENDIX A-5

SPECIAL STATUS SPECIES LISTS

LOWER SAN JOAQUIN FEASIBILITY STUDY

United States Department of the Interior

FISH AND WILDLIFE SERVICE

Sacramento Fish and Wildlife Office
2800 Cottage Way, Room W-2605
Sacramento, California 95825

June 3, 2014

Document Number: 140603040032

Brad Johnson
U.S. Army Corps of Engineers
1325 J Street
Sacramento, CA 95630

Subject: Species List for Lower San Joaquin River Feasibility Study

Dear: Interested party

We are sending this official species list in response to your June 3, 2014 request for information about endangered and threatened species. The list covers the California counties and/or U.S. Geological Survey 7½ minute quad or quads you requested.

Our database was developed primarily to assist Federal agencies that are consulting with us. Therefore, our lists include all of the sensitive species that have been found in a certain area *and also ones that may be affected by projects in the area*. For example, a fish may be on the list for a quad if it lives somewhere downstream from that quad. Birds are included even if they only migrate through an area. In other words, we include all of the species we want people to consider when they do something that affects the environment.

Please read Important Information About Your Species List (below). It explains how we made the list and describes your responsibilities under the Endangered Species Act.

Our database is constantly updated as species are proposed, listed and delisted. If you address proposed and candidate species in your planning, this should not be a problem. However, we recommend that you get an updated list every 90 days. That would be September 01, 2014.

Please contact us if your project may affect endangered or threatened species or if you have any questions about the attached list or your responsibilities under the Endangered Species Act. A list of Endangered Species Program contacts can be found http://www.fws.gov/sacramento/es/Branch-Contacts/es_branch-contacts.htm.

Endangered Species Division

U.S. Fish & Wildlife Service
Sacramento Fish & Wildlife Office
Federal Endangered and Threatened Species that Occur in or may be Affected by Projects in the
LODI SOUTH (479D)
U.S.G.S. 7 1/2 Minute Quad

Report Date: April 24, 2014

Listed Species

Invertebrates

Branchinecta lynchi
vernal pool fairy shrimp (T)

Desmocerus californicus dimorphus
valley elderberry longhorn beetle (T)

Lepidurus packardi
vernal pool tadpole shrimp (E)

Fish

Acipenser medirostris
green sturgeon (T) (NMFS)

Hypomesus transpacificus
Critical habitat, delta smelt (X)
delta smelt (T)

Oncorhynchus mykiss
Central Valley steelhead (T) (NMFS)
Critical habitat, Central Valley steelhead (X) (NMFS)

Oncorhynchus tshawytscha
Central Valley spring-run chinook salmon (T) (NMFS)
winter-run chinook salmon, Sacramento River (E) (NMFS)

Amphibians

Ambystoma californiense
California tiger salamander, central population (T)

Rana draytonii
California red-legged frog (T)

Reptiles

Thamnophis gigas
giant garter snake (T)

Mammals

Sylvilagus bachmani riparius
riparian brush rabbit (E)

Plants

Castilleja campestris ssp. succulenta
succulent (=fleshy) owl's-clover (T)

Key:

- (E) Endangered - Listed as being in danger of extinction.
- (T) Threatened - Listed as likely to become endangered within the foreseeable future.
- (P) Proposed - Officially proposed in the Federal Register for listing as endangered or threatened.
- (NMFS) Species under the Jurisdiction of the National Oceanic & Atmospheric Administration Fisheries Service. Consult with them directly about these species.
- Critical Habitat - Area essential to the conservation of a species.
- (PX) Proposed Critical Habitat - The species is already listed. Critical habitat is being proposed for it.
- (C) Candidate - Candidate to become a proposed species.
- (V) Vacated by a court order. Not currently in effect. Being reviewed by the Service.
- (X) Critical Habitat designated for this species

U.S. Fish & Wildlife Service
Sacramento Fish & Wildlife Office
Federal Endangered and Threatened Species that Occur in or may be Affected by Projects in the
WATERLOO (478C)
U.S.G.S. 7 1/2 Minute Quad

Report Date: April 24, 2014

Listed Species

Invertebrates

Branchinecta lynchi
vernal pool fairy shrimp (T)

Desmocerus californicus dimorphus
valley elderberry longhorn beetle (T)

Lepidurus packardi
vernal pool tadpole shrimp (E)

Fish

Hypomesus transpacificus
delta smelt (T)

Oncorhynchus mykiss
Central Valley steelhead (T) (NMFS)
Critical habitat, Central Valley steelhead (X) (NMFS)

Oncorhynchus tshawytscha
Central Valley spring-run chinook salmon (T) (NMFS)
winter-run chinook salmon, Sacramento River (E) (NMFS)

Amphibians

Ambystoma californiense
California tiger salamander, central population (T)

Rana draytonii
California red-legged frog (T)

Reptiles

Thamnophis gigas

giant garter snake (T)

Mammals

Sylvilagus bachmani riparius
riparian brush rabbit (E)

Plants

Castilleja campestris ssp. succulenta
succulent (=fleshy) owl's-clover (T)

Key:

- (E) Endangered - Listed as being in danger of extinction.
- (T) Threatened - Listed as likely to become endangered within the foreseeable future.
- (P) Proposed - Officially proposed in the Federal Register for listing as endangered or threatened.
- (NMFS) Species under the Jurisdiction of the National Oceanic & Atmospheric Administration Fisheries Service. Consult with them directly about these species.
- Critical Habitat - Area essential to the conservation of a species.
- (PX) Proposed Critical Habitat - The species is already listed. Critical habitat is being proposed for it.
- (C) Candidate - Candidate to become a proposed species.
- (V) Vacated by a court order. Not currently in effect. Being reviewed by the Service.
- (X) Critical Habitat designated for this species

U.S. Fish & Wildlife Service

Sacramento Fish & Wildlife Office

Federal Endangered and Threatened Species that Occur in or may be Affected by Projects in the

STOCKTON WEST (462A)

U.S.G.S. 7 1/2 Minute Quad

Report Date: April 24, 2014

Listed Species

Invertebrates

Branchinecta lynchi
vernal pool fairy shrimp (T)

Desmocerus californicus dimorphus
valley elderberry longhorn beetle (T)

Lepidurus packardi
vernal pool tadpole shrimp (E)

Fish

Acipenser medirostris
green sturgeon (T) (NMFS)

Hypomesus transpacificus
Critical habitat, delta smelt (X)
delta smelt (T)

Oncorhynchus mykiss
Central Valley steelhead (T) (NMFS)
Critical habitat, Central Valley steelhead (X) (NMFS)

Oncorhynchus tshawytscha
Central Valley spring-run chinook salmon (T) (NMFS)
winter-run chinook salmon, Sacramento River (E) (NMFS)

Amphibians

Ambystoma californiense
California tiger salamander, central population (T)

Rana draytonii
California red-legged frog (T)

Reptiles

Thamnophis gigas
giant garter snake (T)

Mammals

Sylvilagus bachmani riparius
riparian brush rabbit (E)

Plants

Cordylanthus palmatus
palmate-bracted bird's-beak (E)

Key:

- (E) Endangered - Listed as being in danger of extinction.
- (T) Threatened - Listed as likely to become endangered within the foreseeable future.
- (P) Proposed - Officially proposed in the Federal Register for listing as endangered or threatened.
- (NMFS) Species under the Jurisdiction of the National Oceanic & Atmospheric Administration Fisheries Service. Consult with them directly about these species.
- Critical Habitat - Area essential to the conservation of a species.
- (PX) Proposed Critical Habitat - The species is already listed. Critical habitat is being proposed for it.
- (C) Candidate - Candidate to become a proposed species.
- (V) Vacated by a court order. Not currently in effect. Being reviewed by the Service.
- (X) Critical Habitat designated for this species

U.S. Fish & Wildlife Service
Sacramento Fish & Wildlife Office
Federal Endangered and Threatened Species that Occur in or may be Affected by Projects in the
STOCKTON EAST (461B)
U.S.G.S. 7 1/2 Minute Quad

Report Date: April 24, 2014

Listed Species

Invertebrates

Branchinecta lynchi
vernal pool fairy shrimp (T)

Desmocerus californicus dimorphus
valley elderberry longhorn beetle (T)

Lepidurus packardi
vernal pool tadpole shrimp (E)

Fish

Acipenser medirostris
green sturgeon (T) (NMFS)

Hypomesus transpacificus
delta smelt (T)

Oncorhynchus mykiss
Central Valley steelhead (T) (NMFS)
Critical habitat, Central Valley steelhead (X) (NMFS)

Oncorhynchus tshawytscha
Central Valley spring-run chinook salmon (T) (NMFS)
winter-run chinook salmon, Sacramento River (E) (NMFS)

Amphibians

Ambystoma californiense
California tiger salamander, central population (T)

Rana draytonii
California red-legged frog (T)

Reptiles

Thamnophis gigas
giant garter snake (T)

Mammals

Sylvilagus bachmani riparius
riparian brush rabbit (E)

Key:

- (E) Endangered - Listed as being in danger of extinction.
- (T) Threatened - Listed as likely to become endangered within the foreseeable future.
- (P) Proposed - Officially proposed in the Federal Register for listing as endangered or threatened.
- (NMFS) Species under the Jurisdiction of the National Oceanic & Atmospheric Administration Fisheries Service. Consult with them directly about these species.
- Critical Habitat - Area essential to the conservation of a species.
- (PX) Proposed Critical Habitat - The species is already listed. Critical habitat is being proposed for it.
- (C) Candidate - Candidate to become a proposed species.
- (V) Vacated by a court order. Not currently in effect. Being reviewed by the Service.
- (X) Critical Habitat designated for this species

U.S. Fish & Wildlife Service
Sacramento Fish & Wildlife Office
Federal Endangered and Threatened Species that Occur in or may be Affected by Projects in the
LATHROP (462D)
U.S.G.S. 7 1/2 Minute Quad

Report Date: April 24, 2014

Listed Species

Invertebrates

Branchinecta lynchi
vernal pool fairy shrimp (T)

Desmocerus californicus dimorphus
valley elderberry longhorn beetle (T)

Lepidurus packardi
vernal pool tadpole shrimp (E)

Fish

Acipenser medirostris
green sturgeon (T) (NMFS)

Hypomesus transpacificus
Critical habitat, delta smelt (X)
delta smelt (T)

Oncorhynchus mykiss
Central Valley steelhead (T) (NMFS)
Critical habitat, Central Valley steelhead (X) (NMFS)

Oncorhynchus tshawytscha
Central Valley spring-run chinook salmon (T) (NMFS)
winter-run chinook salmon, Sacramento River (E) (NMFS)

Amphibians

Ambystoma californiense
California tiger salamander, central population (T)

Rana draytonii
California red-legged frog (T)

Reptiles

Thamnophis gigas
giant garter snake (T)

Mammals

Sylvilagus bachmani riparius
riparian brush rabbit (E)

Key:

- (E) Endangered - Listed as being in danger of extinction.
- (T) Threatened - Listed as likely to become endangered within the foreseeable future.
- (P) Proposed - Officially proposed in the Federal Register for listing as endangered or threatened.
- (NMFS) Species under the Jurisdiction of the National Oceanic & Atmospheric Administration Fisheries Service. Consult with them directly about these species.
- Critical Habitat - Area essential to the conservation of a species.
- (PX) Proposed Critical Habitat - The species is already listed. Critical habitat is being proposed for it.
- (C) Candidate - Candidate to become a proposed species.
- (V) Vacated by a court order. Not currently in effect. Being reviewed by the Service.
- (X) Critical Habitat designated for this species

U.S. Fish & Wildlife Service

Sacramento Fish & Wildlife Office

Federal Endangered and Threatened Species that Occur in or may be Affected by Projects in the MANTECA (461C) U.S.G.S. 7 1/2 Minute Quad

Report Date: April 24, 2014

Listed Species

Invertebrates

Branchinecta lynchi
vernal pool fairy shrimp (T)

Desmocerus californicus dimorphus
valley elderberry longhorn beetle (T)

Lepidurus packardi
vernal pool tadpole shrimp (E)

Fish

Acipenser medirostris
green sturgeon (T) (NMFS)

Hypomesus transpacificus
delta smelt (T)

Oncorhynchus mykiss
Central Valley steelhead (T) (NMFS)

Oncorhynchus tshawytscha
Central Valley spring-run chinook salmon (T) (NMFS)
winter-run chinook salmon, Sacramento River (E) (NMFS)

Amphibians

Ambystoma californiense
California tiger salamander, central population (T)

Rana draytonii
California red-legged frog (T)

Reptiles

Thamnophis gigas
giant garter snake (T)

Key:

- (E) Endangered - Listed as being in danger of extinction.
- (T) Threatened - Listed as likely to become endangered within the foreseeable future.
- (P) Proposed - Officially proposed in the Federal Register for listing as endangered or threatened.
- (NMFS) Species under the Jurisdiction of the National Oceanic & Atmospheric Administration Fisheries Service. Consult with them directly about these species.
- Critical Habitat - Area essential to the conservation of a species.
- (PX) Proposed Critical Habitat - The species is already listed. Critical habitat is being proposed for it.
- (C) Candidate - Candidate to become a proposed species.
- (V) Vacated by a court order. Not currently in effect. Being reviewed by the Service.
- (X) Critical Habitat designated for this species

United States Department of the Interior

FISH AND WILDLIFE SERVICE

Sacramento Fish and Wildlife Office
2800 Cottage Way, Room W-2605
Sacramento, California 95825

September 12, 2014

Document Number: 140912125652

David Colby
US Army Corps of Engineers
1325 J Street
Sacramento, CA 95814

Subject: Species List for Lower San Joaquin

Dear: Mr. Colby

We are sending this official species list in response to your September 12, 2014 request for information about endangered and threatened species. The list covers the California counties and/or U.S. Geological Survey 7½ minute quad or quads you requested.

Our database was developed primarily to assist Federal agencies that are consulting with us. Therefore, our lists include all of the sensitive species that have been found in a certain area *and also ones that may be affected by projects in the area*. For example, a fish may be on the list for a quad if it lives somewhere downstream from that quad. Birds are included even if they only migrate through an area. In other words, we include all of the species we want people to consider when they do something that affects the environment.

Please read Important Information About Your Species List (below). It explains how we made the list and describes your responsibilities under the Endangered Species Act.

Our database is constantly updated as species are proposed, listed and delisted. If you address proposed and candidate species in your planning, this should not be a problem. However, we recommend that you get an updated list every 90 days. That would be December 11, 2014.

Please contact us if your project may affect endangered or threatened species or if you have any questions about the attached list or your responsibilities under the Endangered Species Act. A list of Endangered Species Program contacts can be found http://www.fws.gov/sacramento/es/Branch-Contacts/es_branch-contacts.htm.

Endangered Species Division

U.S. Fish & Wildlife Service

Sacramento Fish & Wildlife Office

Federal Endangered and Threatened Species that Occur in or may be Affected by Projects in the Counties and/or U.S.G.S. 7 1/2 Minute Quads you requested

Document Number: 140912122518

Current as of: September 12, 2014

Quad Lists

Listed Species

Invertebrates

- *Branchinecta lynchi*
  - vernal pool fairy shrimp (T)
- *Desmocerus californicus dimorphus*
  - valley elderberry longhorn beetle (T)
- *Lepidurus packardi*
  - vernal pool tadpole shrimp (E)

Fish

- *Acipenser medirostris*
  - green sturgeon (T) (NMFS)
- *Hypomesus transpacificus*
  - Critical habitat, delta smelt (X)
  - delta smelt (T)
- *Oncorhynchus mykiss*
  - Central Valley steelhead (T) (NMFS)
  - Critical habitat, Central Valley steelhead (X) (NMFS)
- *Oncorhynchus tshawytscha*
  - Central Valley spring-run chinook salmon (T) (NMFS)
  - winter-run chinook salmon, Sacramento River (E) (NMFS)

Amphibians

- *Ambystoma californiense*
  - California tiger salamander, central population (T)
- *Rana draytonii*
  - California red-legged frog (T)

Reptiles

- *Thamnophis gigas*
  - giant garter snake (T)

Mammals

- *Sylvilagus bachmani riparius*
  - riparian brush rabbit (E)

Plants

- *Castilleja campestris ssp. succulenta*
  - succulent (=fleshy) owl's-clover (T)

Quads Containing Listed, Proposed or Candidate Species:

LODI SOUTH (479D)

County Lists

No county species lists requested.

Key:

(E) *Endangered* - Listed as being in danger of extinction.

(T) *Threatened* - Listed as likely to become endangered within the foreseeable future.

(P) *Proposed* - Officially proposed in the Federal Register for listing as endangered or threatened.

(NMFS) Species under the Jurisdiction of the National Oceanic & Atmospheric Administration Fisheries Service. Consult with them directly about these species.

Critical Habitat - Area essential to the conservation of a species.

(PX) *Proposed Critical Habitat* - The species is already listed. Critical habitat is being proposed for it.

(C) *Candidate* - Candidate to become a proposed species.

(V) Vacated by a court order. Not currently in effect. Being reviewed by the Service.

(X) *Critical Habitat* designated for this species

Important Information About Your Species List

How We Make Species Lists

We store information about endangered and threatened species lists by U.S. Geological Survey 7½ minute quads. The United States is divided into these quads, which are about the size of San Francisco.

The animals on your species list are ones that occur within, **or may be affected by** projects within, the quads covered by the list.

- Fish and other aquatic species appear on your list if they are in the same watershed as your quad or if water use in your quad might affect them.
- Amphibians will be on the list for a quad or county if pesticides applied in that area may be carried to their habitat by air currents.
- Birds are shown regardless of whether they are resident or migratory. Relevant birds on the county list should be considered regardless of whether they appear on a quad list.

Plants

Any plants on your list are ones that have actually been observed in the area covered by the list. Plants may exist in an area without ever having been detected there. You can find out what's in the surrounding quads through the California Native Plant Society's online Inventory of Rare and Endangered Plants.

Surveying

Some of the species on your list may not be affected by your project. A trained biologist and/or botanist, familiar with the habitat requirements of the species on your list, should determine whether they or habitats suitable for them may be affected by your project. We recommend that your surveys include any proposed and candidate species on your list. See our Protocol and Recovery Permits pages.

For plant surveys, we recommend using the Guidelines for Conducting and Reporting

U.S. Fish & Wildlife Service

Sacramento Fish & Wildlife Office

Federal Endangered and Threatened Species that Occur in or may be Affected by Projects in the Counties and/or U.S.G.S. 7 1/2 Minute Quads you requested

Document Number: 140912123306

Current as of: September 12, 2014

Quad Lists

Listed Species

Invertebrates

- *Branchinecta lynchi*
  - vernal pool fairy shrimp (T)
- *Desmocerus californicus dimorphus*
  - valley elderberry longhorn beetle (T)
- *Lepidurus packardi*
  - vernal pool tadpole shrimp (E)

Fish

- *Hypomesus transpacificus*
  - delta smelt (T)
- *Oncorhynchus mykiss*
  - Central Valley steelhead (T) (NMFS)
  - Critical habitat, Central Valley steelhead (X) (NMFS)
- *Oncorhynchus tshawytscha*
  - Central Valley spring-run chinook salmon (T) (NMFS)
  - winter-run chinook salmon, Sacramento River (E) (NMFS)

Amphibians

- *Ambystoma californiense*
  - California tiger salamander, central population (T)
- *Rana draytonii*
  - California red-legged frog (T)

Reptiles

- *Thamnophis gigas*
  - giant garter snake (T)

Mammals

- *Sylvilagus bachmani riparius*
  - riparian brush rabbit (E)

Plants

- *Castilleja campestris ssp. succulenta*
  - succulent (=fleshy) owl's-clover (T)

Quads Containing Listed, Proposed or Candidate Species:

WATERLOO (478C)

U.S. Fish & Wildlife Service

Sacramento Fish & Wildlife Office

Federal Endangered and Threatened Species that Occur in or may be Affected by Projects in the Counties and/or U.S.G.S. 7 1/2 Minute Quads you requested

Document Number: 140912123349

Current as of: September 12, 2014

Quad Lists

Listed Species

Invertebrates

- *Branchinecta lynchi*
  - vernal pool fairy shrimp (T)
- *Desmocerus californicus dimorphus*
  - valley elderberry longhorn beetle (T)
- *Lepidurus packardi*
  - vernal pool tadpole shrimp (E)

Fish

- *Acipenser medirostris*
  - green sturgeon (T) (NMFS)
- *Hypomesus transpacificus*
  - Critical habitat, delta smelt (X)
  - delta smelt (T)
- *Oncorhynchus mykiss*
  - Central Valley steelhead (T) (NMFS)
  - Critical habitat, Central Valley steelhead (X) (NMFS)
- *Oncorhynchus tshawytscha*
  - Central Valley spring-run chinook salmon (T) (NMFS)
  - winter-run chinook salmon, Sacramento River (E) (NMFS)

Amphibians

- *Ambystoma californiense*
  - California tiger salamander, central population (T)
- *Rana draytonii*
  - California red-legged frog (T)

Reptiles

- *Thamnophis gigas*
  - giant garter snake (T)

Mammals

- *Sylvilagus bachmani riparius*
  - riparian brush rabbit (E)

Plants

- *Cordylanthus palmatus*
  - palmate-bracted bird's-beak (E)

Quads Containing Listed, Proposed or Candidate Species:

STOCKTON WEST (462A)

County Lists

No county species lists requested.

Key:

(E) *Endangered* - Listed as being in danger of extinction.

(T) *Threatened* - Listed as likely to become endangered within the foreseeable future.

(P) *Proposed* - Officially proposed in the Federal Register for listing as endangered or threatened.

(NMFS) Species under the Jurisdiction of the National Oceanic & Atmospheric Administration Fisheries Service. Consult with them directly about these species.

Critical Habitat - Area essential to the conservation of a species.

(PX) *Proposed Critical Habitat* - The species is already listed. Critical habitat is being proposed for it.

(C) *Candidate* - Candidate to become a proposed species.

(V) Vacated by a court order. Not currently in effect. Being reviewed by the Service.

(X) *Critical Habitat* designated for this species

Important Information About Your Species List

How We Make Species Lists

We store information about endangered and threatened species lists by U.S. Geological Survey 7½ minute quads. The United States is divided into these quads, which are about the size of San Francisco.

The animals on your species list are ones that occur within, **or may be affected by** projects within, the quads covered by the list.

- Fish and other aquatic species appear on your list if they are in the same watershed as your quad or if water use in your quad might affect them.
- Amphibians will be on the list for a quad or county if pesticides applied in that area may be carried to their habitat by air currents.
- Birds are shown regardless of whether they are resident or migratory. Relevant birds on the county list should be considered regardless of whether they appear on a quad list.

Plants

Any plants on your list are ones that have actually been observed in the area covered by the list. Plants may exist in an area without ever having been detected there. You can find out what's in the surrounding quads through the California Native Plant Society's online Inventory of Rare and Endangered Plants.

Surveying

Some of the species on your list may not be affected by your project. A trained biologist and/or botanist, familiar with the habitat requirements of the species on your list, should determine whether they or habitats suitable for them may be affected by your project. We recommend that your surveys include any proposed and candidate species on your list. See our Protocol and Recovery Permits pages.

For plant surveys, we recommend using the Guidelines for Conducting and Reporting

U.S. Fish & Wildlife Service

Sacramento Fish & Wildlife Office

Federal Endangered and Threatened Species that Occur in or may be Affected by Projects in the Counties and/or U.S.G.S. 7 1/2 Minute Quads you requested

Document Number: 140912123422

Current as of: September 12, 2014

Quad Lists

Listed Species

Invertebrates

Branchinecta lynchi
vernal pool fairy shrimp (T)

Desmocerus californicus dimorphus
valley elderberry longhorn beetle (T)

Lepidurus packardi
vernal pool tadpole shrimp (E)

Fish

Acipenser medirostris
green sturgeon (T) (NMFS)

Hypomesus transpacificus
delta smelt (T)

Oncorhynchus mykiss
Central Valley steelhead (T) (NMFS)
Critical habitat, Central Valley steelhead (X) (NMFS)

Oncorhynchus tshawytscha
Central Valley spring-run chinook salmon (T) (NMFS)
winter-run chinook salmon, Sacramento River (E) (NMFS)

Amphibians

Ambystoma californiense
California tiger salamander, central population (T)

Rana draytonii
California red-legged frog (T)

Reptiles

Thamnophis gigas
giant garter snake (T)

Mammals

Sylvilagus bachmani riparius
riparian brush rabbit (E)

Quads Containing Listed, Proposed or Candidate Species:

STOCKTON EAST (461B)

County Lists

U.S. Fish & Wildlife Service

Sacramento Fish & Wildlife Office

Federal Endangered and Threatened Species that Occur in or may be Affected by Projects in the Counties and/or U.S.G.S. 7 1/2 Minute Quads you requested

Document Number: 140912123453

Current as of: September 12, 2014

Quad Lists

Listed Species

Invertebrates

- *Branchinecta lynchi*
  - vernal pool fairy shrimp (T)
- *Desmocerus californicus dimorphus*
  - valley elderberry longhorn beetle (T)
- *Lepidurus packardi*
  - vernal pool tadpole shrimp (E)

Fish

- *Acipenser medirostris*
  - green sturgeon (T) (NMFS)
- *Hypomesus transpacificus*
  - Critical habitat, delta smelt (X)
  - delta smelt (T)
- *Oncorhynchus mykiss*
  - Central Valley steelhead (T) (NMFS)
  - Critical habitat, Central Valley steelhead (X) (NMFS)
- *Oncorhynchus tshawytscha*
  - Central Valley spring-run chinook salmon (T) (NMFS)
  - winter-run chinook salmon, Sacramento River (E) (NMFS)

Amphibians

- *Ambystoma californiense*
  - California tiger salamander, central population (T)
- *Rana draytonii*
  - California red-legged frog (T)

Reptiles

- *Thamnophis gigas*
  - giant garter snake (T)

Mammals

- *Sylvilagus bachmani riparius*
  - riparian brush rabbit (E)

Quads Containing Listed, Proposed or Candidate Species:

LATHROP (462D)

U.S. Fish & Wildlife Service

Sacramento Fish & Wildlife Office

Federal Endangered and Threatened Species that Occur in or may be Affected by Projects in the Counties and/or U.S.G.S. 7 1/2 Minute Quads you requested

Document Number: 140912123525

Current as of: September 12, 2014

Quad Lists

Listed Species

Invertebrates

- *Branchinecta lynchi*
  - vernal pool fairy shrimp (T)
- *Desmocerus californicus dimorphus*
  - valley elderberry longhorn beetle (T)
- *Lepidurus packardi*
  - vernal pool tadpole shrimp (E)

Fish

- *Acipenser medirostris*
  - green sturgeon (T) (NMFS)
- *Hypomesus transpacificus*
  - delta smelt (T)
- *Oncorhynchus mykiss*
  - Central Valley steelhead (T) (NMFS)
- *Oncorhynchus tshawytscha*
  - Central Valley spring-run chinook salmon (T) (NMFS)
  - winter-run chinook salmon, Sacramento River (E) (NMFS)

Amphibians

- *Ambystoma californiense*
  - California tiger salamander, central population (T)
- *Rana draytonii*
  - California red-legged frog (T)

Reptiles

- *Thamnophis gigas*
  - giant garter snake (T)

Quads Containing Listed, Proposed or Candidate Species:

MANTECA (461C)

County Lists

No county species lists requested.

Key:

(E) *Endangered* - Listed as being in danger of extinction.

CALIFORNIA DEPARTMENT OF FISH and WILDLIFE

Quad is (Stockton East (3712182) or Stockton West (3712183) or Lodi South (3812113) or Lathrop (3712173) or Waterloo (3812112) or Manteca (3712172))

CNDDB Element Query Results

ScientificName	CommonName	ElementCode	OccCount	GlobalRank	StateRank	FederalListingStatus	StateListingStatus	CNPSList	OtherStatus	Habitat
Agelaius tricolor	tricolored blackbird	ABPBXB0020	429	G2G3	S2	None	None		ABC_WLBCC-Watch List of Birds of Conservation Concern | BLM_S-Sensitive | CDFW_SSC-Species of Special Concern | IUCN_EN-Endangered | USFWS_BCC-Birds of Conservation Concern	Freshwater marsh | Marsh & swamp | Swamp | Wetland
Ambystoma californiense	California tiger salamander	AAAAA01180	1094	G2G3	S2S3	Threatened	Threatened		CDFW_SSC-Species of Special Concern | IUCN_VU-Vulnerable	Cismontane woodland | Meadow & seep | Riparian woodland | Valley & foothill grassland | Vernal pool | Wetland
Astragalus tener var. tener	alkali milk-vetch	PDFAB0F8R1	65	G2T2	S2	None	None	1B.2		Alkali playa | Valley & foothill grassland | Vernal pool | Wetland
Athene cunicularia	burrowing owl	ABNSB10010	1850	G4	S2	None	None		BLM_S-Sensitive | CDFW_SSC-Species of Special Concern | IUCN_LC-Least Concern | USFWS_BCC-Birds of Conservation Concern	Coastal prairie | Coastal scrub | Great Basin grassland | Great Basin scrub | Mojavean desert scrub | Sonoran desert scrub | Valley & foothill grassland
Atriplex cordulata var. cordulata	heartscale	PDCHE040B0	68	G3T2	S2	None	None	1B.2	BLM_S-Sensitive	Chenopod scrub | Meadow & seep | Valley & foothill grassland
Atriplex joaquinana	San Joaquin spearscale	PDCHE041F3	109	G2	S2	None	None	1B.2	BLM_S-Sensitive	Alkali playa | Chenopod scrub | Meadow & seep | Valley & foothill grassland
Blepharizonia plumosa	big tarplant	PDAST1C011	48	G2	S2	None	None	1B.1		Valley & foothill grassland
Branchinecta mesovallensis	midvalley fairy shrimp	ICBRA03150	101	G2	S2	None	None			Vernal pool | Wetland
Brasenia schreberi	watershield	PDCAB01010	33	G5	S2	None	None	2B.3		Marsh & swamp | Wetland
Buteo swainsoni	Swainson's hawk	ABNKC19070	2394	G5	S2	None	Threatened		ABC_WLBCC-Watch List of Birds of Conservation Concern | BLM_S-Sensitive | IUCN_LC-Least Concern | USFS_S-Sensitive | USFWS_BCC-Birds of Conservation Concern	Great Basin grassland | Riparian forest | Riparian woodland | Valley & foothill grassland
California macrophylla	round-leaved filaree	PDGER01070	155	G2	S2	None	None	1B.1	BLM_S-Sensitive	Cismontane woodland | Valley & foothill grassland

Chloropyron palmatum	palmate-bracted salty bird's-beak	PDSCR0J0J0	26	G1	S1	Endangered	Endangered	1B.1		Chenopod scrub | Meadow & seep | Valley & foothill grassland | Wetland
Cirsium crassicaule	slough thistle	PDAST2E0U0	19	G2	S2	None	None	1B.1	BLM_S-Sensitive	Chenopod scrub | Freshwater marsh | Marsh & swamp | Riparian scrub | Wetland
Delphinium recurvatum	recurved larkspur	PDRAN0B1J0	96	G3	S3	None	None	1B.2	BLM_S-Sensitive	Chenopod scrub | Cismontane woodland | Valley & foothill grassland
Desmocerus californicus dimorphus	valley elderberry longhorn beetle	IICOL48011	201	G3T2	S2	Threatened	None			Riparian scrub
Elanus leucurus	white-tailed kite	ABNKC06010	158	G5	S3	None	None		BLM_S-Sensitive | CDFW_FP-Fully Protected | IUCN_LC-Least Concern	Cismontane woodland | Marsh & swamp | Riparian woodland | Valley & foothill grassland | Wetland
Eryngium racemosum	Delta button-celery	PDAPI0Z0S0	26	G1Q	S1	None	Endangered	1B.1		Riparian scrub | Wetland
Hibiscus lasiocarpos var. occidentalis	woolly rose-mallow	PDMAL0H0R3	173	G5T2	S2	None	None	1B.2		Freshwater marsh | Marsh & swamp | Wetland
Hypomesus transpacificus	Delta smelt	AFCHB01040	27	G1	S1	Threatened	Endangered		AFS_TH-Threatened | IUCN_EN-Endangered	Aquatic | Estuary
Lathyrus jepsonii var. jepsonii	Delta tule pea	PDFAB250D2	130	G5T2	S2.2	None	None	1B.2		Freshwater marsh | Marsh & swamp | Wetland
Lepidurus packardi	vernal pool tadpole shrimp	ICBRA10010	274	G3	S2S3	Endangered	None		IUCN_EN-Endangered	Valley & foothill grassland | Vernal pool | Wetland
Lilaeopsis masonii	Mason's lilaeopsis	PDAPI19030	196	G2	S2	None	Rare	1B.1		Freshwater marsh | Marsh & swamp | Riparian scrub | Wetland
Lytta moesta	moestan blister beetle	IICOL4C020	12	G2	S2	None	None			Valley & foothill grassland
Melospiza melodia	song sparrow ("Modesto" population)	ABPBXA3010	92	G5	S3?	None	None		CDFW_SSC-Species of Special Concern	
Oncorhynchus mykiss irideus	steelhead - Central Valley DPS	AFCHA0209K	31	G5T2	S2	Threatened	None		AFS_TH-Threatened	Aquatic | Sacramento/San Joaquin flowing waters
Sagittaria sanfordii	Sanford's arrowhead	PMALI040Q0	93	G3	S3	None	None	1B.2	BLM_S-Sensitive	Marsh & swamp | Wetland
Spirinchus thaleichthys	longfin smelt	AFCHB03010	45	G5	S1	Candidate	Threatened		CDFW_SSC-Species of Special Concern	Aquatic | Estuary
Sylvilagus bachmani riparius	riparian brush rabbit	AMAEB01021	16	G5T1	S1	Endangered	Endangered			Riparian forest
Symphyotrichum lentum	Suisun Marsh aster	PDASTE8470	172	G2	S2	None	None	1B.2		Brackish marsh | Freshwater marsh | Marsh & swamp | Wetland
Thamnophis gigas	giant garter snake	ARADB36150	271	G2G3	S2S3	Threatened	Threatened		IUCN_VU-Vulnerable	Marsh & swamp | Riparian scrub | Wetland
Trichocoronis wrightii var. wrightii	Wright's trichocoronis	PDAST9F031	9	G4T3	S1	None	None	2B.1		Marsh & swamp | Meadow & seep | Riparian forest | Vernal pool | Wetland
Trifolium hydrophilum	saline clover	PDFAB400R5	49	G2	S2	None	None	1B.2		Marsh & swamp | Valley & foothill grassland | Vernal pool | Wetland

Valley Oak Woodland	Valley Oak Woodland	CTT71130CA	91	G3	S2.1	None	None			Cismontane woodland
Vireo bellii pusillus	least Bell's vireo	ABPBW01114	410	G5T2	S2	Endangered	Endangered		ABC_WLBCC-Watch List of Birds of Conservation Concern | IUCN_NT-Near Threatened	Riparian forest | Riparian scrub | Riparian woodland
Xanthocephalus xanthocephalus	yellow-headed blackbird	ABPBXB3010	11	G5	S3S4	None	None		CDFW_SSC-Species of Special Concern | IUCN_LC-Least Concern	Marsh & swamp | Wetland

PLATES

LOWER SAN JOAQUIN RIVER INTERIM FEASIBILITY STUDY

LSJ Segements

- ▬ No Geotech Fix
- ▬ Seepage Berm, Geometric/Ht Fix
- ▬ Centerline Cut-Off Wall
- ▬ Centerline Cut-Off Wall, Geometric/Ht. Fix
- ▬ New Levee
- ▬ Seismic Fix
- ▬ Seismic Fix/Geometric Fix

- ● Control Structure
- ▬ Sea Level Rise Protection
- ■ Erosion Protection
- ■ 200 Year + 3' Raise

Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

N

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

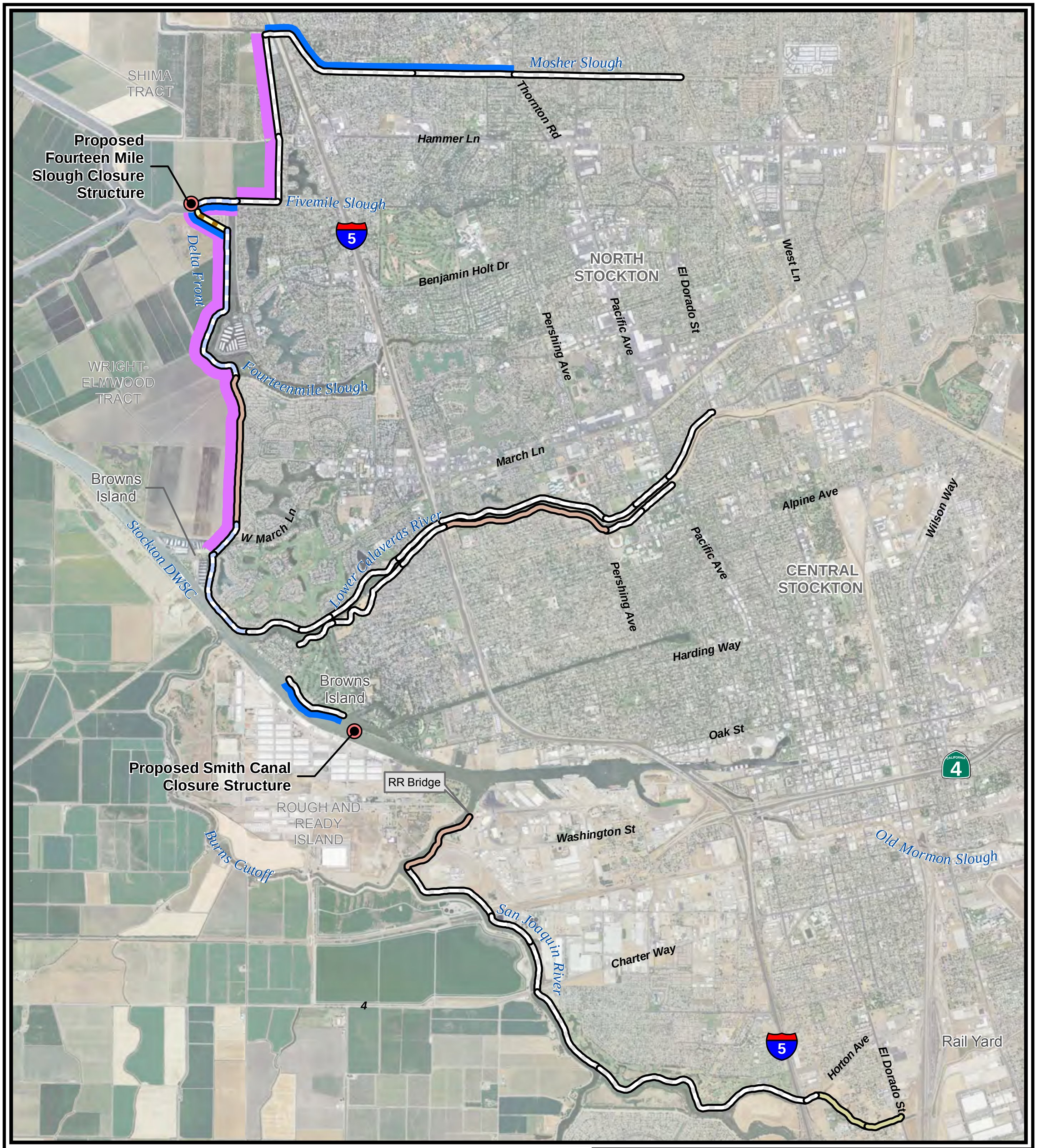
LSJ LEVEE REACHES
ALTERNATIVE 7a
NORTH STOCKTON - EAST

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 1



LSJ Segements

- No Geotech Fix
- Seepage Berm, Geometric/Ht Fix
- Centerline Cut-Off Wall
- Centerline Cut-Off Wall, Geometric/Ht. Fix
- New Levee
- Seismic Fix
- Seismic Fix/Geometric Fix

- ● Control Structure
- Sea Level Rise Protection
- 🟧 200 Year + 3' Raise
- 🟪 Erosion Protection

Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

N

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

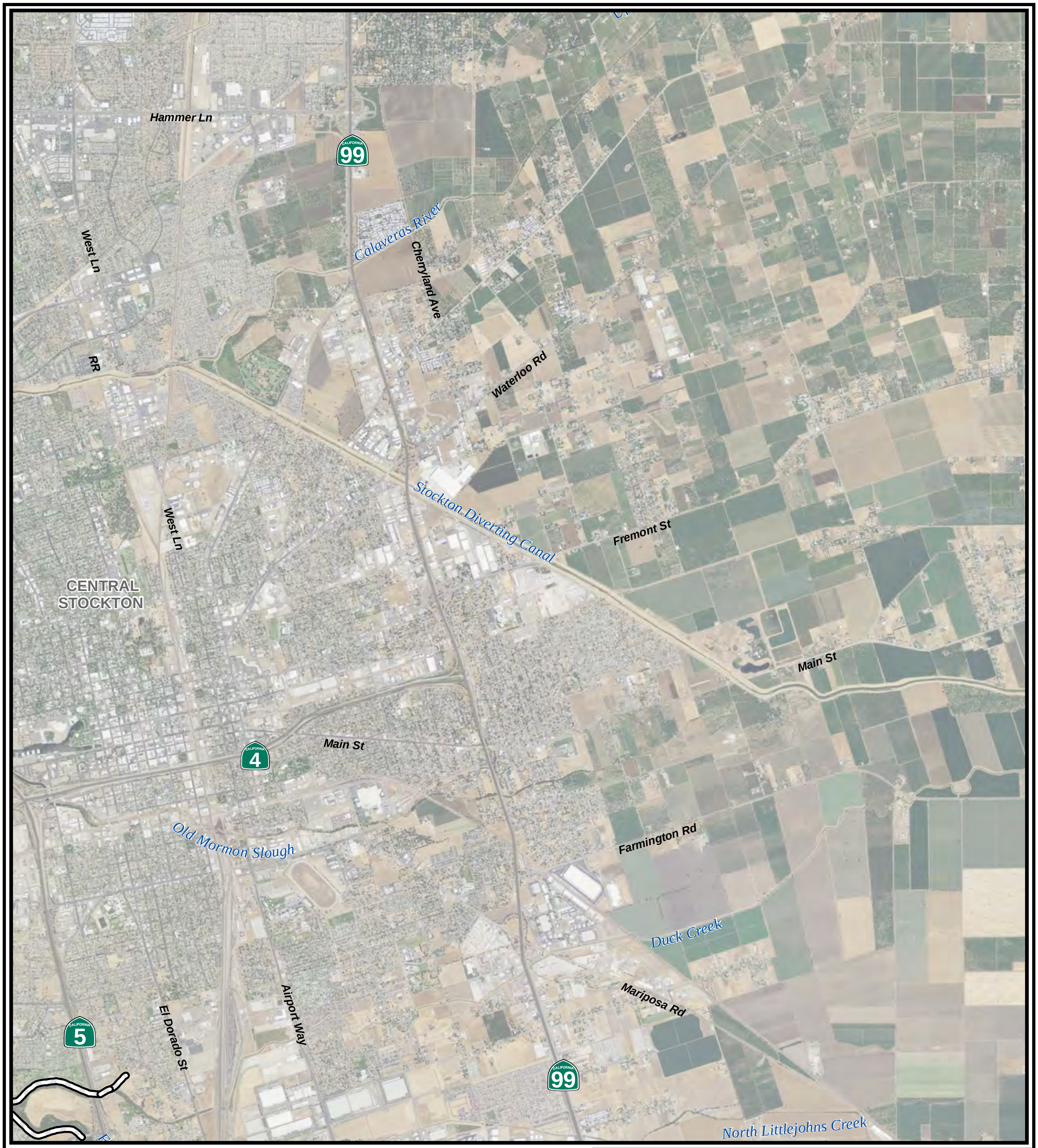
LSJ LEVEE REACHES
ALTERNATIVE 7a
NORTH STOCKTON - WEST

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 2



LSJ Segements

- ▭ No Geotech Fix
- ▭ Seepage Berm, Geometric/Ht Fix
- ▭ Centerline Cut-Off Wall
- ▭ Centerline Cut-Off Wall, Geometric/Ht. Fix
- ▭ New Levee
- ▭ Seismic Fix
- ▭ Seismic Fix/Geometric Fix

- ● Control Structure
- 🟦 Sea Level Rise Protection
- 🟧 200 Year + 3' Raise
- 🟪 Erosion Protection

Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

N

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

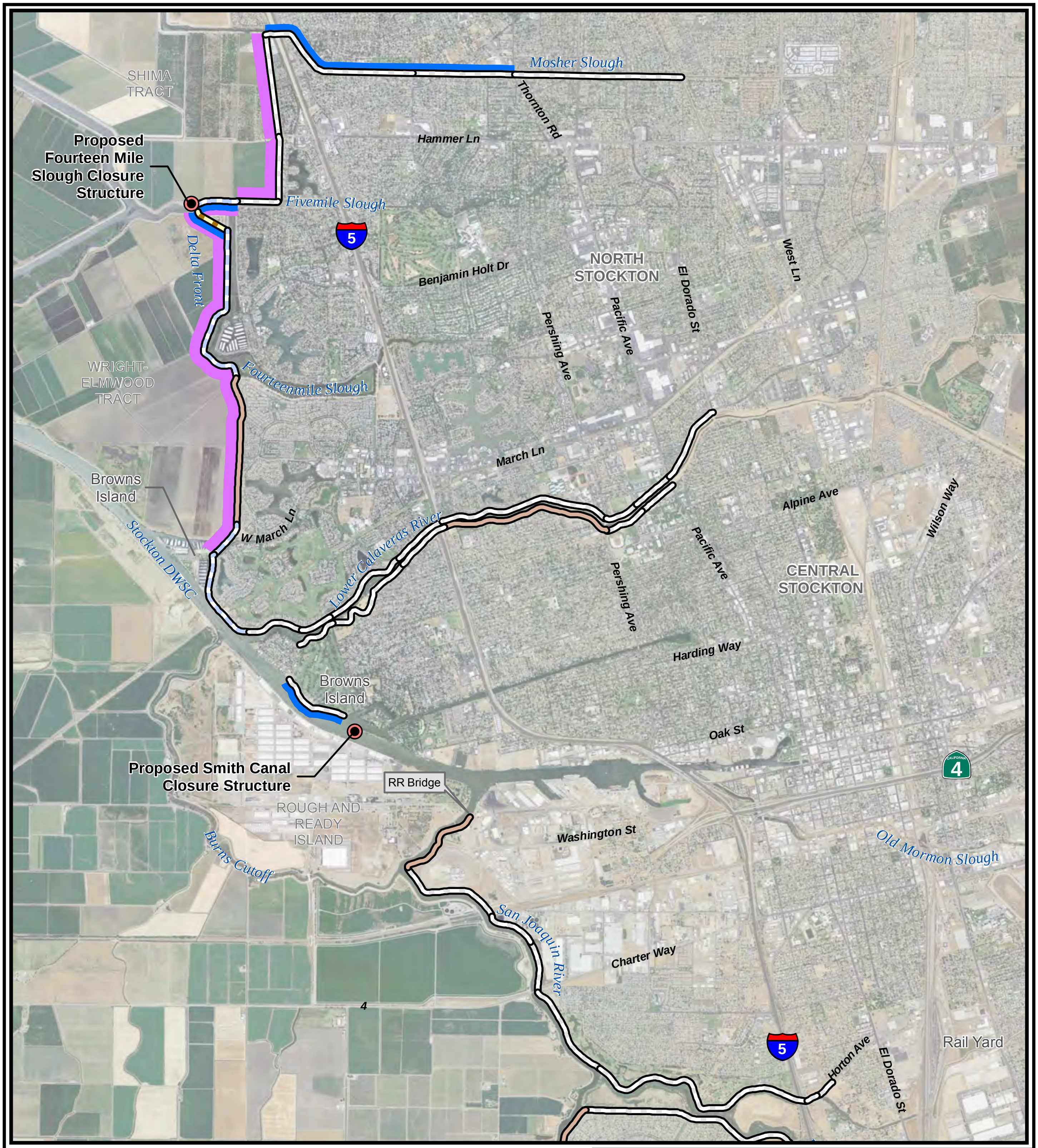
LSJ LEVEE REACHES
ALTERNATIVE 7b
NORTH STOCKTON - EAST

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 3



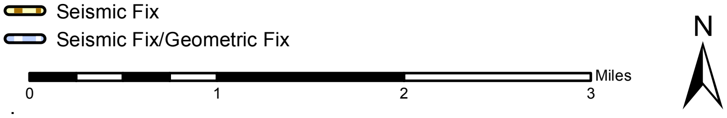
● Control Structure

LSJ Segements

- No Geotech Fix
- Seepage Berm, Geometric/Ht Fix
- Centerline Cut-Off Wall
- Centerline Cut-Off Wall, Geometric/Ht. Fix
- New Levee
- Seismic Fix
- Seismic Fix/Geometric Fix

- Sea Level Rise Protection
- 🟪 Erosion Protection
- 🟧 200 Year + 3' Raise

Aerial Imagery: 2012, NAIP, 1m



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

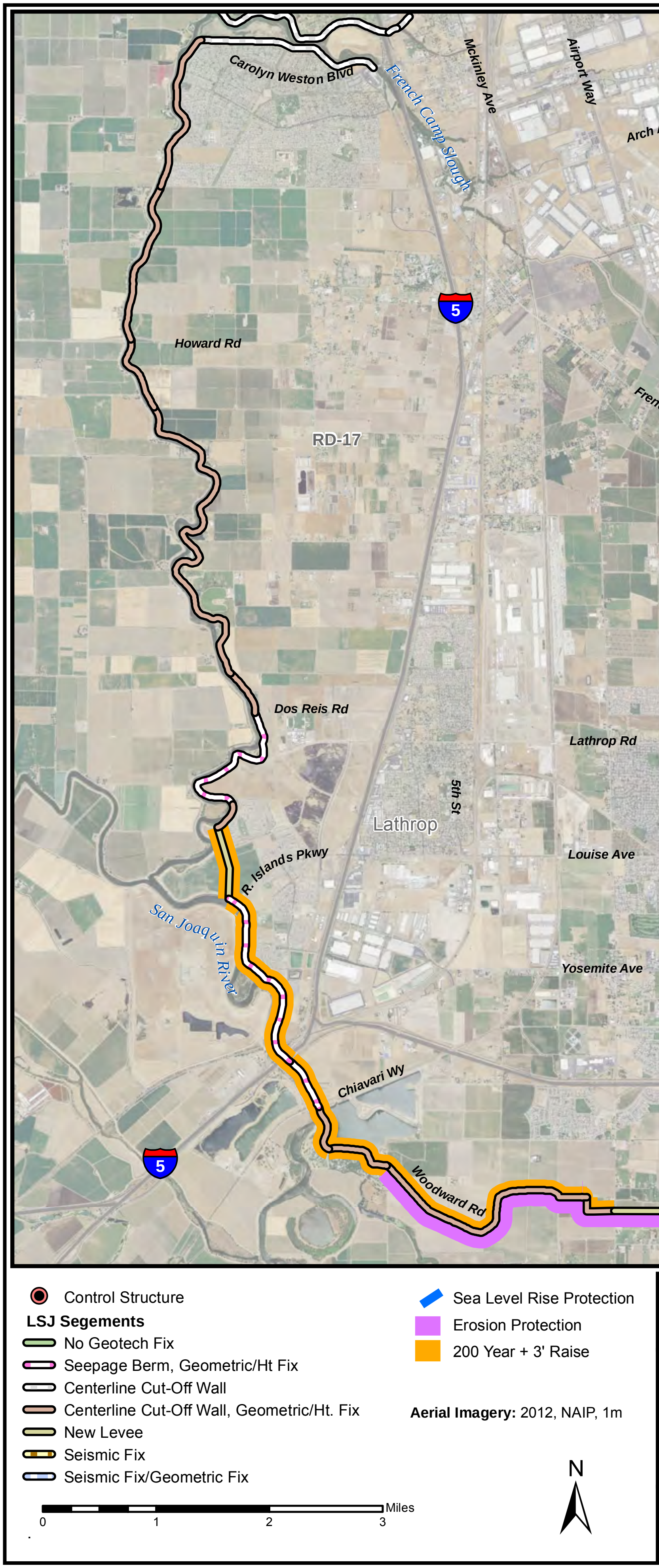
LSJ LEVEE REACHES ALTERNATIVE 7b NORTH STOCKTON - WEST

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 4



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

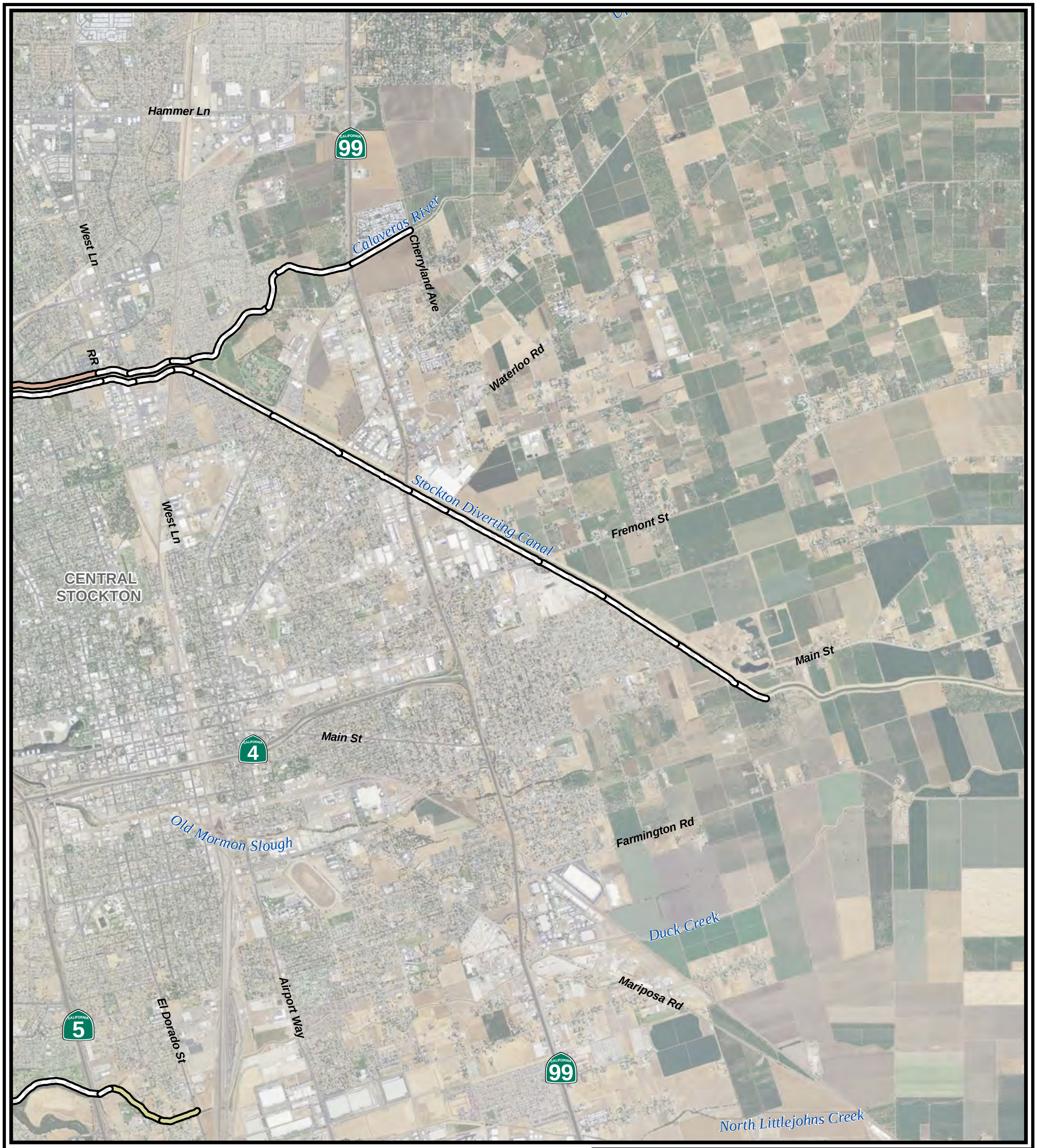
LSJ LEVEE REACHES
ALTERNATIVE 7b
RD-17

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 5



LSJ Segements

- ▬ No Geotech Fix
- ▬ Seepage Berm, Geometric/Ht Fix
- ▬ Centerline Cut-Off Wall
- ▬ Centerline Cut-Off Wall, Geometric/Ht. Fix
- ▬ New Levee
- ▬ Seismic Fix
- ▬ Seismic Fix/Geometric Fix

- ● Control Structure
- ▬ Sea Level Rise Protection
- 🟧 200 Year + 3' Raise
- 🟪 Erosion Protection

Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

N

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

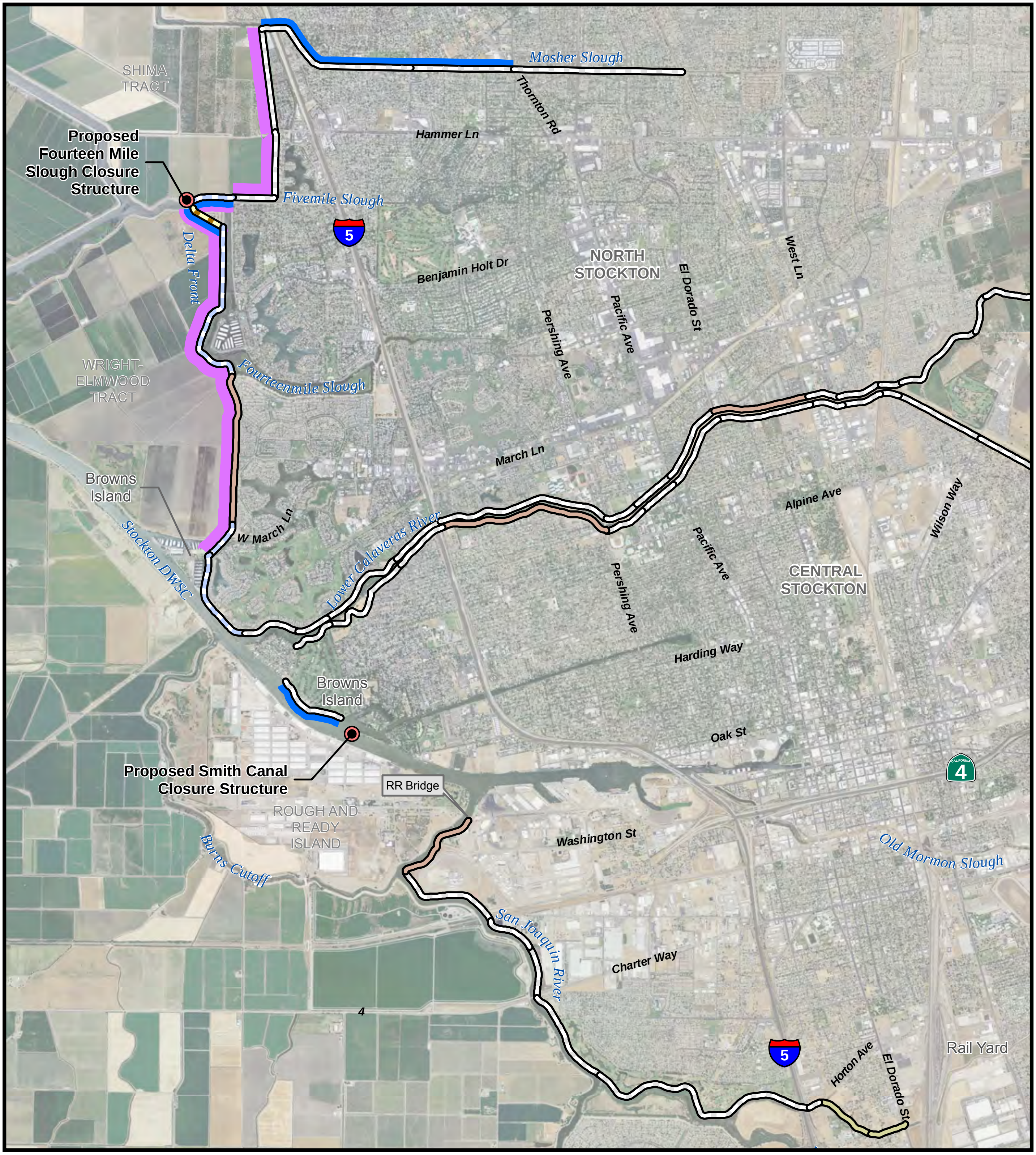
LSJ LEVEE REACHES
ALTERNATIVE 8a
NORTH STOCKTON - EAST

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 6



LSJ Segements

- 🟩 No Geotech Fix
- 🟪 Seepage Berm, Geometric/Ht Fix
- ⬜ Centerline Cut-Off Wall
- 🟫 Centerline Cut-Off Wall, Geometric/Ht. Fix
- 🟨 New Levee
- 🟧 Seismic Fix
- 🟦 Seismic Fix/Geometric Fix

- ● Control Structure
- 🟦 Sea Level Rise Protection
- 🟧 200 Year + 3' Raise
- 🟪 Erosion Protection

Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

N

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

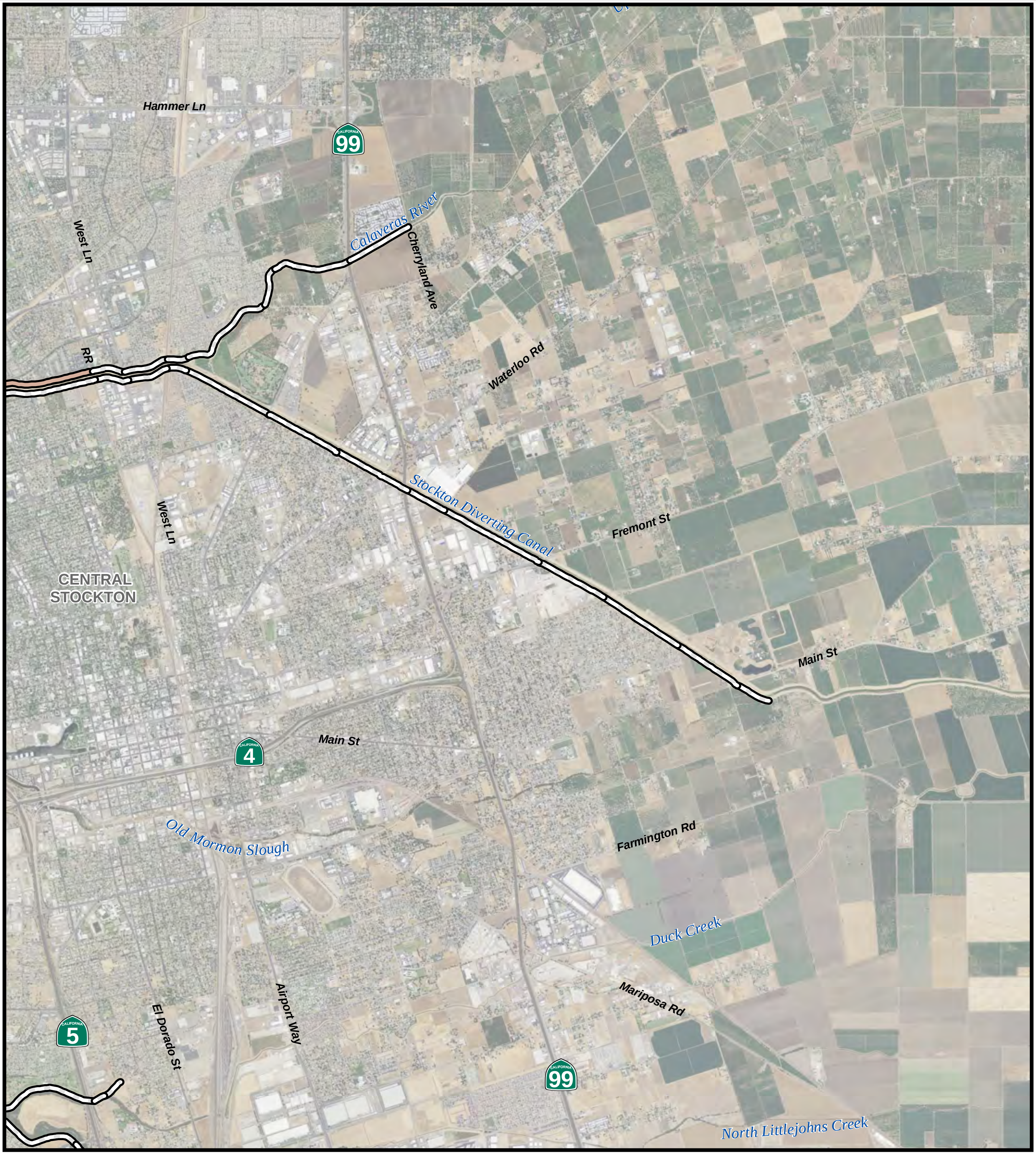
LSJ LEVEE REACHES
ALTERNATIVE 8a
NORTH STOCKTON - WEST

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 7

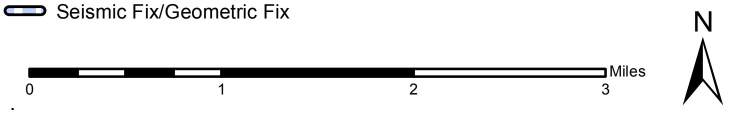


LSJ Segements

- No Geotech Fix
- Seepage Berm, Geometric/Ht Fix
- Centerline Cut-Off Wall
- Centerline Cut-Off Wall, Geometric/Ht. Fix
- New Levee
- Seismic Fix
- Seismic Fix/Geometric Fix

- ● Control Structure
- Sea Level Rise Protection
- 🟧 200 Year + 3' Raise
- 🟪 Erosion Protection

Aerial Imagery: 2012, NAIP, 1m



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

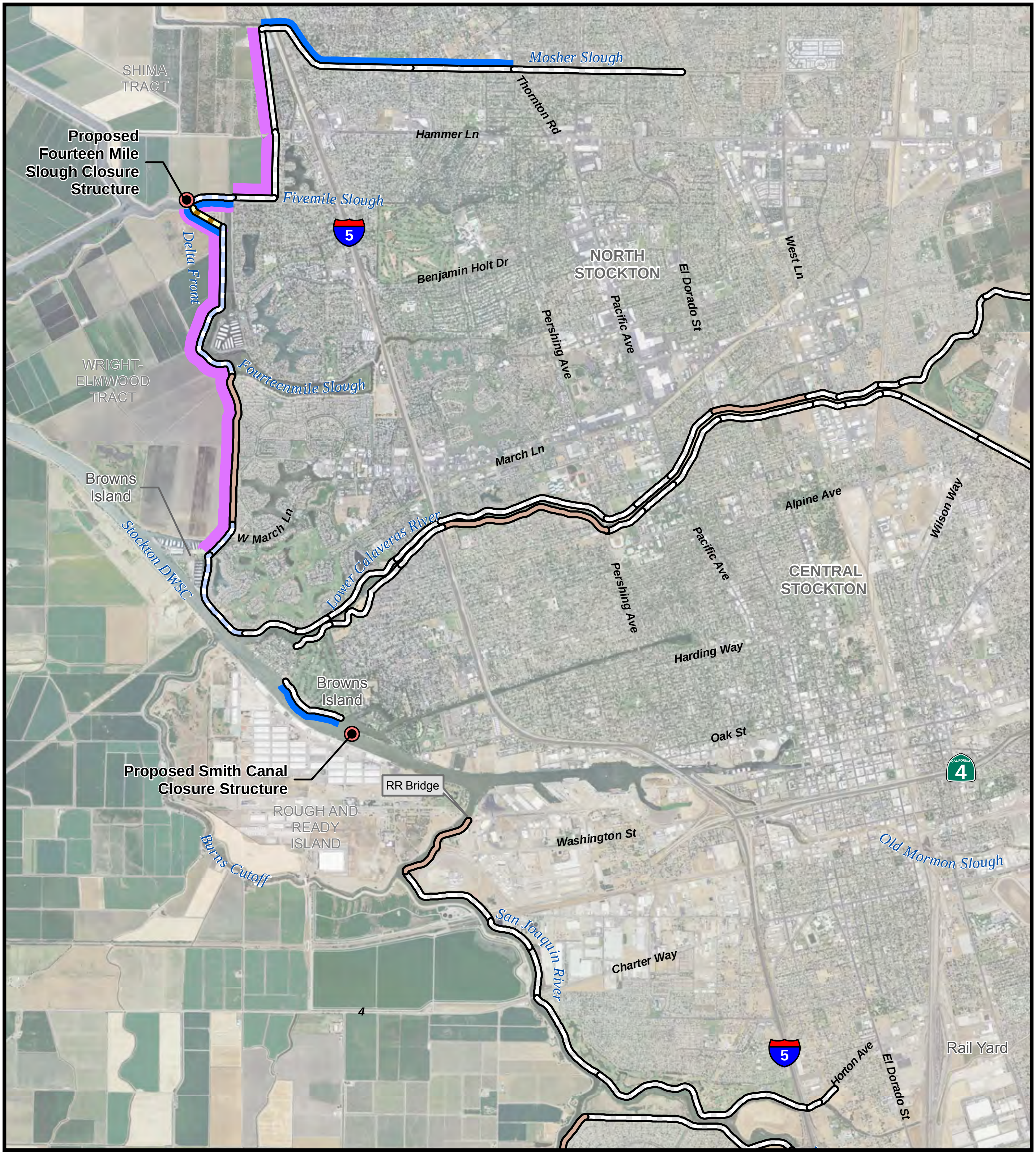
LSJ LEVEE REACHES
ALTERNATIVE 8b
NORTH STOCKTON - EAST

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 8



LSJ Segements

- No Geotech Fix
- Seepage Berm, Geometric/Ht Fix
- Centerline Cut-Off Wall
- Centerline Cut-Off Wall, Geometric/Ht. Fix
- New Levee
- Seismic Fix
- Seismic Fix/Geometric Fix

- ● Control Structure
- Sea Level Rise Protection
- 🟧 200 Year + 3' Raise
- 🟪 Erosion Protection

Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

N

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

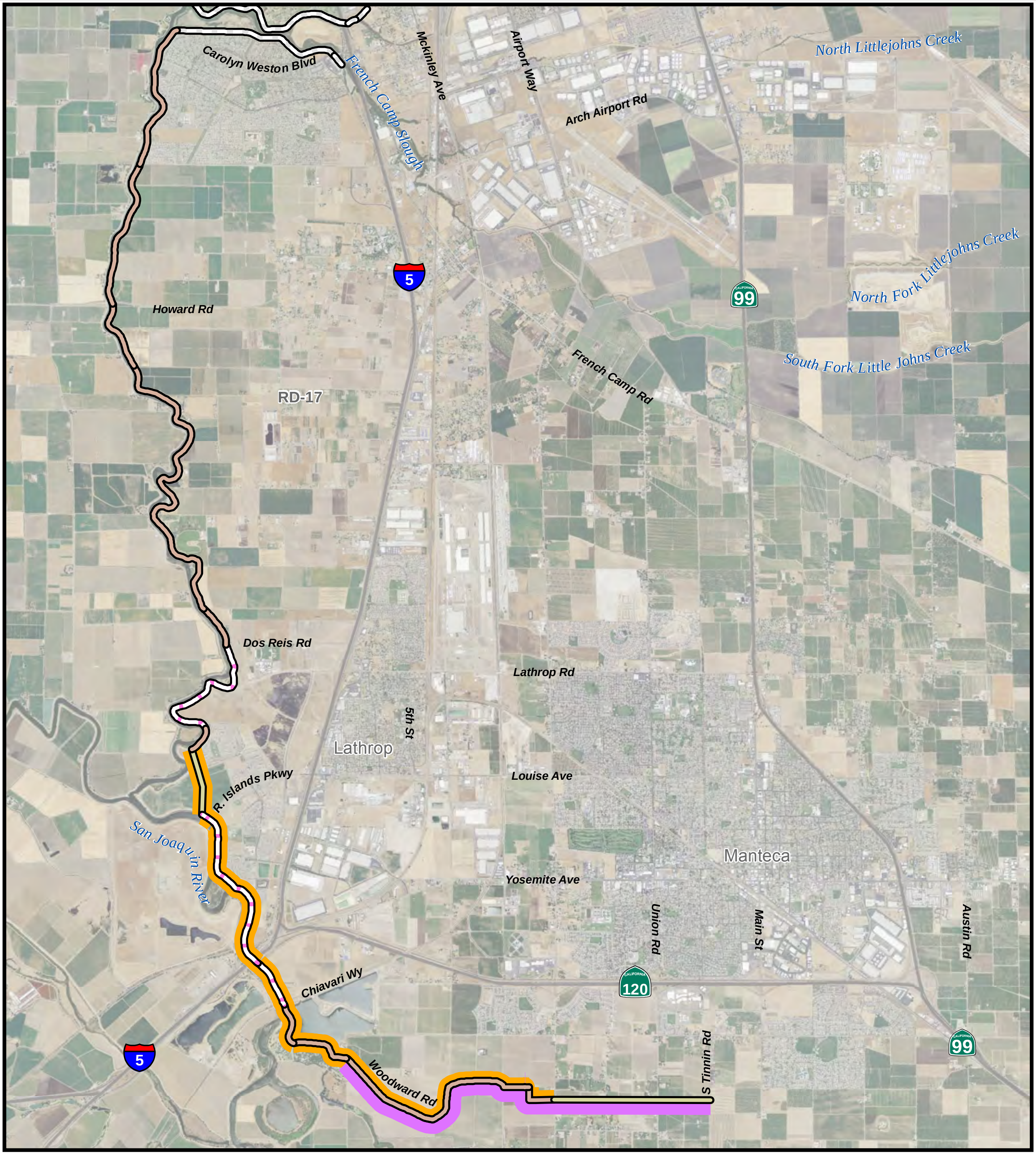
LSJ LEVEE REACHES
ALTERNATIVE 8b
NORTH STOCKTON - WEST

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 9



LSJ Segements

- No Geotech Fix
- Seepage Berm, Geometric/Ht Fix
- Centerline Cut-Off Wall
- Centerline Cut-Off Wall, Geometric/Ht. Fix
- New Levee
- Seismic Fix
- Seismic Fix/Geometric Fix

- ● Control Structure
- Sea Level Rise Protection
- 🟧 200 Year + 3' Raise
- 🟪 Erosion Protection

Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

N

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

LSJ LEVEE REACHES
ALTERNATIVE 8b
RD-17

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 10

LSJ Segements

- ▬ No Geotech Fix
- ▬ Seepage Berm, Geometric/Ht Fix
- ▬ Centerline Cut-Off Wall
- ▬ Centerline Cut-Off Wall, Geometric/Ht. Fix
- ▬ New Levee
- ▬ Seismic Fix
- ▬ Seismic Fix/Geometric Fix
- ▬ Mormon Channel Improvements

- ● Control Structure
- ▬ Sea Level Rise Protection
- 🟧 200 Year + 3' Raise
- 🟪 Erosion Protection

Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

N

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

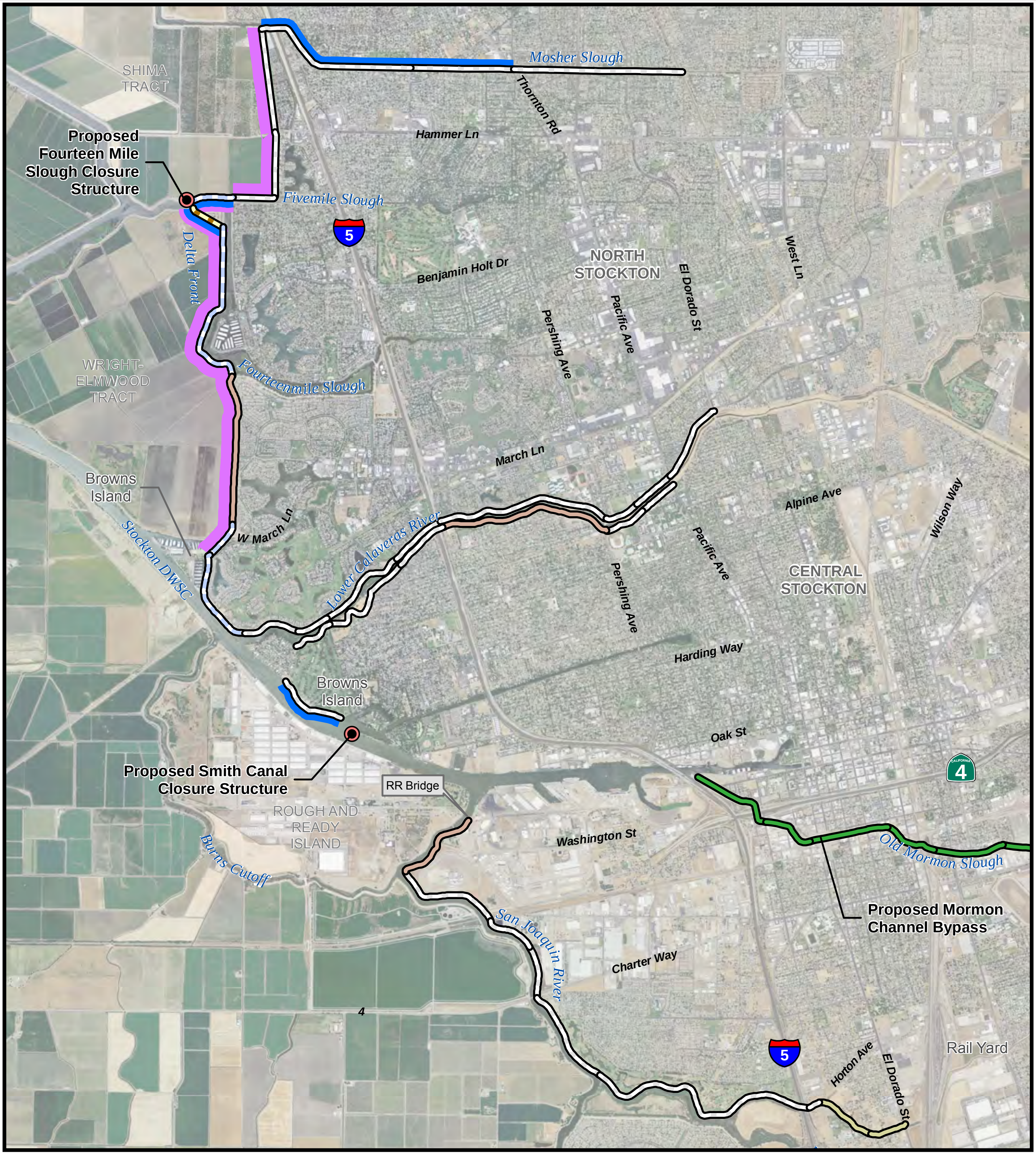
LSJ LEVEE REACHES
ALTERNATIVE 9a
NORTH STOCKTON - EAST

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 11



LSJ Segements

- No Geotech Fix
- Seepage Berm, Geometric/Ht Fix
- Centerline Cut-Off Wall
- Centerline Cut-Off Wall, Geometric/Ht. Fix
- New Levee
- Seismic Fix
- Seismic Fix/Geometric Fix
- Mormon Channel Improvement

- ● Control Structure
- Sea Level Rise Protection
- 🟧 200 Year + 3' Raise
- 🟪 Erosion Protection

Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

N

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

LSJ LEVEE REACHES
ALTERNATIVE 9a
NORTH STOCKTON - WEST

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 12

LSJ Segements
- No Geotech Fix
- Seepage Berm, Geometric/Ht Fix
- Centerline Cut-Off Wall
- Centerline Cut-Off Wall, Geometric/Ht. Fix
- New Levee
- Seismic Fix
- Seismic Fix/Geometric Fix
- Mormon Channel Improvements

- ● Control Structure
- Sea Level Rise Protection
- 🟧 200 Year + 3' Raise
- 🟪 Erosion Protection

Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

N

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

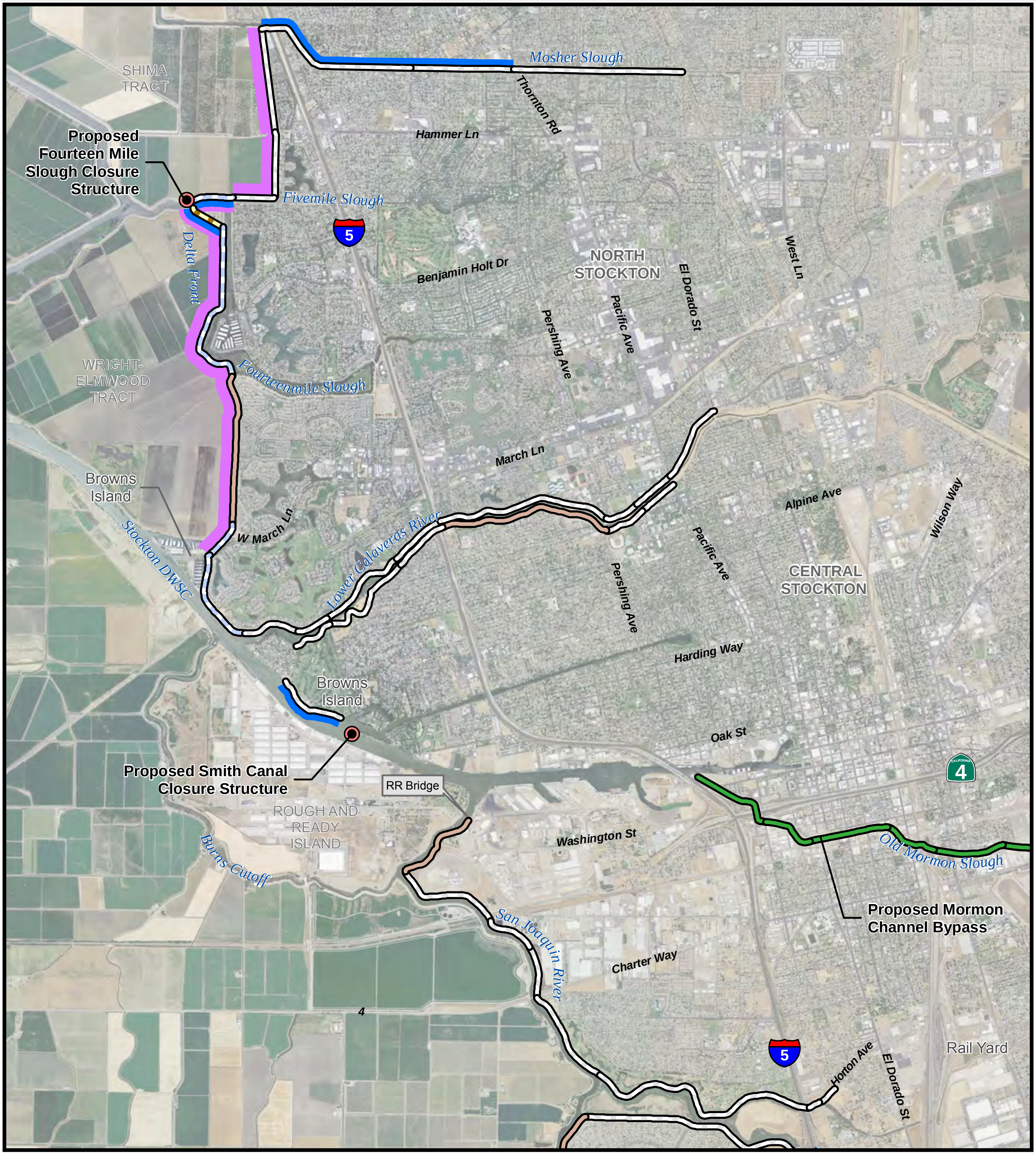
LSJ LEVEE REACHES
ALTERNATIVE 9b
NORTH STOCKTON - EAST

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 13



LSJ Segements

- No Geotech Fix
- Seepage Berm, Geometric/Ht Fix
- Centerline Cut-Off Wall
- Centerline Cut-Off Wall, Geometric/Ht. Fix
- New Levee
- Seismic Fix
- Seismic Fix/Geometric Fix
- Mormon Channel Improvement

- ● Control Structure
- Sea Level Rise Protection
- 🟧 200 Year + 3' Raise
- 🟪 Erosion Protection

Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

N

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

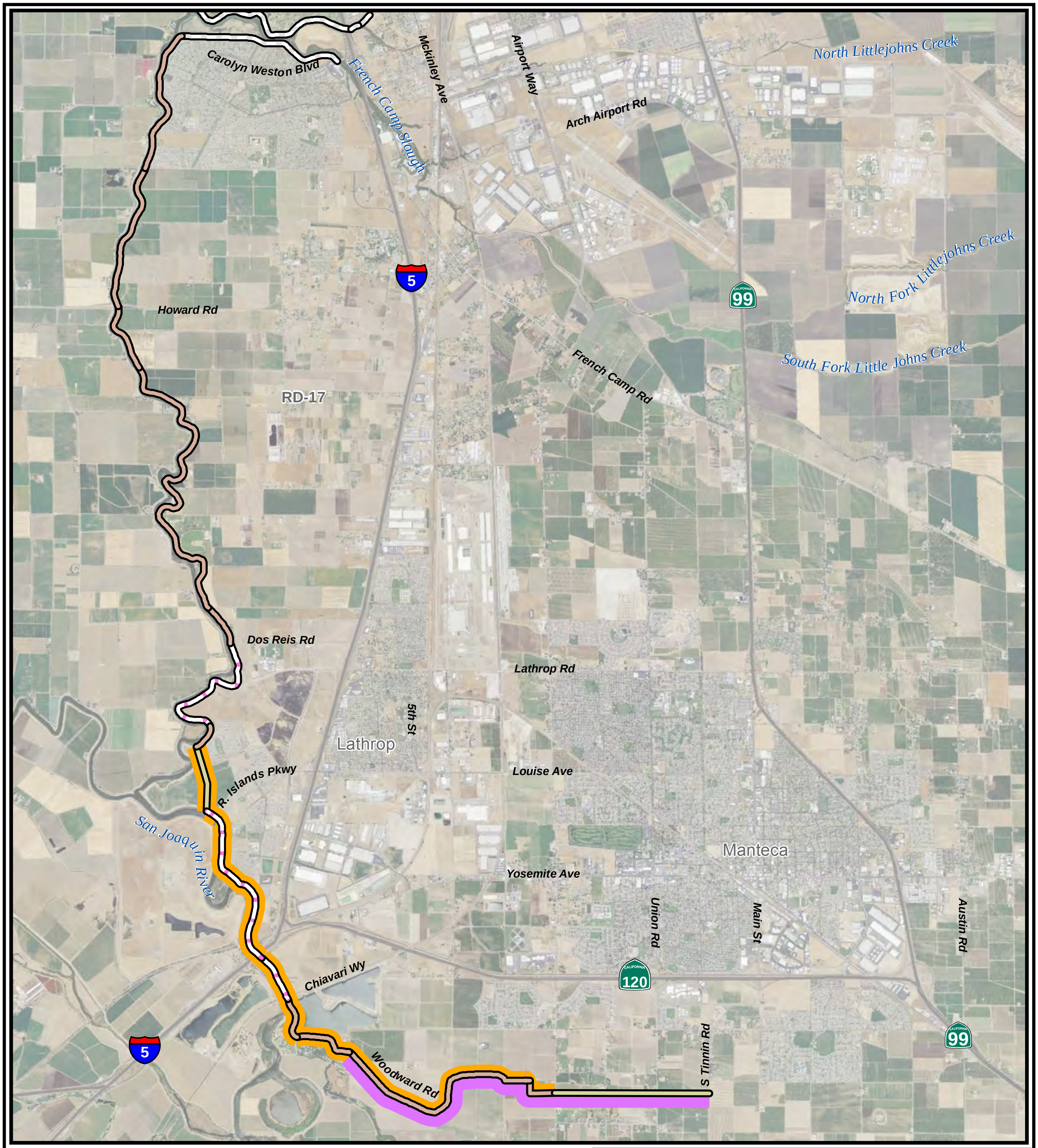
LSJ LEVEE REACHES
ALTERNATIVE 9b
NORTH STOCKTON - WEST

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 14



LSJ Segements

- ▬ Seepage Berm, Geometric/Ht Fix
- ▬ Centerline Cut-Off Wall
- ▬ Centerline Cut-Off Wall, Geometric/Ht. Fix
- ▬ New Levee
- ▬ Seismic Fix
- ▬ Seismic Fix/Geometric Fix
- ▬ Mormon Channel Improvements

- ● Control Structure
- ▬ Sea Level Rise Protection
- 🟧 200 Year + 3' Raise
- 🟪 Erosion Protection

Aerial Imagery: 2012, NAIP, 1m

0 1 2 3 Miles

N

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

LSJ LEVEE REACHES
ALTERNATIVE 9b
RD-17

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUGUST 2014

PLATE 15

US Army Corps of Engineers®
Sacramento District
Engineering Division

Lower San Joaquin Feasibility Study – Environmental Impact Report/ Supplemental Environmental Impacts Statement

San Joaquin County, California

Appendix B: Engineering Appendix

February 2015

{This page left intentionally blank}

CONTENTS

CONTENTS (Cont'd)

CHAPTER 3 – TSP ALTERNATIVE LS-7a

LIST OF TABLES

LIST OF FIGURES

ACRONYMS

ACE	Annual Chance Excedance
ACRA	Abbreviated Cost Risk Analysis
ASTM	American Society for Testing and Materials
CFS	cubic feet per second (flow)
CM	Construction Management
CS	Central Stockton (geographical area)
CVHS	Central Valley Hydrology Study
CWWBS	Civil Works Work Breakdown Structure
DRMS	Delta Risk Management Study
DSM	Deep Soil Mixing
DWR	California Division of Water Resources
EC	Engineer Circular
ED	Engineering Division (USACE)
EO	Executive Order
ESA	Environmental Site Assessment ID
ETL	Engineer Technical Letter (USACE)
ER	Engineer Regulation
FEMA	Federal Emergency Management Agency
FLO-2D	Flood routing model simulating channel flow, unconfined overland, and street flow over complex topography
GIS	Geographic Information Systems
HEC-HMS	Hydraulic Engineering Center – Hydraulic Modeling System
HEC-RAS	Hydraulic Engineering Center – River Analysis System
HTRW	Hazardous, Toxic, Radioactive Waste
HQ	Head Quarters (USACE)
IDC	Interest During Construction
LiDAR	Light Detection and Ranging
LMA	Local Management Agency
LS, LSJ	Lower San Joaquin
LSJFS	Lower San Joaquin Feasibility Study
MCACES	Micro-Computer Aided Cost Estimating System
NAD	North American Datum
NAVD	North American Vertical Datum
NS	North Stockton (geographical area)
NSSDA	National Standards for Spatial Data Accuracy
OMRR&R	Operation and Maintenance, Repair, Replacement and Rehabilitation
PACR	Post Authorization Change Report
PCET	Parametric Cost Estimating Tool
PDT	Project Development Team
PED	Preconstruction, Engineering, and Design
RD	Reclamation District
RE	Real Estate
RMSE(r)	Root Mean Square Error in r

ACRONYMS (Cont'd)

ROW	Right of Way
SEWD	Stockton East Water District
SJAFCA	San Joaquin Area Flood Control Agency
SJR	San Joaquin River
SLR	Sea Level Rise
SoP	Standard of Practice
SPK	Sacramento District (USACE)
TPCS	Total Project Cost Summary
TSP	Tentatively Selected Plan
ULDC	Urban Levee Design Criteria
UNET	one dimensional unsteady flow model for open channel flow
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
VE	Value Engineering (study)
WTP	Water Treatment Plant

{This page left intentionally blank}

CHAPTER 1 – INTRODUCTION

1.1 Project Description and Background

Since initiating the Lower San Joaquin Feasibility Study (LSJFS), the Project Delivery Team (PDT) representatives have developed a comprehensive flood control plan for San Joaquin County. The PDT initially developed a framework based on known constraints from the varying organizations. The Federal constraints centered on adhering to Corps of Engineers (Corps) study policies for a project to be authorized for federal construction funding. The goal of the California Division of Water Resources (DWR) was the completion of the study by January 2015 to meet the goal of registering the project for state bond appropriations during the same month. San Joaquin Area Flood Control Agency's (SJAFCA) goal was the continuing effort to provide safety to their community during rising floods.

While the LSJFS began as a traditional feasibility study, it was later reprogrammed under the new Corps planning modernization 3x3x3 (3^3) and as such and was placed on a shorter schedule with matching appropriations. The transition to 3^3 occurred during the winter/spring of 2012. The original study began approximately a year earlier in the fall of 2010.

1.2 Purpose and Scope

This summary provides a synopsis of the engineering analysis conducted during the feasibility work phase of the study by the engineering portion of the PDT. The objective is to summarize the designs and cost estimates completed through the final array of alternatives and TSP.

1.3 Sponsors

The LSJFS was initiated as a cost share agreement between SJAFCA and the Corps in February of 2009. The Central Valley Flood Protection Board represented by the California DWR signed on as a secondary non-federal sponsor in July of 2010. The local sponsor's design team was represented by Peterson Brustad, Inc.

CHAPTER 2 – GENERAL DESIGN CONSIDERATIONS

2.1 General

The goal of the engineering appendix is to provide a summary of the methods developed to reduce flood damages. The recommended flood risk reduction area is provided in Figure 1.

2.2 Datum

The North American Datum of 1983 (NAD 83) State Plane California Coordinate System Zone III (U.S. Survey Feet) was used for horizontal control. The North American Vertical Datum of 1988 (NAVD 88) was used as the vertical datum.

2.3 Alignment and Segments

2.3.1 Incremental Study Segments thru Final Array

Following the preliminary screening effort, levees which qualified for the initial screening were estimated for fix-in-place construction and associated costs as small segments. Fix-in-place costs were estimated for small segments to provide flexibility during the refinement stages of alternatives analysis. These smaller segments allow the refinements to add or delete segments incrementally. The study area contains 92 miles of levee which were classified into approximately 130 segments that were on average 3,700 feet in length. The result of this segmentation is presented in Figures 2 through 5.

Segment stationing went unchanged during the various phases of the study. The packaging of the number of segments varied as segments were added or deleted depending on the formulation of the array.

2.4 Alternative Reaches

2.4.1 Geographical Study Segments

Study segments were developed geographically based on the adjacent water feature or tract name. Segments were created for Mosher Creek, Fourteen Mile Slough, the Calaveras River, the delta front levees between Mosher Creek and the Calaveras River, Mormon Channel, Stockton Diverting Canal, Smith Canal, San Joaquin River, French Camp Slough, Duck Creek, and Paradise Cut Bypass. A geographical feature would often times dictate where a segment would begin or end. These geographical features which were used as natural starting and stopping locations were highways, levee embankments, water features, embankments, etc. See 2.3.1 General for additional information, and map Figures 2 through 5 for individual segments.

2.4.2 Initial Alternatives

A list of measures were created by the PDT during the planning charette of January 2013 to use in the formulation of alternative plans. A measure is a feature or an activity that can be implemented at a specific geographic site to address one or more planning objectives. For example, a measure could be a fix for an earthen levee such as a cut-off wall or seepage berm. The measures were categorized into structural and non-structural solutions. Seventy-three measures were identified as potential options for the study. The six criteria which were used to rank the 73 measures were meets objective, cost, environmental impacts, acceptability (by the community), completeness, and 21st century flood plain management.

The decision to consider a measure was based on its ranking among the 6 criteria used to rank the measures including a geotechnical engineering recommendation, and a decision to implement based on need. The 73 measures were reduced to 22 measures after ranking the measures based on the criteria. Measures were identified for 3 distinct geographical areas. The areas were divided into North Stockton, Central Stockton, and Reclamation District (RD) 17 (South Stockton). Six alternatives were recommended for North Stockton, five alternatives were recommended for Central Stockton, and five alternatives were recommended for RD17. The alternatives were created through a combination of flood containment using hydraulic breach scenarios plus a common sense approach to reach lengths which might terminate at highways or high ground. The Mormon Channel bypass and Paradise Cut options were recommended as incremental alternatives for further evaluation during the Value Engineering Study. Tables 1, 2, and 3 provide further details of the initial arrays. Figure 6 through 8 are provided as representative alternatives for North Stockton, Central Stockton, and RD-17 areas respectively.

2.4.3 Focused Array

Hydraulic design provided model runs of breach simulations which were performed for the initial alternatives. Some of the alternatives were modified based on their performance after a levee breach. An example of flooding containment is shown for the North Stockton area in Figure 9. The results from Figure 9 were used as a tool by our hydraulic designer to further refine alternatives.

The following summarizes a focused array used to begin identifying the TSP.

Alternative 1: The No-Action Alternative. Under this plan no effort is made to further reduce the risk of flooding. The areas identified in the initial alternatives are a combination of project and non-project levees which either have geometric deficiencies, height deficiencies, through and under seepage issues, landside stability, or erosion issues.

Alternative 2A – Fix-in-Place, No Bypass: Alternative 2A is a combination of North Stockton Alternative F, Central Stockton Alternative D, and RD17 Alternative E.

Table A. Description of Implementing Alternative 2A (Figure 11)

Initial Alternative Features	Specific Structural Features: Smith Canal, Mosher Slough and 14 Mile Slough Closure Structures.
NS-F, CS-D, RD-17-E	
Description: Delta Front North and South, and Calaveras River addresses the right bank of the Calaveras River and the delta front as flooding sources. This alternative includes closure structures across Mosher Slough and Fourteen mile Slough. Additionally the Calaveras River, Diverting Canal, and San Joaquin River (SJR) address the San Joaquin River, Stockton Diverting Canal, Calaveras River, French Camp Slough and Duck Creek as flooding sources and includes the Smith Canal closure structure. Finally the north portion of the SJR of RD-17 with a tieback levee and levee extension address the San Joaquin River and French Camp Slough as flooding sources.	

Alternative 2A is shown in Figure 11 for reference. Further evolution of Alternative 2A provided for levee improvements connecting the existing Delta Front levees to the railroad tracks along the north bank of Mosher Slough. Figure 11 does not show the Mosher slough levee as part of the alternative which was included later.

Alternative 2B – Fix-in-Place, No Bypass: Alternative 2B is a combination of North Stockton Alternative B, Central Stockton Alternatives B and C, and RD-17 Alternative E.

Table B. Description of Implementing Alternative 2B (Figure 12)

Initial Alternative Features	Specific Structural Features: Smith Canal, Mosher Slough and 14 Mile Slough Closure Structures.
NS-B, CS-B, CS-C, RD-17-E	
Description: Delta Front north and south, Calaveras River and SJR address the delta and tidal portion of the Calaveras River as flooding sources. The alternative includes closure structures across Mosher Slough, Smith Canal, and 14 Mile Slough. For the San Joaquin River Front in Central Stockton the SJR, French Camp Slough, and Duck Creek are addressed as sources of flooding. The SJR North with Tieback and Extension in RD-17 address the SJR and French Camp Slough as flooding sources. This alternative also extends the tie-back levee to address flanking issues.	

Alternative 2B is shown in Figure 12 for reference. Further evolution of Alternative 2B provided for levee improvements connecting the existing Delta Front levees to the railroad tracks along the north bank of Mosher Slough. Figure 12 does not show the Mosher slough levee as part of the alternative which was included later.

Alternative 3 – Fix-in-Place with Bypass: Alternative 3 is Alternative 2A (North Stockton Alternative B, Central Stockton Alternatives B and C, and RD-17 Alternative E) with the addition of the Mormon Channel Bypass.

Table C. Description of Implementing Alternative 3 (Figure 13)

Initial Alternative Features	Specific Structural Features: Smith Canal, Mosher Slough and 14 Mile Slough Closure Structures.
NS-B, CS-B, CS-C, RD-17-E, Mormon Slough Bypass	
Description: The delta and tidal portion of the Calaveras River, and SJR are addressed as the flooding sources. The alternative includes a closure structure across Mosher Slough and Smith Canal. Additionally the San Joaquin River, French Camp Slough, and Duck Creek are addressed as sources of flooding. For RD-17 the north portion of the SJR with levee tieback and levee extension is included. This alternative addresses the San Joaquin River and French Camp Slough as flooding sources. The alternative includes the Mormon Slough bypass which diverts floods off the Stockton Diverting Canal and the Calaveras River.	

Alternative 3 is shown in Figure 13 for reference. Further evolution of Alternative 3 provided for levee improvements connecting the existing Delta Front levees to the railroad tracks along the north bank of Mosher Slough. Figure 13 does not show the Mosher slough levee as part of the alternative which was included later. Alternative 3 evolved into Alternatives 7, 8, 9, and 10.

Further evolution of alternatives included levee raises which became Alternative 4. For more detailed information on the focused array, reference the draft integrated feasibility report (draft report).

2.4.4 Final Array

The final array contained combinations of the best hydraulically performing and economically justified alternatives from the focused array. A majority of the alternatives reduced residual damages to a point where additional measures couldn't be justified. The economic analysis conducted during evaluation of the focused array of alternatives evaluated if increases in levee height would be economically justified. It was determined that increases in levee height to meet the DWR Urban Levee Design criteria for 2070 sea level conditions had higher net benefits. Therefore, all alternatives presented in the final array include levee raises that met ULDC requirements in 2070 as a design assumption.

Final array alternatives are provided in Table D. A new naming convention was used for the final array alternatives. As seen below, focused array alternative 2B plus levee raises for sea level rise is labeled LS-7b, focused array 2A plus sea level raise is labeled LS-8b. Refer to Table D for further nomenclature.

Table D. Final Array of Alternatives Information for the LSJ Study

Focused Name	Final Name	Information	Geographical Areas
2B + SLR[4] (LS-7)	LS-7b	Cut-off Wall (>75% of the fix), ~ 42 repair miles, construction footprint: ~ 364 acres	North, Central, RD-17 (Delta Front, Lower Calaveras, and San Joaquin River Levee Improvements)
2A + SLR (LS-8)	LS-8b	Cut-off Wall (>80% of the fix), ~ 53 repair miles, construction footprint: ~ 418 acres	North, Central, RD-17 (Delta Front, Lower Calaveras, Stockton Diverting Canal, and San Joaquin River Levee Improvements)
3 + SLR (LS-9)	LS-9b	Cut-off Wall (~80% of the fix), ~ 43 repair miles, construction footprint: ~ 401 acres	North, Central, RD-17 (Delta Front, Lower Calaveras, San Joaquin River Levee Improvements and Mormon Channel Bypass)
LS-7 w/o RD-17	LS-7a	Cut-off Wall (>85% of the fix), ~ 23 repair miles, construction footprint: ~152 acres	North and Central Stockton (Delta Front, Lower Calaveras, San Joaquin minus RD-17)
LS-8 w/o RD-17	LS-8a	Cut-off Wall (>90% of the fix), ~ 33 repair miles, construction footprint: ~ 214 acres	North and Central Stockton (Delta Front, Lower Calaveras, Stockton Diverting Canal, San Joaquin minus RD-17)
LS-9 w/o RD-17 (LS-10)	LS-9a	Cut-off Wall (>92% of the fix), ~ 33 repair miles, construction footprint: ~ 219 acres	North and Central Stockton (Delta Front, Lower Calaveras, San Joaquin minus RD-17, Mormon Channel Bypass)

[3] – assuming District Corps policy of 20’ landside easement

[4] – SLR is sea level rise

Just prior to a TSP decision on which alternative to formulate for, USACE is recommending that only North and Central Stockton geographically defined areas be considered for TSP inclusion. The geographical area of RD-17 conflicts with Corp policy EO 11988 which is being coordinated with the sponsor.

2.5 Topographic Data

2.5.1 General

The primary source of topographic or terrain data for the construction of the HEC-RAS models was LiDAR data compiled by DWR under the Central Valley Floodplain Evaluation and Delineation Study (CVFED) and Delta Risk Management Study (DRMS). The minimum expected horizontal accuracy was tested to meet or exceed a 3.5-foot horizontal accuracy at 95 percent confidence level using RMSE(r) x 1.7308 as defined by the National Standards for Spatial Data Accuracy (NSSDA). Final ground surface LiDAR point elevation data in areas other than open terrain meet or exceed NSSDA standards of 0.6 feet RMSE vertical (Accuracy z = 1.2 feet at the 95% confidence level). Accuracy was tested to meet a 0.6-foot fundamental vertical accuracy at 95 percent confidence level using RMSE(z) x 1.9600 as defined by the NSSDA. The horizontal datum is NAD83 (2007) and the vertical datum the North American Vertical Datum of 1988 (NAVD88). CVFED LiDAR data was acquired in a period of several weeks between March 17, 2008 and April 4, 2008.

2.6 Hydrology

2.6.1 General

Hydrology for the San Joaquin River was based on analysis conducted by the California Department of Water Resources (DWR) and USACE for the 2002 Sacramento-San Joaquin Comprehensive Study. Hydrology for the Calaveras River and Mormon Slough was based on analysis conducted for the feasibility study between 2010 and 2014 by the Local Sponsors and USACE and followed procedures compatible with the California Department of Water Resources Central Valley Hydrology Study (CVHS). The following provides a summary of the hydrologic flow frequency analysis utilized as inputs to hydraulic analysis. The hydrology appendix provides additional details.

a. San Joaquin River. The upstream boundary for the San Joaquin River hydraulic model is the USGS stream gage San Joaquin River near Vernalis. The drainage area at the stream gage is 13,536 square miles. Records at the USGS stream gage only account for flow in the channel and do not account for overbank flow. During large floods, flow on the waterside of the right bank levee outflanks the gage before discharging into the main channel at the RD17 tieback levee. Hydrologic frequency analysis presented herein accounts for all flow passing the gage, including channel and right overbank flow.

The Sacramento-San Joaquin Comprehensive study included the entire Sacramento and San Joaquin Valleys. Thirty-day regulated flow hydrographs developed for 50% (1/2) Annual Chance Exceedance (ACE), 10% (1/10) ACE, 4% (1/25) ACE, 2% (1/50) ACE, 1% (1/100) ACE, 0.5% (1/200) ACE, and 0.2% (1/500) was used in the hydraulic analysis.

The flood frequency analysis involved evaluations of long term historical records at the stream gages. The adopted statistics and period of record for the unregulated conditions near Vernalis are provided in Table E. A tabulation of the flood frequency estimates for flood durations between 1-day and 30-days is provided in Table F.

Table E. Rain Flood Frequency Statistics, San Joaquin River near Vernalis

Unregulated Conditions

Flood Duration	Adopted Log Mean	Adopted Log Standard Deviation	Adopted Log Skew	Record (Years)	
				Years Evaluated	Years Used
1-Day	4.375	0.450	-0.1	1917 - 1998	82
3-Day	4.333	0.445	-0.1	1917 - 1998	82
7-Day	4.251	0.433	-0.2	1917 - 1998	82
15-Day	4.148	0.412	-0.2	1917 - 1998	82
30-Day	4.042	0.392	-0.2	1917 - 1998	82

Table F

Flood Frequency Flow Estimates, San Joaquin River near Vernalis

Unregulated Conditions

Flood Duration	Duration Average Discharge by ACE (CFS)						
	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
1-Day	24100	88400	140300	188300	244700	310400	412900
3-Day	21900	79100	124900	167000	216500	273900	363100
7-Day	18400	62500	95200	124000	156500	193000	247300
15-Day	14500	46400	69200	89000	111100	135600	171700
30-Day	11400	34300	50200	63800	78700	95200	119200

A regulated set of hydrographs was obtained from “hand off” points in the lower basin reservoir simulation model. These hydrographs were then used as input to a UNET unsteady flow hydraulic model of the San Joaquin River. A review of the mainstem storm centerings found that peak stages along the San Jaoquin River within the study area are generated by the San Joaquin River at Vernalis storm centering. The model was run for three different upstream levee failure scenarios.

(1) Infinite levee with no overtopping (Infinite). This is considered the extreme high estimate because no floodplain storage is allowed. All flow is confined to the leveed channel. This describes the extreme upper limit of potential peak flow at Vernalis relative to the levee assumption.

(2) Overtopping without Failure (No Fail). This model assumed all levees would overtop but would not fail. This may not be the most likely condition because some levees would likely fail prior to overtopping (probability of poor performance indicated by the fragility curve).

(3) With levee failure condition (With Fail). This model assumed all levees would fail at the 50% fragility point. This may not be the most likely condition because not all levees would fail at the 50% fragility point during the same flood.

A comparison of peak flows for the different levee assumptions is described in Table G. The comp study models were only run for floods larger than 10% ACE.

Table G
Sensitivity of Upstream Levee Failures, San Joaquin River near Vernalis

Regulated Conditions

Levee Scenario	Peak Discharge by ACE (CFS)						
	50% ACE	**10% ACE**	**4% ACE**	**2% ACE**	**1% ACE**	**0.5% ACE**	**0.2% ACE**
Infinite Levee	NA	36900	47000	58400	90800	145500	233700
No Failure	NA	35100	42300	47700	78200	144500	224100
With Failure	NA	32900	43000	50300	77300	113300	166600
Source: 2002 Sacramento-San Joaquin Comprehensive Study UNET model results.							

The peak flow of infinite height assumption was found to always be greater for a given ACE. The greatest difference between infinite height and no fail scenarios occurred at the 2% (1/50) ACE to 1% (1/100) ACE event which is probably around the flood magnitude that most system levees are overtopped. The No-Fail and With-Fail conditions are similar for floods smaller than 1% (1/100) ACE. The No-fail is larger than the with-fail condition for floods larger than 1% (1/100) ACE. The most likely condition is probably between the no-fail and with-fail conditions.

The overtopping with no failure scenario was adopted as the most likely hydraulic condition for this study to support the risk analysis. This probability of overtopping levee failure is accounted for the FDA model using a fragility curve that assumes 100% failure probability at the levee crest.

This assumption helps make a breach probability more statistically independent rather than dependent on each other and is consistent with historical observations that the probability of a breach does not appear to be highly dependent on other breaches occurring. There is no specific guidance on how to apply overtopping assumptions to system wide risk analysis. However, the approach is consistent with our risk and uncertainty guidance. The overtopping without failure assumption is also consistent with the DWR Urban Levee Design Criteria and FEMA mapping approaches. A table of adopted regulated peak flows for this study is provided in Table H. Due to upstream conditions, hydrographs for channel and right overbanks are required for events greater than a 1% (1/100) ACE event.

Table H
Flood Frequency Flow Estimates, San Joaquin River near Vernalis

Regulated Conditions

	Peak Discharge by ACE (CFS)						
Peak Flow	**50% ACE**	**10% ACE**	**4% ACE**	**2% ACE**	**1% ACE**	**0.5% ACE**	**0.2% ACE**
Chanel	6400	35100	42300	47700	78200	124600	165200
Right Overbank	0	0	0	0	0	20400	60500
Total	6400	35100	42300	47700	78200	144500	224100
Note: Peak channel plus right overbank flow may not equal peak total flow due to hydrograph timing.							

The California Department of Water Resources is currently conducting a study of Central Valley Hydrology. The Central Valley Hydrology Study (CVHS) will provide more recent hydrologic frequency estimates throughout the study area. However, the results were not finalized at the time of this study. The draft flood frequency estimates from the CVHS study were compared to the comp study estimates and found to be similar.

c. Calaveras River and Mormon Slough. The upstream hydraulic model boundary for and Calaveras River and Mormon Slough is the USACE stream gage Mormon Slough at Bellota. The drainage area at the gage is 470 square miles. Hydrologic analysis is described in the hydrology appendix dated April 2014. Flood frequency curves and a suite of 10-day hydrographs were developed for the Mormon Slough at Bellota gage.

The period of record analyzed is 104 years from 1907 to 2010. Unregulated flow frequency statistics for the the Mormon Slough at Bellota Gage are provided in Table I. Unregulated discharges by frequency and duration are provided in Table J. The unregulated flood discharge data is used in the levee performance analysis using risk and uncertainty procedures. The one-day duration was used as the unregulated flow in the performance analysis. Although the frequency analysis utilized 104 years of record, an equivalent period of record of 52-yrs was utilized in performance analysis to account for uncertainty in estimating the ungaged unregulated flow between New Hogan Dam and Bellota. The durations indicate how long an average flood of the given Annual Chance Exceedance is above a given discharge.

Table I
Rain Flood Frequency Statistics, Mormon Slough at Bellota

Unregulated Conditions

Flood Duration	Adopted Log Mean	Adopted Log Standard Deviation	Adopted Log Skew	Record (Years)	
				Years Evaluated	Years Used for Statistics
1-Day	3.775	0.482	-0.810	1907 - 2010	104
3-Day	3.608	0.475	-0.753	1907 - 2010	104
7-Day	3.417	0.464	-0.666	1907 - 2010	104
15-Day	3.240	0.461	-0.671	1907 - 2010	104
30-Day	3.079	0.448	-0.668	1907 - 2010	104

Table J
Flood Frequency, Mormon Slough at Bellota

Unregulated Conditions

Flood Duration	Duration Average Discharge by ACE (CFS)						
	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
1-Day	6900	21700	29700	35300	40500	45400	51300
3-Day	4600	14600	20200	24200	28000	31600	36100
7-Day	2900	9300	13000	15800	18500	21100	24500
15-Day	2000	6100	8600	10300	12100	13800	16000
30-Day	1300	4100	5700	6800	7900	9000	10400

A rainfall runoff model was used to derive concurrent local flow hydrographs as internal boundary conditions in the HEC-RAS hydraulic model reaches downstream of Mormon Slough at Bellota. A table of adopted regulated peak flows for this study is provided in Table K.

Table K
Flood Frequency, Mormon Slough at Bellota

Regulated Conditions

	Duration Average Discharge by ACE (CFS)						
	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
Peak Flow	3520	9530	10640	12500	12500	12500	16000

d. Delta Stage-Frequency. A stage frequency analysis was conducted at four stream gages in the Sacramento-San Joaquin Delta that serve as downstream boundary conditions in the hydraulic models. The stage-frequency analysis was conducted for DWR stream gages; Old River at Clifton Court Ferry (B95340), Middle River at Bowden Highway (B95500), San Joaquin River at Ringe Pump (B95620), and Stockton Ship Channel at Burns Cutoff (B95660) . Stage-frequency estimates were developed for three future sea level conditions including 2010, 2020, and 2070. The frequency analysis is described in detail in the Technical Memorandum, Delta Stage-Frequency Analysis for Alternative Comparisons, 9 May 2014.

The stage frequency analysis was based on stage data from the period from 1953 to 2009. Historical peak stages would have been higher under existing (2010) sea level conditions. Historical stage data were adjusted to 2010 sea level conditions for use in the frequency analysis.

Graphical stage-frequency curves were developed for each gage by plotting the historical stage records using Weibul plotting positions. Extrapolation of the stage frequency curves from 2% ACE to 0.2% ACE events was based on hydraulic model simulations of the San Joaquin River system. These relationships between stage and flow at each gage site are currently the best available analysis of hydraulic conditions in the delta for extreme flood events. The resulting stage frequency curves are provided in Tables L, M, and N.

Future Sea level Rise was computed following the method outlined in EC 1165-2-212 for three scenarios. Curve I is based on the historical rate of sea level rise. Curve II reflects an intermediate estimate of the future rate of sea level rise. Curve III reflects a high estimate of the future rate of sea level rise. The Curve II rates were used to estimate future increases in sea level over the period 2010 through 2070 and are provided in Table O. The rates provided for Curve I and Curve III are provided to describe the sensitivity of 2070 sea level conditions to this assumption. Future sea level rise was assumed to impact all flood frequencies the same amount. The Delta consists of a network of channels and it was assumed the hydraulic characteristics for higher sea level conditions would be very similar to the existing conditions.

Table L
Mean Stage estimates by Annual Chance of Exceedance, 2010 Sea Level Conditions

ACE	Mean Stage (Feet-NAVD88)			
	Old River at Clifton Court Ferry	Middle River at Borden Hwy	Stockton Ship Channel at Burns Cutoff	San Joaquin River at Ringe Pump
0.002 (1/500)	13.08*	11.20*	13.01*	12.91*
0.005 (1/200)	12.12*	9.90*	12.12*	12.02*
0.010 (1/100)	11.44*	9.80*	10.10*	10.00*
0.020 (1/50)	9.95	9.57	9.90	9.80
0.040 (1/25)	9.75	9.50	9.70	9.60
0.100 (1/10)	9.35	9.10	9.30	9.20
0.200 (1/5)	8.70	8.55	8.70	8.60
0.300 (1/3)	7.70	7.80	8.15	8.05
0.500 (1/2)	7.15	7.25	7.70	7.60
0.950 (1/1.05)	6.35	6.45	6.70	6.60

* Stage estimates for events larger than 0.020 (1/50) ACE are based on hydraulic model extrapolation. While suitable for economic analysis, estimates should be refined for design
Future Sea Level based EC 1165-2-212 Curve II
Note: Curve I and II estimates can be computed using values in Table 18.

Table M
Mean Stage estimates by Annual Chance of Exceedance, 2020 Sea Level Conditions

ACE	Mean Stage (Feet-NAVD88)			
	Old River at Clifton Court Ferry	Middle River at Borden Hwy	Stockton Ship Channel at Burns Cutoff	San Joaquin River at Ringe Pump
0.002 (1/500)	13.24*	11.36*	13.17*	13.07*
0.005 (1/200)	12.28*	10.06*	12.28*	12.18*
0.010 (1/100)	11.60*	9.96*	10.26*	10.16*
0.020 (1/50)	10.11	9.73	10.06	9.96
0.040 (1/25)	9.91	9.66	9.86	9.76
0.100 (1/10)	9.51	9.26	9.46	9.36
0.200 (1/5)	8.86	8.71	8.86	8.76
0.300 (1/3)	7.86	7.96	8.31	8.21
0.500 (1/2)	7.31	7.41	7.86	7.76
0.950 (1/1.05)	6.51	6.61	6.86	6.76

* Stage estimates for events larger than 0.02 (1/50) ACE are based on hydraulic model extrapolation. While suitable for economic analysis, estimates should be refined for design
Future Sea Level based EC 1165-2-212 Curve II
Note: Curve I and II estimates can be computed using values in Table 18.

Table N
Mean Stage estimates by Annual Chance of Exceedance, 2070 Sea Level Conditions

ACE	Mean Stage (Feet-NAVD88)			
	Old River at Clifton Court Ferry	Middle River at Borden Hwy	Stockton Ship Channel at Burns Cutoff	San Joaquin River at Ringe Pump
0.002 (1/500)	14.74*	12.86*	14.67*	14.57*
0.005 (1/200)	13.78*	11.56*	13.78*	13.68*
0.010 (1/100)	13.10*	11.46*	11.76*	11.66*
0.020 (1/50)	11.61	11.23	11.56	11.46
0.040 (1/25)	11.41	11.16	11.36	11.26
0.100 (1/10)	11.01	10.76	10.96	10.86
0.200 (1/5)	10.36	10.21	10.36	10.26
0.300 (1/3)	9.36	9.46	9.81	9.71
0.500 (1/2)	8.81	8.91	9.36	9.26
0.950 (1/1.05)	8.01	8.11	8.36	8.26

* Stage estimates for events larger than 0.020 (1/50) ACE are based on hydraulic model extrapolation. While suitable for economic analysis, estimates should be refined for design
Future Sea Level based EC 1165-2-212 Curve II
Note: Curve I and II estimates can be computed using values in Table 18.

Table O
Sea Level Rise from 2010 Conditions

Year	Sea Level Rise from 2010 Conditions (Feet)		
	Curve I (Sensitivity)	Curve II (Adopted)	Curve III (Sensitivity)
2010	0.00	0.00	0.00
2015	0.05	0.07	0.10
2020	0.10	0.16	0.23
2025	0.15	0.26	0.37
2030	0.21	0.37	0.53
2035	0.28	0.49	0.70
2040	0.34	0.62	0.90
2045	0.42	0.77	1.12
2050	0.49	0.92	1.35
2055	0.58	1.09	1.60
2060	0.66	1.27	1.87
2065	0.75	1.46	2.16
2070	0.85	1.66	2.47

Rate of Sea Lever Rise based on EC 1165-2-212

e. Interior Drainage. An interior drainage analysis was performed by Peterson-Brustad Incorporated (PBI) for Bear Creek, Mosher Creek, and French Camp Slough sub-basins impacting the study area. A storm centered over the urban area of Stockton was utilized for the analysis. The interior drainage analysis evaluated rainfall runoff and flood depths for 50% (1/2) ACE through 0.2% (1/500) ACE flood events. Storm events with 72-hour durations were evaluated. The analysis is typically 3-days for storm water detention basins. The analysis utilized an HEC-HMS model to compute sub basin runoff and a FLO-2D two dimensional hydraulic model to route the runoff through the study area. The analysis indicated that interior drainage was not a significant factor in estimating annualized flood damages within the study area. Therefore, interior drainage was not studied in further detail in the alternatives analysis.

2.7 Hydraulics

2.7.1 General

The following provides a summary of the hydraulic design and evaluation of the final array of alternatives.

a. Hydraulic Models: Four separate hydraulic models, adapted from existing hydraulic models, were utilized to evaluate the final alternatives for this study. Water surface profiles for the San Joaquin River were computed using a HEC-RAS unsteady one-dimensional flow model of the San Joaquin River system. Water surface profiles for Calaveras River and Mormon Slough were computed using a HEC-RAS unsteady flow model of the system. Levee breach simulations for the area North of French Camp Slough were conducted using the North FLO-2D model. Levee breach simulations for the area south of French Camp Slough were conducted using the south FLO-2D model.

b. Hydraulic Design Features. Hydraulic design features incorporated into the alternatives included levee raises, erosion protection, closure structures and setback levees.

b. Wind Wave Analysis: An analysis of wind wave run-up, wind wave setup, overtopping discharge, and wind wave erosion was conducted for levee reaches within the study area.

c. Project Performance and Flood Risk. Performance and Flood Risk were assessed using the USACE FDA model version 1.2.5a (USACE, 2010). The FDA model combines flow-frequency, stage-discharge, geotechnical fragility, and stage-damage relationships to estimate damages. Uncertainty in each relationship is incorporated by assigning uncertainty estimates and applying a Monte Carlo type approach to combine the results.

d. Potential Adverse Effects. A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system. Depending on the location within the project area induced flooding was determined to be either equal to the no action alternative, or was reduced compared to the induced flooding potential for the no-action alternative.

e. Climate Change. The delta reaches of the study area are affected by changes in sea level. Project performance was estimated for both 2010 (beginning of economic analysis) and 2070 (end of economic analysis) conditions using the hydraulic model results for 2010 and 2070 sea level conditions at downstream boundary conditions.

f. California State Urban Level of Protection (ULOP). Although the California State Urban Levee of Protection is not a federal objective of the study, it is a local sponsor objective. For levees to meet the ULOP requirements they must be designed to meet the requirements in the State of California Urban Levee Design Criteria (ULDC). The hydraulic performance of each alternative relative to the ULOP requirements was conducted. The results are provided in the hydraulic appendix.

g. General Hydraulic Design: All project features were designed to meet current USACE design requirements. It should be noted there is no specific design requirements for levee height. The design height of the final alternatives is based on reasonably maximizing net benefits. The determination of maximum net benefits is described in the economic appendices and the plan formulation document.

2.8 Soil Design

2.8.1 General

The geotechnical appendix presents the results of geotechnical analyses and feasibility level geotechnical recommendations to address technical deficiencies in the flood risk management system protecting the Lower San Joaquin River Feasibility Study area (LSJRFS). For the geotechnical engineering evaluation of the LSJRFS area, the following tasks were performed and summarized in the report:

- review of currently available geology, geomorphology, and geotechnical information
- review of past performance and flood control system construction history/improvements
- identification of levee performance deficiencies through geotechnical analysis and engineering judgment
- probabilistic geotechnical analysis and development of levee performance curves
- seismic study of existing levees
- development of geotechnical conclusions and recommendations

2.8.2 Design Criteria

USACE standard levee design and construction criteria as established in both national (HQ) and local (District and Division) policy documents were followed during analyses and development of mitigation regarding geometry, seepage and stability, vegetation and access, fill material, bank protection, and seismicity and liquefaction.

2.8.3 Evaluation of Existing Condition

Existing conditions were initially characterized by 14 Index points representing approximately 40-miles of existing levees within the study area. These 14 index points were selected for geotechnical analysis to represent the critical surface and subsurface conditions of each planning reach in order to identify the geotechnical deficiencies of the reach. The sections were selected based on previous geotechnical analysis, past levee performance, existing levee improvements, subsurface data, laboratory test results, surface conditions, field reconnaissance, and levee geometry. As part of the Planning process additional lengths of existing levees and also potential new levee alignments were added, expanding the project study area to roughly 90 miles. All of the existing and proposed levees with-project conditions were analyzed using the 14 index points.

Potential sources of levee distress considered in the analyses were underseepage through the levee foundation, through-seepage through the levee embankment, and instability of the landside levee slope under steady state conditions. The levees were evaluated against the above mentioned performance modes at five different water surface elevations. Using this method of selecting loading conditions, the levee performance curves would theoretically represent probability of poor performance at multiple flood frequencies.

For the results of the fragility curve, a judgment based conditional probability function was provided based on the existing and past erosion history of the levee and riverbank, maintenance, encroachments, vegetation on the levee slopes and within the levee critical area, animal burrows and other external damaging conditions. The total conditional probability of poor performance of the levee as a function of water elevation was developed by combining the probability of poor performance functions for four failure modes: underseepage, through-seepage, slope instability, and judgment.

2.8.4 Conclusions

2.8.4.1 South Stockton

The analyses performed for South Stockton indicated that the levees represented by index points LR-1, LR-2, and LR-3 in RD-17 did not meet minimum levee design criteria at various flood frequencies. Historical documentation indicates performance-related issues with seepage, slope instability, and erosion. The measures identified in this study to mitigate these performance issues, to create with-project conditions, typically included a cutoff wall and/or seepage berm.

2.8.4.2 Central Stockton

The analyses performed for Central Stockton indicated that the levees represented by index points FR-1 in RD-404, and SL-1 and SL-2 along Stockton Diverting Canal did not meet minimum levee design criteria at various flood frequencies. Historical documentation indicates performance-related issues with seepage and erosion along RD-404, erosion along the left bank of the Calaveras River with isolated areas of seepage, and erosion along the left bank of Stockton Diverting Canal. The measures identified in this study to mitigate these performance issues, to create with-project conditions, typically included a cutoff wall and/or seepage berm.

2.8.4.3 North Stockton

The analyses performed for North Stockton indicated that the levees represented by index points CR-1/CR-2 and D-4 along the right bank of the Calaveras River, and index point D-BS along Delta Brookside, did not meet minimum levee design criteria at various flood frequencies. Historical documentation indicates performance-related issues with settlement, seepage, erosion, and animal burrowing activity along the Delta Brookside study area, and seepage and erosion along Delta Lincoln Village study area. The measures identified in this study to mitigate these performance issues, to create with-project conditions, typically included a cutoff wall and/or seepage berm.

2.8.4.4 Seismic Study

The results of seismic and liquefaction evaluation indicated isolated areas throughout the study area that are capable of inducing significant deformation of the levees. Some of the levees in North Stockton are classified as frequently hydraulically loaded levees due to the tide and may be susceptible to significant deformation due to a seismic event. However, most of the study area is unlikely to be capable of inducing flow failures, and thus deformation is not likely.

2.8.5 Recommended Design Recommendation

With the exception of some proposed closure structures and set-back levees, the predominant project recommendation was fix-in-place of existing structures. The predominant measure chosen to mitigate areas of poor performance was a cutoff wall and/or a seepage berm.

2.9 Civil Design

2.9.1 General

The PDT's decision at the beginning of the feasibility study was to utilize a computer based cost estimating system. The system would produce preliminary estimates within the short time frame and resources which the team faced under 3^3. The quantitative work was based off Figures 2 through 5.

2.9.2 Abbreviations and Names

The following abbreviations correspond to the following location names for Figures 2 through 5, and for the cost estimating results below.

Table P. Names and Abbreviations for Levee Reaches for the North and Central Stockton Area and RD17

Abbreviation	Location Name
ST	Shema Tract (between Mosher Creek and Five Mile Creek)
MC	Mosher Creek
FM	Fourteen Mile Slough
FS	Five Mile Slough (between Shema Tract and Fourteen Mile Slough)
TS	Ten Mile Slough (between Fourteen Mile and Calaveras)
CR	Calaveras River
SDC	Stockton Diverting Canal
MS	Mormon Channel
SJR	San Joaquin River in the areas of the delta, RD404, and RD17
FCS	French Camp Slough
PTC	Potter Creek (SDC extension)
SC	Smith Canal
DC	Duck Creek (French Camp Slough extension)
PC	Paradise Cut

2.9.3 Parametric Estimating

The parametric software tool SPK used to calculate construction quantities is called PCET (short for parametric cost estimating tool). The PCET program contains levee fix templates for calculating quantities by inputting geometric variables and design inputs. These variables conformed to EM 1110-2-1913 "Design and Construction of Levees," Sacramento District CESPK-ED-G, SOP-03: "Geotechnical Levee Practice," ETL 1110-2-571 "Guidelines for Landscaping and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures." PCET inputs relied on ULE and National Levee Database datasets. Unit costs were then applied to PCET outputs in order to determine parametric costs. These unit costs were based on past projects in the vicinity of Sacramento, adapted to the San Joaquin area.

2.9.4 Segmental Cost Estimates

Based on experience with similar projects, the PDT began work using small project reach segments which were estimated for cost. This was a particularly useful strategy since the hydraulic flood plain analysis work wasn't complete and without it one could not predict where a flood protection project would begin or end. Furthermore, any future refinements of the work wouldn't be possible unless existing segments located beside the flood protection project were already completed (and could be either added incrementally, or deleted). The study area contains 92 miles of levee which was further evaluated using approximately 130 segments that were on average 3,700 feet in length. The result of this segmentation is presented in Figures 2 through 5. The figures help define the resultant fix locations presented in Table 4.

2.9.5 Alternative Estimates

2.9.5.1 General Construction

Alternative cost estimates were prepared for focused and final array alternatives. The cost estimate for these alternatives are based on estimated quantities that are translated to costs when implementing an array of new levees, fixing existing levees, or incorporating new features within existing levees. The estimates are based on the type of fix needed such as a cutoff wall, a seepage berm, rock revetments, or general geometry improvements. What was also taken into consideration was the probability of requiring a bridge, or if channel improvements were needed. Other cost considerations included real estate acquisitions, environmental and cultural resources mitigation, O&M, design costs, encroachment relocations, and construction management costs.

2.9.5.2 Construction with Raise

Corps guidance requires that sea level rise be taken into consideration for a 50 year time horizon. The ensuing sea level rise factors into the planning for existing project levee heights. The PDT concluded that a few areas required this levee improvement in height which resulted in levee raises in a few locations along RD17, North Stockton, Central Stockton and the Delta Front. The sea level rise estimates were added to the final array of alternatives creating LS-7, LS-8 and LS-9. Only minimal height raises were needed to meet this objective and were included into the focused array estimates. The incremental addition of SLR proved to be economically cost justified with increased net benefits. Alternatives LS-2A through LS-4 do not incorporate height improvements for sea level rise, and thus were not considered further for the focused array. The list in Table D does not include alternatives LS-2A through LS-4 for this reason.

2.9.5.3 Real Estate

The study initially based the cost estimate of determining affected real estate parcels on the District's standard 20-foot landside easement. The cost segments were evaluated on land use types which were orchard, agriculture, residential, or commercial. The sponsor requested an exception to reduce the landside easement to 10-feet. The smaller easement was granted since alternatives LS-7a, LS-8a, and LS-9a have on average approximately 600 parcels which would likely require a real estate take. The 10-foot easement was only adopted for existing federal system levees where the levee toe remains fixed. The system is considered a new levee if the toe of the levee encroaches on the existing easement and a 15-foot easement is required. If the easement on an existing levee whose toe remains fixed is less than 10-feet there is a requirement to purchase additional property necessary to comply with the 10-foot requirement. A waterside easement of 15-feet is required regardless of whether this is an existing levee (Federal) or if this is a new levee (non-Federal). Securing this easement is expected to be a relatively low cost and was excluded from the parametric estimates. The cost is to be evaluated during the TSP revision. Encroachment and woody vegetation removal, remediation, or relocation costs were not added to the total project cost because the local sponsor owns and maintains the 10-foot landside easement requirement already.

For new levees the design will include a 15-foot right-of-way (ROW) per the ETL measured from the levee toe for both water side and land side. Should a seepage or stability berm be required the ROW is measured from the toe of these berms.

2.9.5.4 Operation and Maintenance

Operation and maintenance costs were reflective of additional effort by the local managing agency (LMA) to properly maintain new features. The increased level of effort was qualitatively evaluated and assigned a percentage based on increased O&M cross section and best judgment. The LMA's annual budgets were used to prorate costs per length of maintained area and were multiplied by the increased percentage of effort to obtain an annualized O&M cost. Some of the items that were qualitatively evaluated when determining the increased level of effort were the following.

- Inspection area
- Mowing and vegetation control
- Rodent control
- Pumps, valves, and appurtenances

Operation, Maintenance, Repair, Replacement and Rehabilitation (OMRR&R) values did not include the LMA's existing budgets to maintain new features. In many cases the project improvements should reduce O&M efforts. However, the PDT determined that additional OMRR&R costs should be factored to account for project features over a 50-year design life. After selection of the TSP, future discussions with the LMA's about project features and how O&M will be implemented shall refine these estimates further.

2.9.5.5 Encroachments

Department of Water Resources (DWR) levee logs contributed to most of the utility inventory. Other logs were available as GIS data from the City of Stockton. For areas with no coverage the unallocated item cost and construction cost contingency was used for estimating purposes.

Utility relocation costs were generated from a series of typical penetration conditions. Most often the fix involved raising pipe(s) invert above the design water surface level through the levee. This typically involved replacing the pipeline and adding positive closure valves to meet Corps EM 1110-2-1913 policy.

2.9.5.6 Pre-Construction, Engineering and Design (PED), and Construction Management (CM)

The cost estimates included both PED and CM which were assigned a percentage of the construction, environmental mitigation, and utility relocations. PED was assigned a value of 15% based on historical values. CM was assigned 10% of the costs.

2.9.6 Borrow Sites/Disposal Areas

Five borrow areas have been identified as potential borrow sites. The first of these locations is west of the Stockton East Water District (SEWD) water treatment plant (WTP). The SEWD is interested in providing a borrow site near the WTP in order to excavate through a fairly impenetrable clay layer that would allow water recharging to occur more easily after the borrow material has been removed. This site would be 265 acres and could potentially be excavated as deep as 20-feet.

Another site would be at the Tidewater development near French Camp Slough and Highway 99. This site is a 93 acre basin with potentially 1,700 acre-feet of earth volume.

At the Mariposa Lake Development nestled between Mariposa Road and State Route 4 east of S.R. 99 is another potential borrow site. The entire site is approximately 6 square miles and approximately 3,500 acres of the site would be available for borrow.

Over 1 million cubic yards of unsuitable soil are expected to be used at commercial and local disposal sites. Additionally, some of this soil can be used to mitigate for the borrow areas and fill in low spots. The estimate is that 50% of excavated material will be able to be reused.

2.9.7 Construction Access, Haul Routes and Staging Areas

For construction and staging areas the early planning analysis indicates that sufficient sponsor, county, or city property exists that additional areas do not need to be purchased. These local properties in the form of empty lots, right-of-ways, and easements would be available for these functions. Thus, specific access and staging areas were not identified. In areas where the sponsor lacked proper access or easement, the “unallocated items” and contingencies within the cost estimate would appropriately cover the additional lands needed to facilitate construction of the flood risk management features.

During the early planning of alternatives, haul routes were not identified. Haul routes are expected to be fairly direct between the borrow areas and the construction. Borrow areas are expected to be located within 25 miles of the construction. Additionally, multiple borrow areas are expected to be needed. It is unclear which borrow areas would continue to be viable until the start of construction, and thus the time and effort spent identifying specific haul routes may not prove beneficial.

2.10 Cost Engineering

2.10.1 General

The cost estimates under the study have been prepared under ER 1110-2-1302 Civil Works Cost Engineering which describes levels of detail with respect to cost. The classes are based on ASTM E 2516-06, Standard Classification for Cost Estimate Classification System. The Parametric Cost Estimating Tool (PCET) used to parametrically define the initial and final array of alternatives is based on a Class 4 level of detail. The Tentatively Selected Plan (TSP) is based on a Class 3 level of detail prepared using computer aided cost software (MCACES) and is referred to as the TSP in this report.

The quantities and project cost estimates for the final array of alternatives were prepared by Civil Design utilizing unit costs for typical construction items as developed by Cost Engineering Section and other cost data furnished by the Environmental Planning and Real Estate sections. A summary of estimates for the final array of alternatives is provided in the appendix to this engineering summary.

Real estate estimates were based on footprint requirements for project construction, operation and maintenance provided by Civil Design Section A. Alternative estimates were prepared based on refinements to the preliminary layouts, features, and measures as determined by screening analysis as performed by Planning Division, and input from the potential non-Federal sponsors. Design guidance for cost estimates comes from ER 1110-2-1302, Civil Works Cost Engineering.

2.10.2 Cost Engineering Analysis

This section indicates Cost Engineering results for the final array of alternatives leading to the TSP. There are seven alternatives in the final array as listed below. For descriptions of the alternatives, see Section 2.4.4 – Final Array.

2.10.3 Preliminary Cost Analysis

2.10.3.1 Quantity Takeoffs

Quantities for most project items relative to levee construction/modifications were developed by Civil Design Section using a spreadsheet tool. This spreadsheet utilizes generic cross sections with predetermined cost elements (typical levee work such as clearing and grubbing, earth fill, aggregate base, etc). Civil Design provides quantities for those elements based on input of design levee parameters as determined by the Geotechnical Section.

2.10.3.2 General Methodology in Cost Estimate Preparation

During this period of alternatives study leading to the TSP, ER 1110-2-1302 requires Class 4 Cost Estimates as a minimum. Class 4 estimates are primarily stochastic in nature with an expected accuracy range index of 3 to 12 where the value of ‘3’ represents +30/-15 percent and a value of 12 represents a +120/-60 percent range. In developing the class 4 cost estimates for the alternatives, the Cost Engineering team (Cost Engineers and Civil Design Engineers) utilized a number of different methods to determine project costs.

2.10.3.3 Levee Improvement Cost Summary

Generic/parametric/characteristic unit construction costs for many typical levee improvement elements were developed using estimating software MII (MCACES, 2nd Generation). For a typical element such as a slurry wall or borrow material (acquisition and placement), a unit cost was established based on a 'typical' crew, production rate, material cost, assumed/typical haul distance, etc. Davis Bacon labor rates (2014), MII Equipment rates (2011 Equipment Book), current fuel prices (2014) and generic/typical Contractor markups were utilized to establish unit costs. For any particular levee improvement (such as to fix-in-place the levee by degrading, placing a slurry wall/seepage barrier and restoring the levee), the estimating exercise sums the quantities times the unit costs, adds a percentage for such items as mobilization and demobilization, and indicates a total cost per linear foot of levee improvement.

2.10.3.4 Historical Cost Data

Historical unit costs for some items have been utilized based on cost estimates for past projects in the vicinity of Sacramento. For example, pump station costs were based on costs for similar pump stations developed for the Natomas PACR. Cost data was also supplied by other disciplines, specifically Real Estate and Environmental (Mitigation).

2.10.3.5 Cost Engineering Experience

Cost Engineering judgment and experience was used to base some costs on a percentage of construction costs (e.g. Preconstruction Engineering and Design / PED cost, Construction Management cost). The percentages are based on historical data and typical rates used by SPK Cost Engineers in the past.

Each alternative consists of several separable areas divided into reaches/sub-alternatives of various lengths and each reach has an associated type of levee improvement. The sum of all applicable costs for each reach is entered into a spreadsheet that is a compilation of total project costs. The total project cost summaries (first cost) follow the Civil Works Work Breakdown Structure (CWWBS) code of accounts. Feature Codes typically involved in this estimate are 01-Lands and Damages (Real Estate), 02-Relocations, 06-Fish and Wildlife Facilities, 11-Levees and Floodwalls, 18-Cultural Resource Preservation, 30-Preconstruction Engineering and Design, and 31-Construction Management. The 30 and 31 accounts involve any costs associated with USACE staffing on the project for the federal share and anticipated costs associated with local sponsor costs for the non-federal share. The cost estimate for each Alternative is the summation of the costs from the major cost categories. The costs do not account for life cycle costs.

2.10.3.6 Environmental and Cultural Considerations

Environmental and cultural mitigation costs were developed as a percentage of total construction cost (on an incremental cost segment basis). The percentages for environmental costs ranged from minimal (5%) to high (35%) and dollar values were based on past historical SPK projects and judgment. The percentage for cultural costs were estimated at approximately 1% of the construction costs and included in the total project costs. Maps and geospatial tools were used to help evaluate segments and identify potential impacted resources. In addition, mitigation for borrow sites and for flood reduction management features were included in the overall environmental mitigation costs.

2.10.3.7 OMRR&R Costs

For a description of how the O&M costs were derived, refer to section 2.9.5.4. Table Q. provides the annual cost of OMRR&R for each alternative.

Table Q. Annual LSJ OMRR&R Costs

OMRR&R COSTS		
Alternative	**OMRR&R Annual Cost**	**OMRR&R Lifespan Cost (50 yr)**
7a	$274,800	$13,740,000
7b	$386,700	$19,335,000
8a	$296,600	$14,830,000
8b	$408,500	$20,425,000
9a	$344,800	$17,240,000
9b	$456,700	$22,835,000

2.10.3.8 Total Project Schedule (including Construction)

No formal construction schedule has been developed at this stage, but the assumption has been made that the yearly federal monetary allotment for the project will be approximately $100M. The initial PED portion of the project is assumed to take about 2 years, with approximate total duration until construction completion for each alternative in the final array as indicated in the following table:

APPROXIMATE DURATION	
Alternative	**Years**
7a	12
7b	15
8a	12
8b	15
9a	12
9b	15

2.10.3.9 Cost Uncertainties & Risk Analysis

There are inherent uncertainties in the costs at this level of design (alternatives analysis) since there is no detailed design, plans or specs. There are also inherent uncertainties as the construction contractor(s) are responsible for obtaining the construction materials, accomplishing the work in a timely manner as per the project due date, using overtime and/or multiple crews to accomplish the same, etc. Funding appropriations are typically uncertain. The Central Valley of California is home to many threatened/endangered species that require much of the work to be done within certain construction windows, typically May-October.

For this project, more than 50% of the costs for this project are directly related to levee improvements. A large percentage of this is obtaining and hauling materials for placement of levee fill or impervious fill material (clay cap). For the purposes of the cost estimate, the assumption has been made that stone material will be placed from the landside (trucked). Stone materials are expected to come from either the Bay Area or the Sierra Nevada mountains. Much of the existing levee material can be re-used but still must be hauled to/from stockpiles. Impervious fill is assumed to come from within 25 miles (one-way haul). The potential contractors are free to obtain borrow from wherever they see fit, as long as it meets specs. Haul costs in general have some uncertainty as material supply locations are up to the contractor, as well as whether the contractor uses their own trucks or utilizes independent truckers for hauling. Another work feature of high risk/costs are cutoff walls, particularly those using the deep soil mixing (DSM) method, which requires significant placement time.

An Abbreviated Cost Risk Analysis (ACRA) using the Cost MCX Abbreviated Risk Analysis Template (spreadsheet) was performed for each of the final array of alternatives. The alternative was divided into its main component areas (e.g. North Stockton, Central Stockton, and RD17) and risks were assessed relative to each area. The summary sheet for each alternative ACRA is included in the appendix to this engineering summary.

The ACRA meeting was held 4 NOV 2013 with the project manager and most PDT members. The meeting focused primarily on risk identification using the CRA template and brainstorming techniques. The risk analysis process involved dividing project costs into typical risk elements and placing them into a Risk Register, then identifying the risks/concerns relative to those risk elements, and then justifying the likelihood of the risk occurring and the impact if the risk occurs. A Risk Matrix utilizing weighted likelihood/impacts is used to establish the cost contingency to use for each risk element (work feature) for use in alternatives comparisons. Project risks were identified and the risk register developed within the spreadsheet for the component areas of each alternative. The likelihood of an impact on each risk element was assessed by the PDT. The draft risk register and results were then forwarded to the PDT for review.

Risk elements were identified for each alternative based on the Civil Works Work Breakdowns Structure (CWWBS) and work feature. Prime construction work features identified were Earthwork, Cutoff Walls, DSM walls (Seismic), and Slope/Erosion Protection, These items typically accounted for 80 percent or more of the costs, except for the Central Stockton area, where there are several diversion structures and bridges that are, with remaining construction features such as mob/demob, relocations, and hydroseeding, lumped together in a category for ‘Remaining Construction Items.’ The risk register thus serves the purpose of historical documenting as well as to support follow-on risk studies as the project and its accompanying risks evolve. The results of the ACRA therefore reflect the risk register parameters and are considered adequate for establishing contingencies for alternatives comparison.

To fully recognize its benefits, risk analysis must be considered as an ongoing process conducted concurrent to, and iteratively with, other important project processes such as scope and execution plan development, resource planning, procurement planning, budgeting and scheduling.

2.10.3.10 Screening Level Costs

For draft Project First cost for each alternative (including the contingencies), see Chapter 3 of the draft report. All costs are considered preliminary and are only to be used to compare the relative cost between the Alternatives. Focus on the Cost Engineering data has been on the alternatives. Once the PDT has selected the TSP and any locally preferred plan (if different from the TSP), Feasibility Level design details and quantities (by Civil Design) and Cost Engineering data must be developed. This includes creation of feasibility level plans and associated quantities, development of a detailed MII estimate, a Total Project Schedule (including Construction), PDT estimates for Planning, Engineering and Design, an updated Cost and Schedule Risk Analysis and a Total Project Cost Summary (TPCS) extending costs out through the life of the Project. The MII estimate must be detailed indicating labor, equipment and materials with accompanying production rates.

2.10.4 Key Assumptions

2.10.4.1 Quantities and Parametric Cost Estimates

Cross Sections for the various levee improvements or new levees are representative of the levee reach. Where design is insufficient to produce detailed quantities for each reach, the use of these typical cross-sections represents quantities adequate to screen alternatives to the point of determining a tentatively selected plan. Unit Costs utilized are fair and reasonable.

2.10.4.2 Haul Distances

Levee Fill Borrow will come from within 25 miles (one-way haul).

2.10.4.3 Project Schedule

For each area of construction, PED and Real Estate acquisition will occur over 1 to 2 years prior to commencement of construction. For construction, the duration developed is based on the assumption that the yearly federal monetary allotment for the project will be approximately $100M.

2.10.4.4 Real Estate

Real Estate Costs are reasonable.

2.10.4.5 Environmental Mitigation

Costs provided by the Environmental Specialists in Planning are reasonable.

2.10.4.6 Cultural Resources

Costs of 1.5% of the total project costs for Cultural Resources Surveys (cost shared) and 0.5% of the Federal Cost share for Data Recovery (100% federal cost) are sufficient.

2.10.4.7 PED Costs

A value of 15% of the Federal Share Construction Costs & 15% of Non-Federal Construction Costs are consistent with those used in recent years for feasibility studies performed by the Sacramento District.

2.10.4.8 Construction Management Costs

A value of 10% of Federal Share Construction Costs & 10% of Non-Federal Construction Costs are consistent with those used in recent years for feasibility studies performed by the Sacramento District.

2.11 Value Engineering

A Value Engineering Study was performed on the preliminary alternatives for this project in July 2013 with the final report date of 19 August 2013.

The objectives of the VE study were to validate, refine and optimize alternatives; facilitate communication; and improve value (increase performance and/or reduce cost). By meeting the objectives, the VE study was able to begin the process of identifying a final array of alternatives. The VE study introduced Value Metrics which analyzed cost and performance in order to calculate a project value. By the end of the VE study the effort had identified a draft final array which eventually led to the final array provided in Section 2.4.4 and Table D.

2.12 Environmental Engineering

Engineering Regulation (ER) 1165-2-132, HTRW Guidance for Civil Works Projects requires that a site investigation be conducted to identify and evaluate existing and potential HTRW issues. This HTRW Site Summary report was conducted in accordance with ER 1165-2-132 and ASTM 1526-05, Phase I ESA as a supplemental guidance. Regulatory database search reports and regulatory agencies' websites were reviewed and assessed for HTRW sites in the Study Area, along the 40 miles long levees proposed for new levee construction, modification and upgrades to the existing levees.

The Study Area for this report is defined as an area 40 miles wide along the proposed levees identified for the alternatives. The Lower Mormon Slough section was a separate study and was conducted as a Phase I Environmental Site Assessment (ESA) was completed in March 2014.

The Phase 1 report provides the data as being reasonably accurate as of May 2014. The status of HTRW sites are constantly changing and new HTRW sites may be added to the regulatory databases over time. Currently unknown HTRW sites may also be located within the study area but would not be included in this report.

The Phase 1 report lists over 100 sites which are located within 0.25 miles of the LSJ proposed levees. The alternatives share all of the known sites except for seven active/closed sites located near the Calaveras and the Stockton Diverting Canal (LS-8a). An assessment was made of the Phase 1 report list for sites located within approximately 900 feet of the Calaveras/Stockton Diverting Canal portion of the 8a levees which are presented in Table R. below.

Table R. Active and Closed Hazardous Waste Sites Specific to Alternative LS-7a and the Potential for Levee Site Clean-Up as Low, Medium, or Possible During Construction

Site	Possible Contaminant	Distance to Levee (ft)	Active or Closed Site	Potential for Levee Clean-up
Brea Ag Service 1905 N. Broadway	Pesticide, fertilizer, gw contamination	~ 250-ft	Unknown	Possible
Colon Property 5681 E. Marsh Rd.	Junkyard, possible lead in soil	~ 350-ft	Active	Medium
Beacon Property #27 3300 Waterloo Rd.	Gasoline contamination	~ 650-ft	Closed Site	Low
Fisco Warehouse 1648 Shaw Rd.	Diesel contamination	~ 900-ft	Closed Site	Low
Don’s Buggy Shop 3245 Wilson Way N	Gasoline contamination	~ 800-ft	Closed Site	Low
Certified Grocers of California 1990 Piccoli St N	Diesel contamination	~ 900-ft	Closed Site	Low
PG&E (Case #2) 4040 West Ln N	Gasoline contamination	~ 900-ft	Closed Site	Low

There is a low probability of having significant costs for contaminated soil removal based on the information provided in the Phase 1 report and from the results in Table R. The costs associated for HTRW for LS-7a are anticipated to be negligible compared to the overall construction costs. Based on this assessment, it does not appear that HTRW would have an impact on plan selection with respect to the LS-7 alternatives.

Alternative LS-9 includes the Mormon Channel bypass which was not included in the Phase 1 assessment described above. However, a Phase 1 assessment was provided for Mormon Channel early in 2014. The report highlights multiple locations of surface and subsurface waste along the banks and within the channel. Surface debris characterized in the report can be removed and disposed of properly without much incidence. What is unknown is the extent of the subsurface waste due to the surface waste which is noted. It does not appear that LS-9 would be precluded from continuing to be a viable alternative due to the anticipated costs associated with site remediation. However, it does appear that if a significant HTRW effort in Mormon Channel is needed, if the LS-9 alternative is selected as the recommended plan, and if the alternatives are all within proximity of potentially being selected, then more consideration should be given to understanding the effort relative to the LS-9 HTRW issue.

CHAPTER 3 – TSP ALTERNATIVE LS-7a

3.1 General

The proposed alternative is meant to improve the existing levee system and reduce flood risk for the Central and North Stockton area.

Alternative LS-7 is identified as the preferred plan with higher net benefits than LS-8 and LS-9. LS-7a is compliant with Executive Order (EO) 11988 which removes RD17 from the study area and therefore is not in conflict with the EO guidance. The EO requires federal agencies to avoid long and short term adverse impacts associated with the occupancy and modification of flood plains and to avoid direct and indirect support of floodplain development wherever there is a practical alternative. LS-7a has a project length of 22 ½ miles and includes geometric improvements to existing levees, cutoff walls, seismic fixes, erosion protection, control structures, and approximately 1 mile of new levee along Duck Creek. The extent of the project is shown in Figure 15. In addition, LS-7a would accommodate for height deficiencies due to future sea level rise.

The improved levee system includes a tie-back levee along the downstream portion of Duck Creek which ties into high ground near the Union Pacific Railroad berm. The new levee functions to keep high flows from flanking the existing levee system into central Stockton.

The project includes fixes and new levee along the following tributaries.

- French Camp Slough
- Duck Creek
- Mosher Creek
- Shima Tract
- Five Mile Creek
- Fourteen Mile Slough
- Ten Mile Slough
- Calaveras River

3.1.1 Feature Description – LS-7a

This section provides feature descriptions for Alternative LS-7a. The main features of LS-7a are the North and Central Stockton levee improvements.

For the individual levee segments that make up LS-7a, all of them required either geometric fixes to attain Corps standards and/or a structural improvement was necessary due to through-seepage, underseepage, or seismic deficiencies.

3.1.1.1 North Stockton Feature

The North Stockton feature length is 13.3 miles which requires 10.3 miles of cutoff wall. A cutoff wall is needed to reduce through and under-seepage. Fourteen Mile Slough and a little less than half of Ten Mile Slough did not require a cutoff wall. Reference Figures 2 and 3 for this information and for other information on the North Stockton area below.

A seismic fix was found to be required for 3 miles of levee for North Stockton. Most of California is under threat of seismic activity and these particular segments are under hydraulic loading for portions of the day which increases the risk of failure during a seismic event. Seven segments of Fourteen Mile Slough required a seismic fix (FM_20_L, FM_30L, FM_40L, and FM_60L). Two sections of Ten Mile Slough required a seismic fix (TS_10L, TS_20L).

For North Stockton a seepage berm was not recommended due to the higher cost of implementing a seepage berm relative to cutoff wall. Due to the density of housing and other infrastructure the lack of available real estate precluded the use of seepage berms in the area. A recommendation for new levee was also not a suggested part of the plan.

Levee geometry improvements are required for 4.5 miles of the North Stockton levee system. Geometric fixes would be required on Fourteen Mile Slough, the Calaveras River and Ten Mile Slough. Affected segments are FM_30L, FM_60L, CR_90R, TS_10L, TS_20L, and TS_30L.

Erosion protection improvements are required for 4.9 miles of levee along Fourteen Mile Slough, Five Mile Slough, Shima Tract, and Ten Mile Slough. This erosion protection is needed to diminish the effects of near daily hydraulic loading against the levee in these areas including wind and wave loading during storm events. The affected segments are FM_30L, FM_40L, FM_60L, FS_10R, ST_10R, ST_20R, TS_20L and TS_30L.

One control structure has been identified as being needed at Fourteen Mile Slough at high flow events. This structure would have adjustable gates and a pumping station to control water levels on Lincoln and Brookside Village levees. The operation and frequency of the gates will be defined during PED phase, but are expected to remain open normally.

3.1.1.2 Central Stockton Feature

Central Stockton features total 9.2 miles of improvements, all of which include cutoff wall. Reference Figure 2, 3, and 4 for this information and for other information on the Central Stockton area below.

A seepage berm, seismic fix, and new levee were not recommended for Central Stockton.

Levee geometry improvements are required for 2 miles of the Central Stockton levee system. Geometric fixes would be required for one levee segment of the Calaveras River and one levee segment of the San Joaquin River. Affected segments are CR_40L, and SJR_30R. Segment SJR_10_R would require geometry improvements for sea level raise.

Levees improvements along Duck Creek are necessary as a result of not improving the RD-17 levee system. These improvements help prevent flanking of the existing levees by high water from the Lower San Joaquin River. The Duck Creek levee segments are DC_20R, and DC_30R, extending to the Union Pacific Railroad embankment.

A control structure is required at Smith Canal at high flow events to keep both banks of Smith Canal from overtopping. The structure would have adjustable gates that will remain normally open and close during higher water events.

3.2 Estimated Costs

Estimated costs for the tentatively selected plan are based on parametric cost estimates. A more refined estimate of the TSP cost will be provided as part of Milestone 3.

3.2.1 Total Cost for LS-7a

The combined costs of North and Central Stockton to achieve the LS-7a alternative is provided in Table U. below.

Table S. Parametric Costs for Implementing Lower San Joaquin Alternative LS-7a

Fish and Wildlife Facilities	$49,820,000
Levees and Floodwalls	416,758,000
Floodway Control & Diversion Structures	36,631,000
Cultural Resource Preservation	14,592,000
Lands and Damages	130,971,000
Relocations	25,528,000
Pre-Construction, Engineering & Design	77,670,000
Construction Management	51,779,000
Project Cost Totals	**$803,750,000**

3.3 Construction Schedule

The construction schedule is presented in Table T below. Table T. provides a breakout of the schedule for PED, real estate, and construction for North and Central Stockton. Escalation costs are not factored into the schedule in Table T.

The schedule concludes that Central Stockton is constructed prior to North Stockton. The benefits during construction are greater if constructed in this order. The benefits also outweigh the increased escalation costs incurred by higher by constructing Central Stockton first.

Table T. Construction Schedule for the LSJ TSP Relative to PED, Real Estate, and Construction with Respect to Years for LS-7a

LSJR CONSTRUCTION SCHEDULE - ALTERNATIVE 7a

BASIN	DESCRIPTION	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028
		YEAR 1	YEAR 2	YEAR 3	YEAR 4	YEAR 5	YEAR 6	YEAR 7	YEAR 8	YEAR 9	YEAR 10	YEAR 11	YEAR 12
CENTRAL STOCKTON	PED	■	■										
	REAL ESTATE	■	■										
	CONSTRUCTION		■	■	■								
NORTH STOCKTON	PED			■	■	■	■						
	REAL ESTATE			■	■	■	■	■					
	CONSTRUCTION					■	■	■	■	■	■	■	■

The construction schedule was formulated on a variety of inputs and best estimates for production rates. The three big design constraints that needed to be evaluated holistically were: annual appropriations, construction production rates, and air emission concerns. While no specific one of these areas would drive the schedule, they all serve as inputs to the construction schedule. For the purposes of this study, an annual appropriation of 100 million (federal) was targeted along with real estate constraints of 1 year for non-title and 2 years for title acquisitions.

3.4 Conclusion

Alternative LS-7a is the recommended plan for the Lower San Joaquin Feasibility Study based on the FDA analysis for maximizing net benefits. Alternative LS-7a includes levee fixes for 22 1/2 miles including geometric improvements to existing levees, cutoff walls, seismic fixes, erosion protection, control structures. The recommended plan includes the construction of approximately 1 mile of new levee along Duck Creek and any new levees and 7a would accommodate for height deficiencies due to future sea level rise.

The cost of the recommended plan is provided in Chapter 3 of the draft report. Approximately 75% of the cost is projected for upgrades to the North Stockton area. Construction can reasonably be expected to last 12 years.

For more information on specific analysis presented refer to the various engineering appendices including geotechnical engineering/soils, and hydrology/hydraulics.

APPENDIX

TABLES

Table 1. Geographical Location and Description of Initial Alternatives for the LSJ Feasibility Study for the North Stockton and Central Stockton Area.

Geographical Location	Alternative	Description of Alternative
North Stockton	A	Delta Front from the intersection of Twin Brooks Lane and I-5 south along the existing levee located west of I-5, west on 5-Mile Slough, then south along the east side of the slough parallel to Hatchers Cir and Fort Donelson Dr encircling the north side of Lincoln Village West and continuing between W. Swain Rd and Canyon Creek Road to nearly Pershing Ave.
North Stockton	B	Delta Front from the intersection of Twin Brooks Lane and I-5 south along the existing levee located west of I-5, west on 5-Mile Slough, then south along the levee parallel to Hatchers Cir and Fort Donelson Dr continuing south along Brookside Road around Brookside Golf and Country Club continuing upstream of the right bank of the Calaveras River to El Dorado Street.
North Stockton	C	Delta Front from the intersection of Twin Brooks Lane and I-5 south along the existing levee located west of I-5, west on 5-Mile Slough, then south along the west side of the slough parallel to Hatchers Cir and Fort Donelson Dr encircling the south side of Lincoln Village West and continuing between W. Swain Rd and Canyon Creek Road to nearly Pershing Ave.
North Stockton	D	From I-5 and Lincoln Village West along the south side of the slough continuing south along Brookside Road around Brookside Golf and Country Club continuing upstream of the right bank of the Calaveras River to El Dorado Street.
North Stockton	E	From the Delta front up the right bank of the Calaveras River past the Stockton Diverting Canal to Cherryland Avenue.
North Stockton	F	Delta Front from the intersection of Twin Brooks Lane and I-5 south along the existing levee located west of I-5, west on 5-Mile Slough, then south along the levee parallel to Hatchers Cir and Fort Donelson Dr continuing south along Brookside Road around Brookside Golf and Country Club continuing upstream of the right bank of the Calaveras River to Cherryland Avenue.
Central Stockton	A	The left bank of the Calaveras River from approximately the intersection of Yacht Harbor Drive and Fairway Drive to the intersection with the Mormon Channel bypass.
Central Stockton	B	The east side of the Delta from just south of Country Club Blvd across the Smith Canal entrance (to Peninsula with closure gate structure). From the left bank of the Calaveras River from approximately the intersection of Yacht Harbor Drive and Fairway Drive to Pacific Avenue.
Central Stockton	C	From just south of the Port of Stockton shipping channel and the San Joaquin River to upstream of French Camp Slough to Walker Slough past I-5 to the first bend past I-5 on Walker Slough.
Central Stockton	D	The left bank of the Calaveras River from approximately the intersection of Yacht Harbor Drive and Fairway Drive to the intersection with the Mormon Channel bypass. The east side of the Delta from just south of Country Club Blvd across the Smith Canal entrance (to Peninsula). From just south of the Port of Stockton shipping channel and the San Joaquin River to upstream of French Camp Slough to Walker Slough past I-5 to the first bend past I-5 on Walker Slough.
Central Stockton	E	From the left bank of the Calaveras River from approximately the intersection of Yacht Harbor Drive and Fairway Drive to Pacific Avenue. Improvements around the existing levee around Smith Canal.

Geographical Location	Alternative	Description of Alternative
Central Stockton	F	The east side of the Delta from just south of Country Club Blvd across the Smith Canal entrance (to Peninsula with closure gate structure). From the left bank of the Calaveras River from approximately the intersection of Yacht Harbor Drive and Fairway Drive to Pacific Avenue. From just south of the Port of Stockton shipping channel and the San Joaquin River to upstream of French Camp Slough to Walker Slough past I-5 to the first bend past I-5 on Walker Slough.
Central Stockton	G	Diversion and improvement to Mormon Channel capacity of up to 1,200 cfs from Stockton Diverting Canal. The improvements along Mormon Channel would extend over 33,400 linear feet (6.3 miles), and include flood containment berms, bridge and culvert replacements, road relocations and channel clearing. This alternative provides for floodplain restoration in accordance with E.O. 11988 ecosystem/floodplain restoration goals.

Table 2. Geographical Location and Description of Initial Alternatives for the LSJ Feasibility Study for the San Joaquin River RD17 Area.

Geographical Location	Alternative	Description of Alternative
RD17	A	From I-5 at the south fork of Walker Slough around Westin Ranch via French Camp Slough south along the San Joaquin River to State Route 20.
RD17	B	South from State Route 20 along the tieback alignment to South Airport Way.
RD17	C	From I-5 at the south fork of Walker Slough around Westin Ranch via French Camp Slough south along the San Joaquin River along the tieback alignment to South Airport Way. (Alts A+C)
RD17	D	From I-5 at the south fork of Walker Slough around Westin Ranch via French Camp Slough south to Galley Way and French Camp Road. At Galley Way/French Camp Road traverse east, then south along S. Wolfe Way, east along W. Bowman Rd one-fourth the distance to I-5. From this location on Bowman Rd continue directly south to Dos Reis Rd and continue back to SJ River and continue along the tieback alignment to South Airport Way.
RD17	E	From I-5 at the south fork of Walker Slough around Westin Ranch via French Camp Slough south along the San Joaquin River along the tieback alignment to
RD17	F	Weston Ranch Ring Levee – includes new levee around Weston Ranch development plus an extension of RD 404 levees to prevent flanking during lower frequency events. The levees would total 6.3 miles.
RD17	G	San Joaquin River setback and tie-back extension – includes setback levees to limit protection of undeveloped floodplain within RD17. This alternative extends the tieback levee at the southern-most end of the reclamation district to minimize the probability of flanking during high water events. The setback/tie-back covers a total of 21.5 miles of levee.

Table 3. Geographical Location and Description of Initial Alternatives for the LSJ Feasibility Study for the Mormon Channel Bypass and Paradise Cut.

Alternative	Description of Alternative
Mormon Channel	Diversion and improvement to Mormon Channel capacity of up to 1,200 cfs from Stockton Diverting Canal. The improvements along Mormon Channel would extend over 33,400 linear feet (6.3 miles), and include flood containment berms, bridge and culvert replacements, road relocations and channel clearing. This alternative provides for floodplain restoration in accordance with E.O. 11988 ecosystem/floodplain restoration goals.
Paradise Cut	From the San Joaquin River to the intersection of W. Grimes Rd and S. Tracy Blvd.

Table 4. Dominant Failure Mode by Index Point

USACE Index	Failure Mode(s)
BL1	Under-seepage; erosion
BL2	Under-seepage; erosion
BL3	Under-seepage; erosion
BL4	Under-seepage; erosion
BR1	Under-seepage; erosion
BR2	Under-seepage; erosion
BR3	Under-seepage; erosion
BR4	Under-seepage; erosion
CL1	Through-seepage; landside stability; erosion
CL2	Through-seepage; landside stability; erosion
CR1	Through-seepage; landside stability; erosion
CR2	Through-seepage; landside stability; erosion
D1	Erosion; landside stability
D2	Erosion; landside stability
D3	Under-seepage; landside stability; erosion
D4	Landside stability; erosion
D5	Landside stability; erosion
D6	Through-seepage; erosion
FL1	Under-seepage; erosion
FR1	Under-seepage; erosion
LR1	Erosion; under-seepage
LR2	Seepage (through- and under-); landside stability; erosion
LR3	Seepage (through- and under-); landside stability; erosion
LR4	Seepage (through- and under-); landside stability; erosion
LR5	Seepage (through- and under-); landside stability; erosion
LR6	Seepage (through- and under-); erosion; landside stability
LR7	Seepage (through- and under-); landside stability; erosion
SL1	Landside stability; through-seepage
SL2	Landside stability; through-seepage
SR1	Landside stability; through-seepage

FIGURES

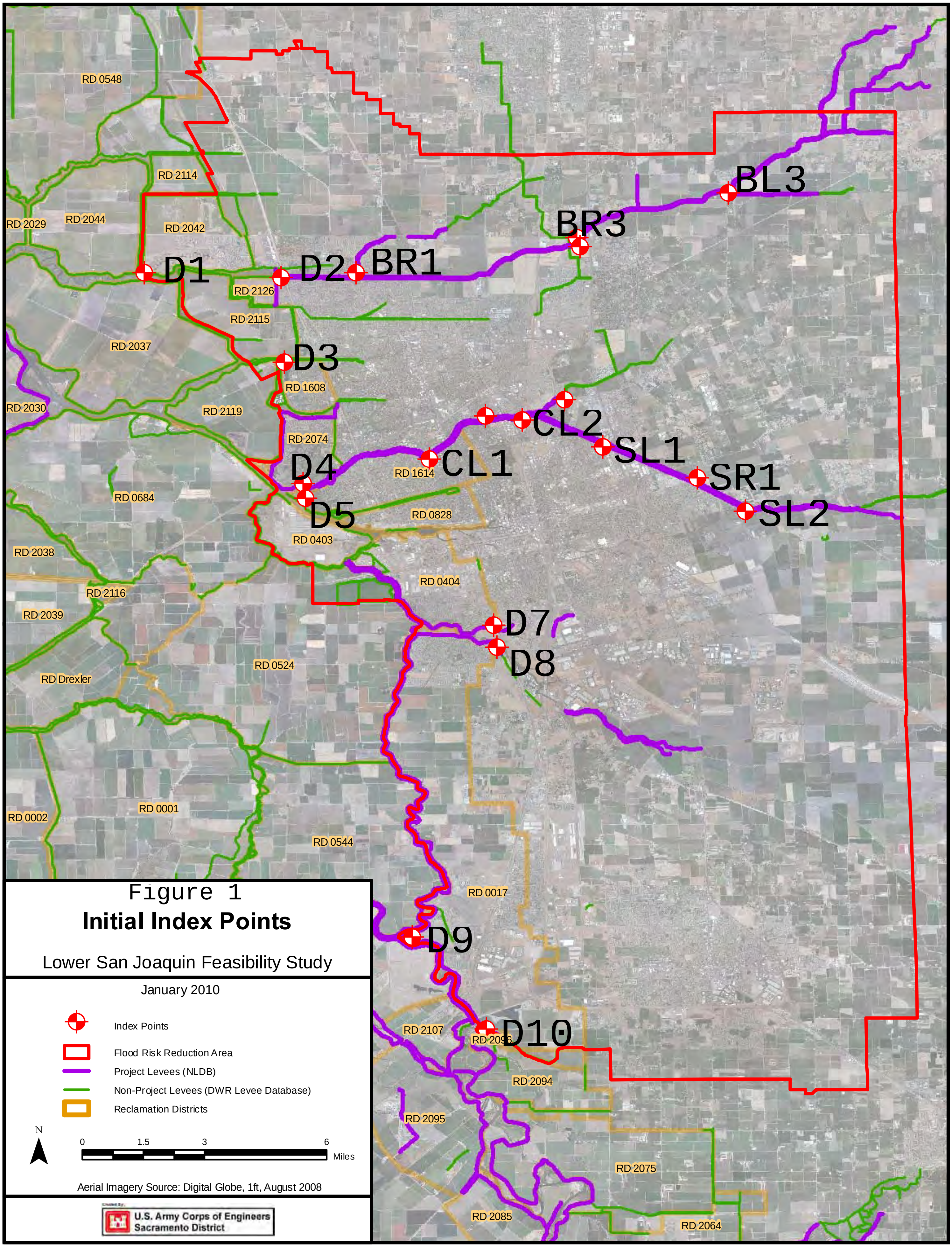


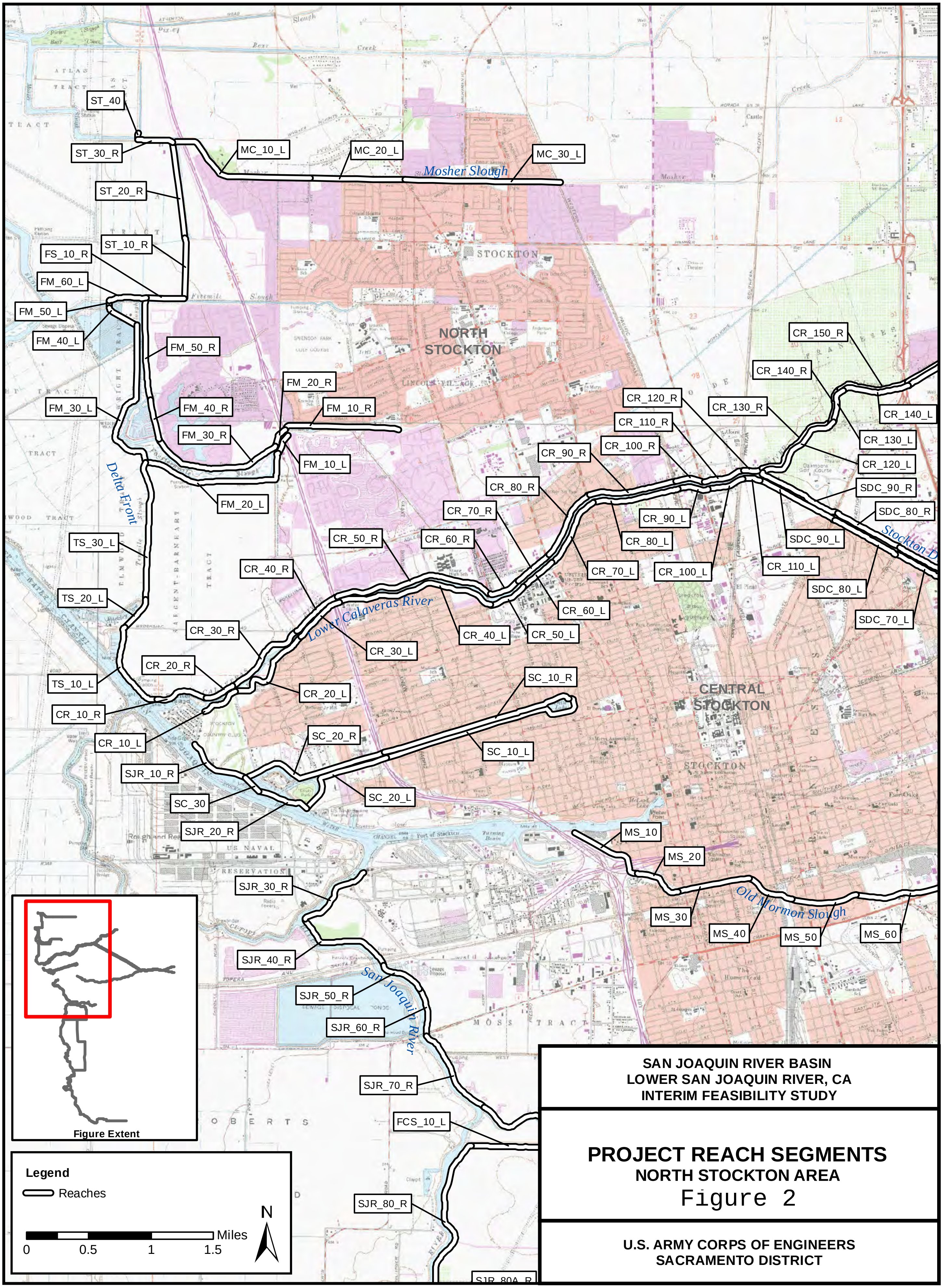
Figure 1

Initial Index Points

Lower San Joaquin Feasibility Study

January 2010

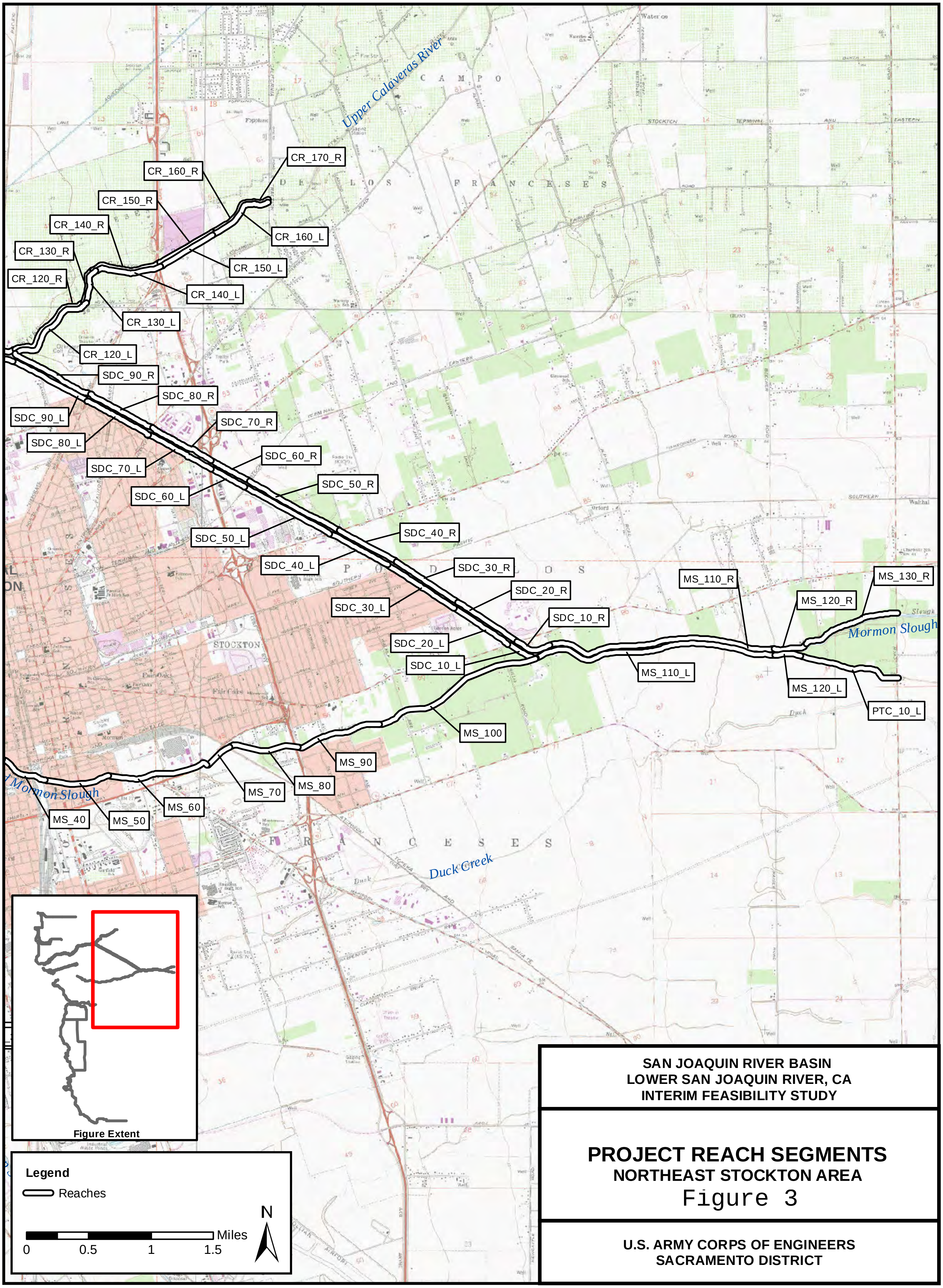
Aerial Imagery Source: Digital Globe, 1ft, August 2008



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

PROJECT REACH SEGMENTS
NORTH STOCKTON AREA
Figure 2

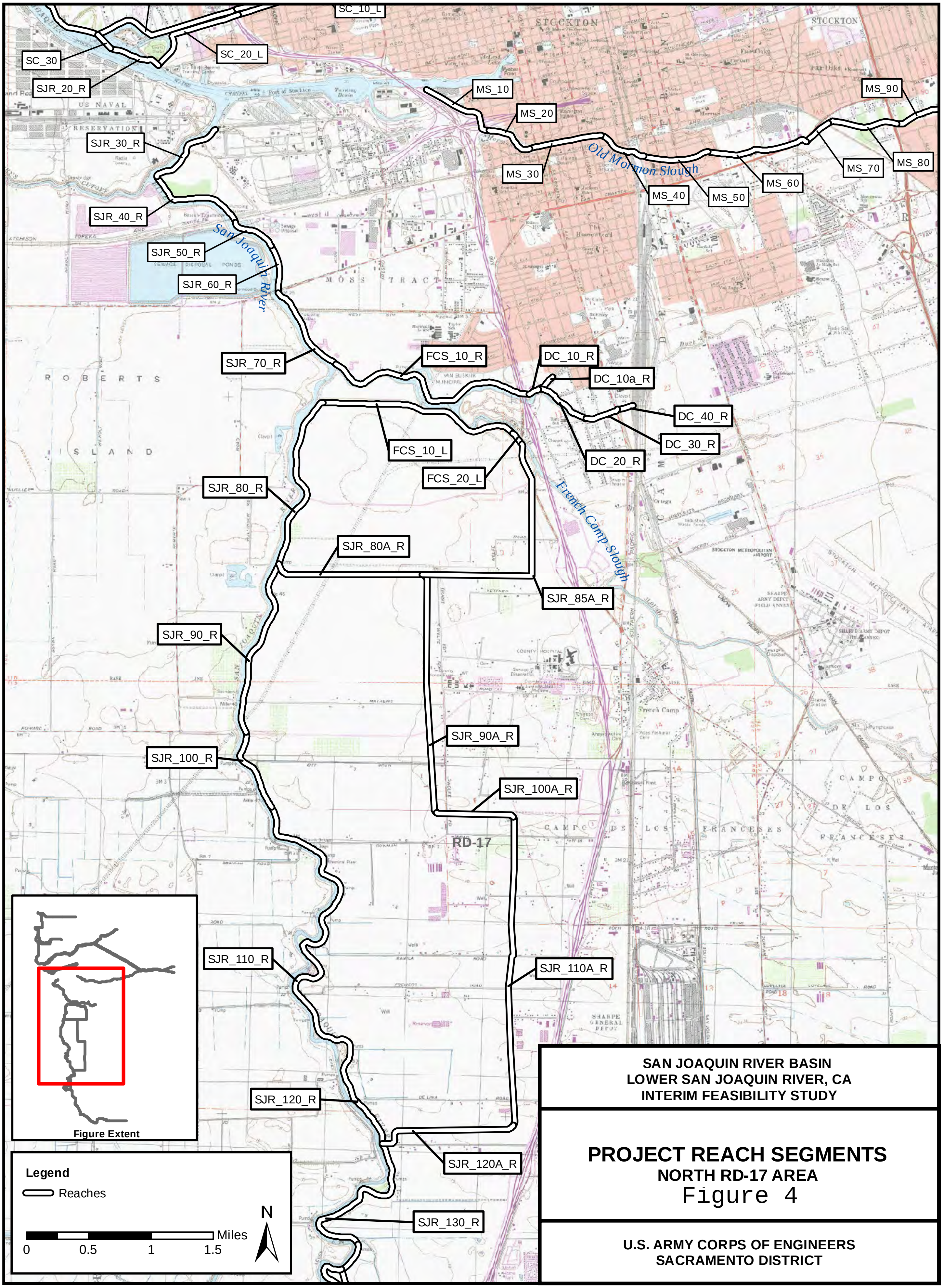
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

PROJECT REACH SEGMENTS
NORTHEAST STOCKTON AREA
Figure 3

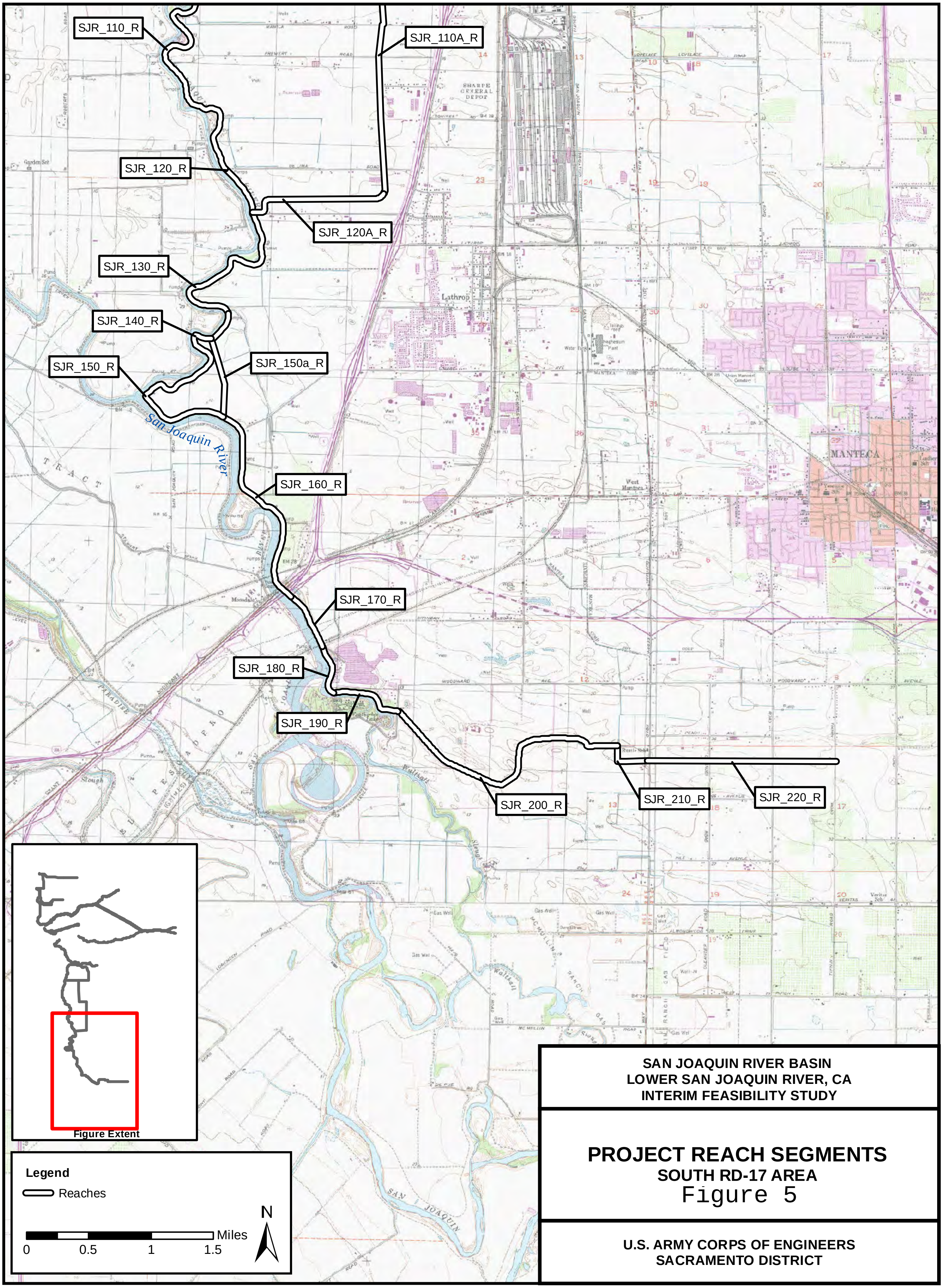
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

PROJECT REACH SEGMENTS
NORTH RD-17 AREA
Figure 4

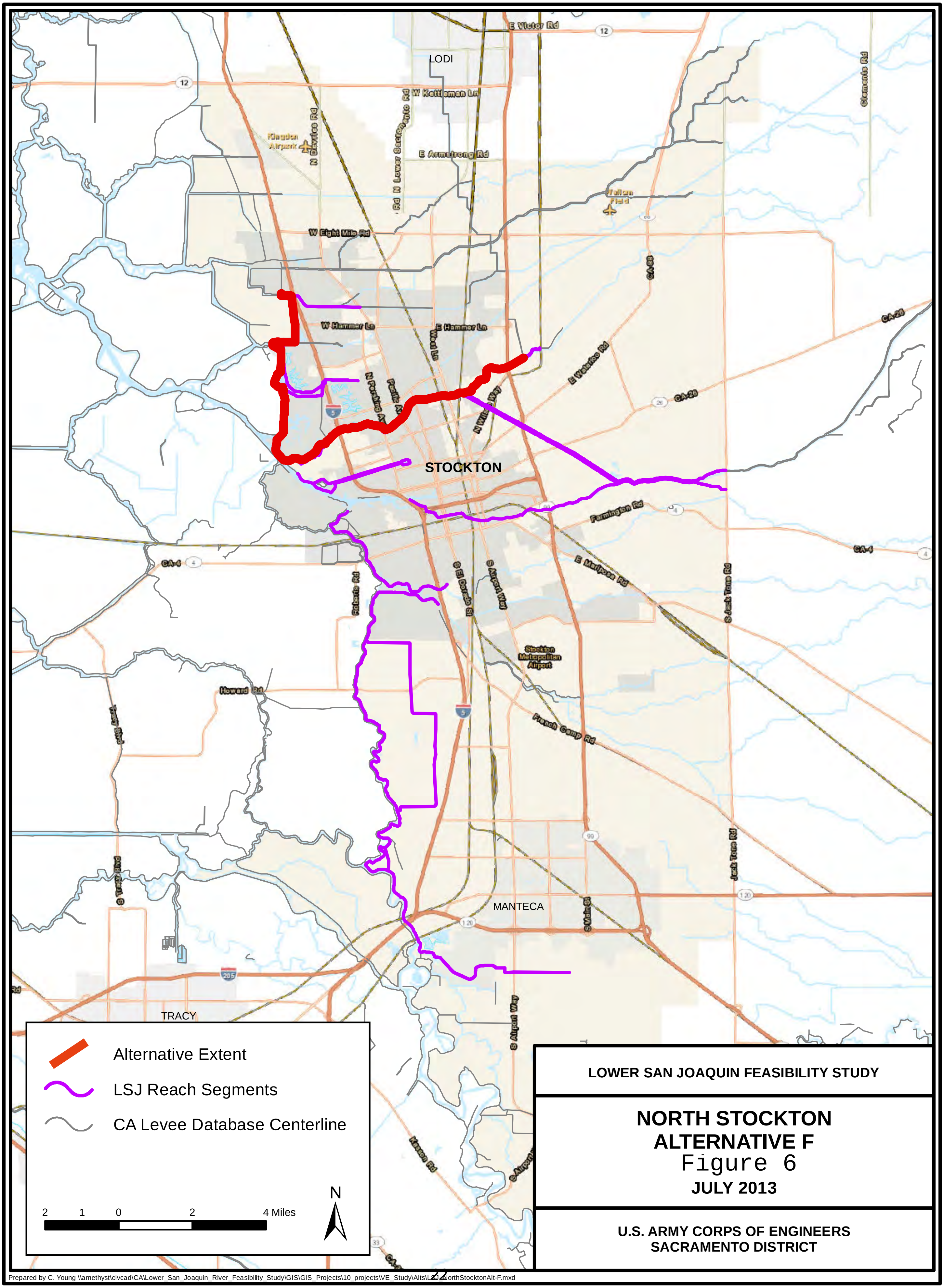
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

PROJECT REACH SEGMENTS
SOUTH RD-17 AREA
Figure 5

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



LOWER SAN JOAQUIN FEASIBILITY STUDY

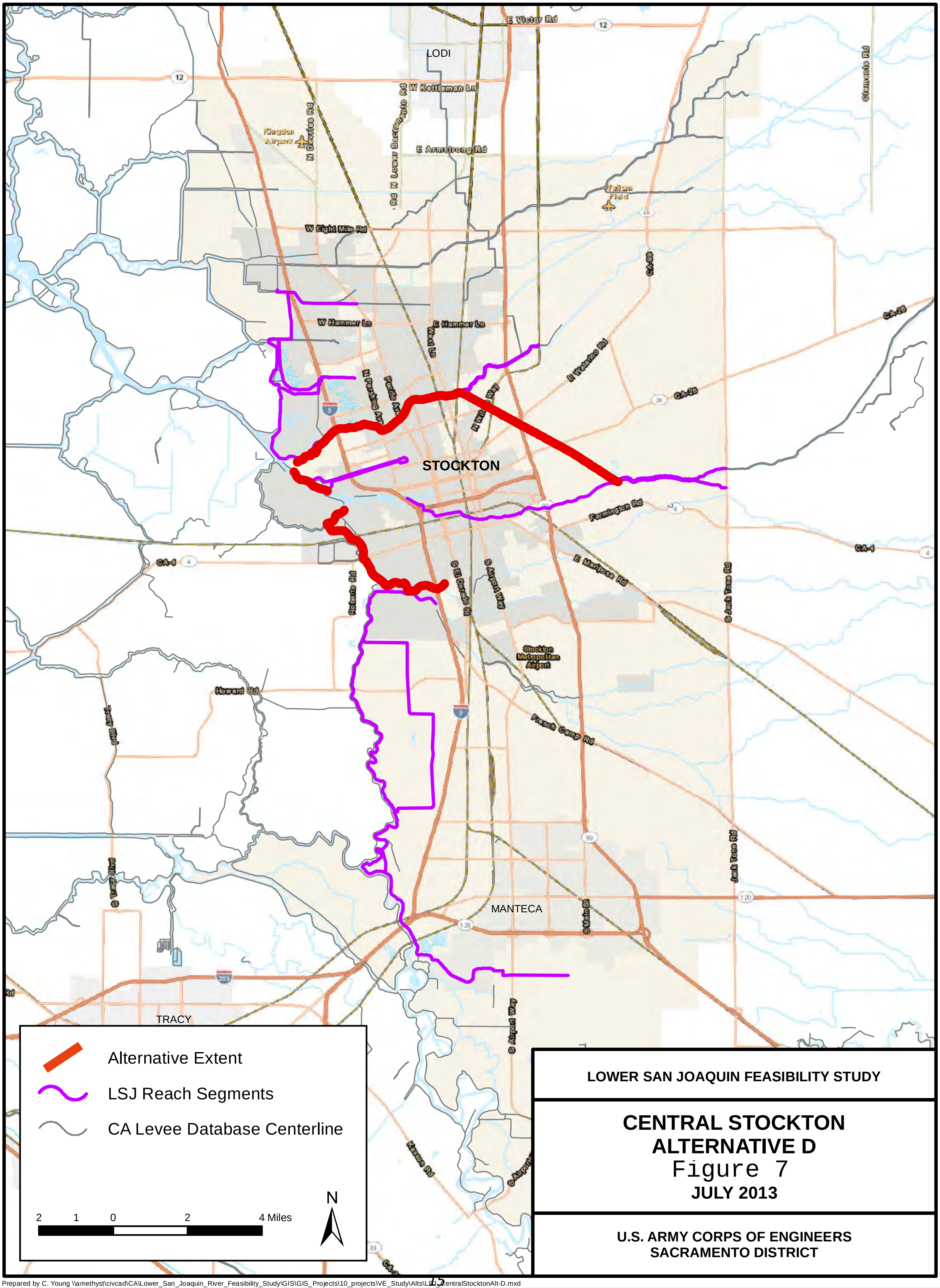
NORTH STOCKTON
ALTERNATIVE F

Figure 6

JULY 2013

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young \\amethyst\civcad\CA\Lower_San_Joaquin_River_Feasibility_Study\GIS\GIS_Projects\10_projects\VE_Study\Alts\L NorthStocktonAlt-F.mxd



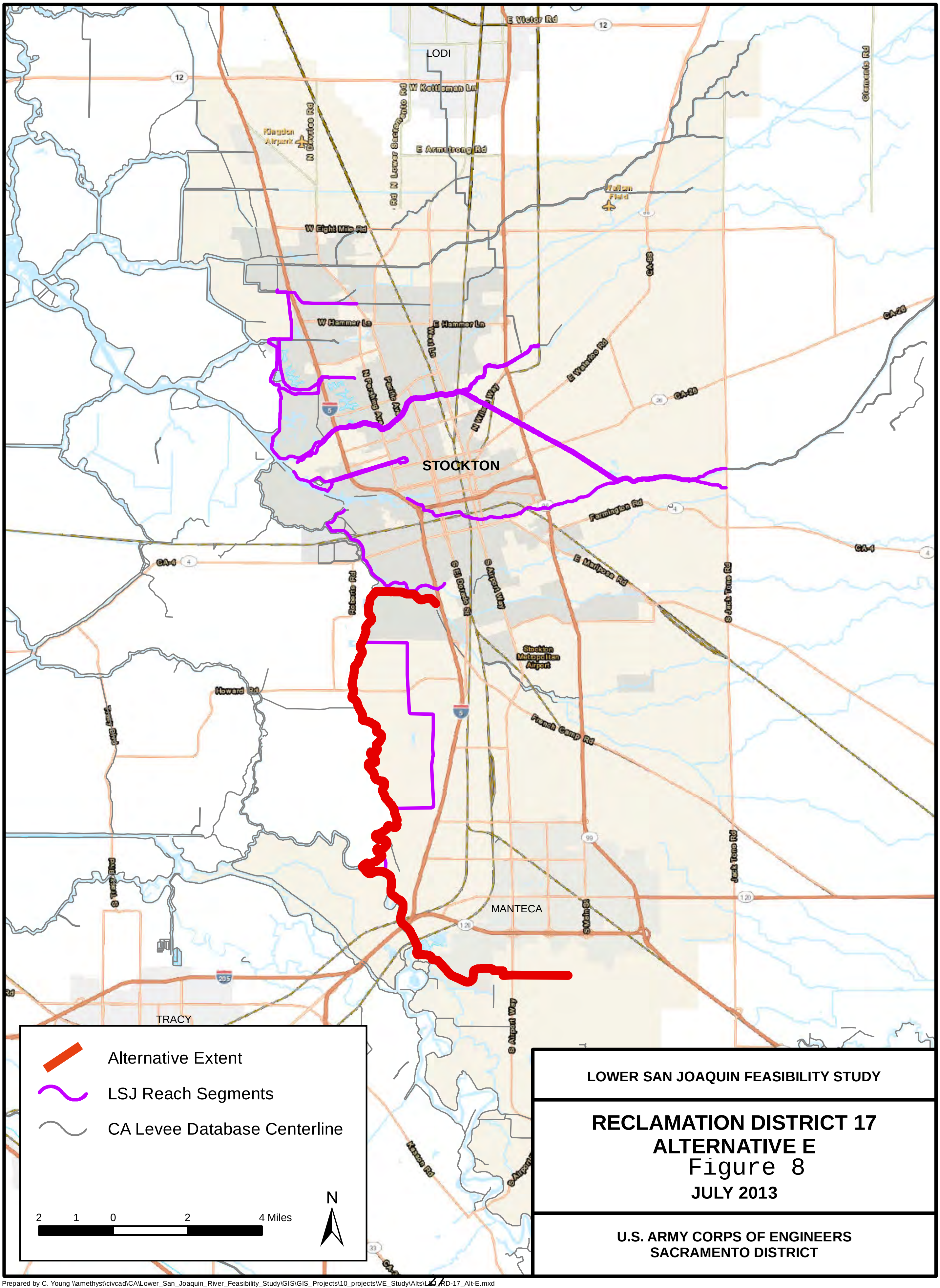
LOWER SAN JOAQUIN FEASIBILITY STUDY

CENTRAL STOCKTON
ALTERNATIVE D
Figure 7

JULY 2013

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young \\amethyst\civcad\CA\Lower_San_Joaquin_River_Feasibility_Study\GIS\GIS_Projects\10_projects\VE_Study\Alts\LSJ_CentralStocktonAlt-D.mxd



LOWER SAN JOAQUIN FEASIBILITY STUDY

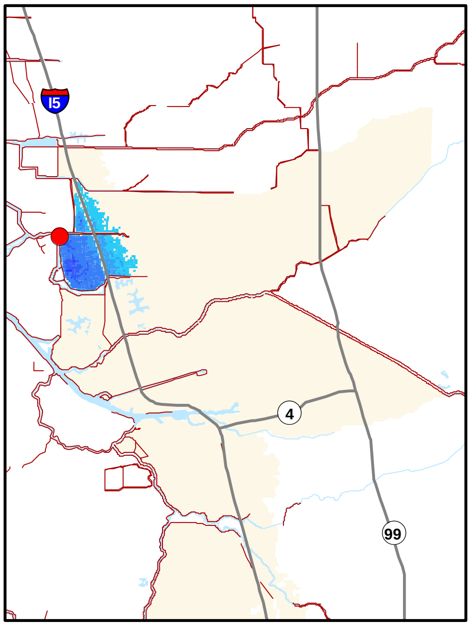
RECLAMATION DISTRICT 17
ALTERNATIVE E

Figure 8

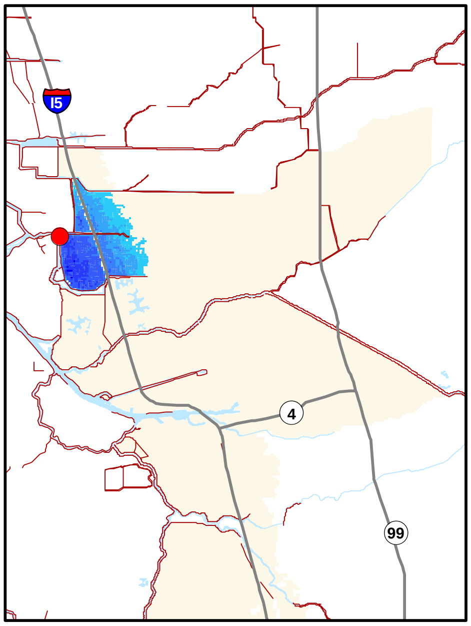
JULY 2013

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

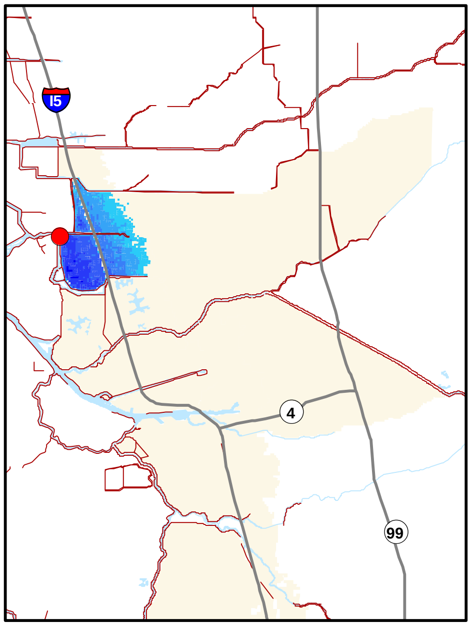
Prepared by C. Young \\amethyst\civcad\CA\Lower_San_Joaquin_River_Feasibility_Study\GIS\GIS_Projects\10_projects\VE_Study\Alts\LSJ_RD-17_Alt-E.mxd



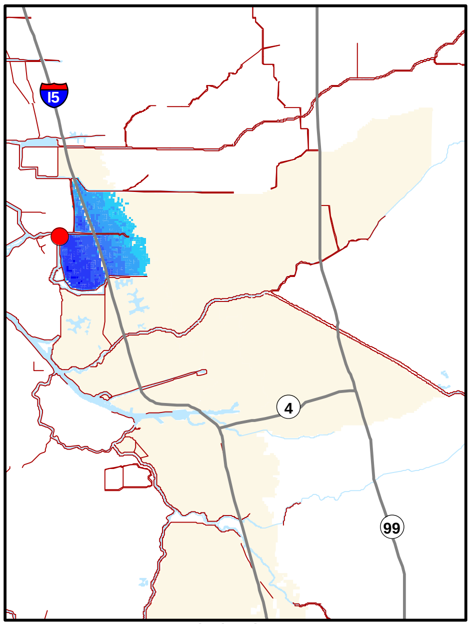
50% (1/2) ACE



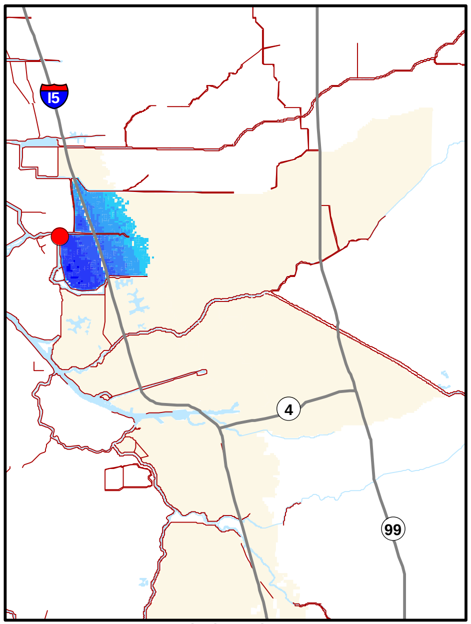
10% (1/10) ACE



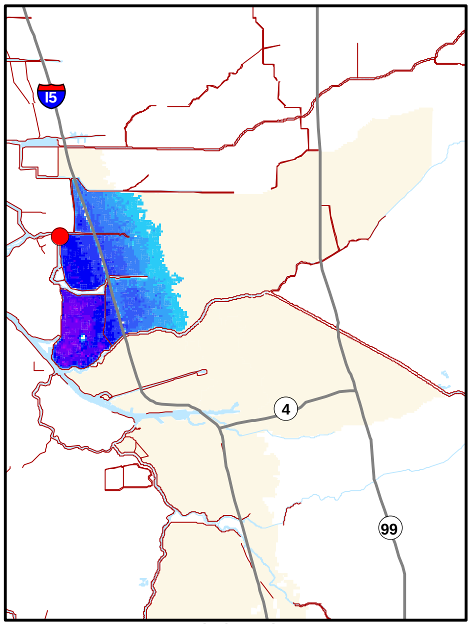
4% (1/25) ACE



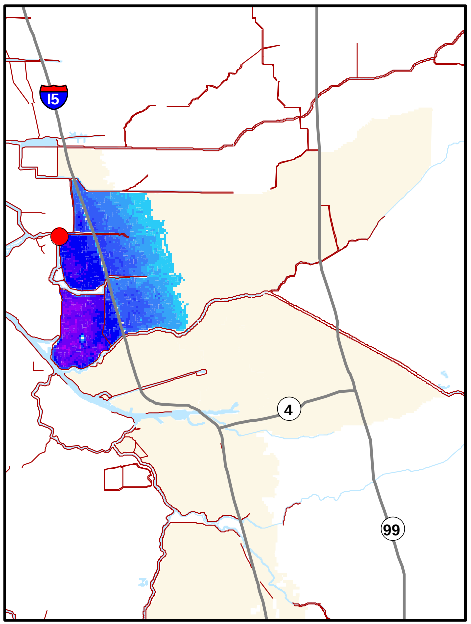
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-D3

Depth (FT)

- 🟦 0.01 - 2.00
- 🟦 2.01 - 4.00
- 🟦 4.01 - 6.00
- 🟦 6.01 - 8.00
- 🟦 8.01 - 10.00
- 🟦 10.01 - 12.00
- 🟪 12.01 - 14.00
- 🟪 14.01 - 16.00
- 🟪 16.01 - 18.00
- 🟪 18.01 - 20.00
- 🟪 20.01 - 22.00
- 🟪 22.01 - 24.00
- — Levees (CLD, NLD)
- ■ LSJ Damage Areas

0 5 Miles

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

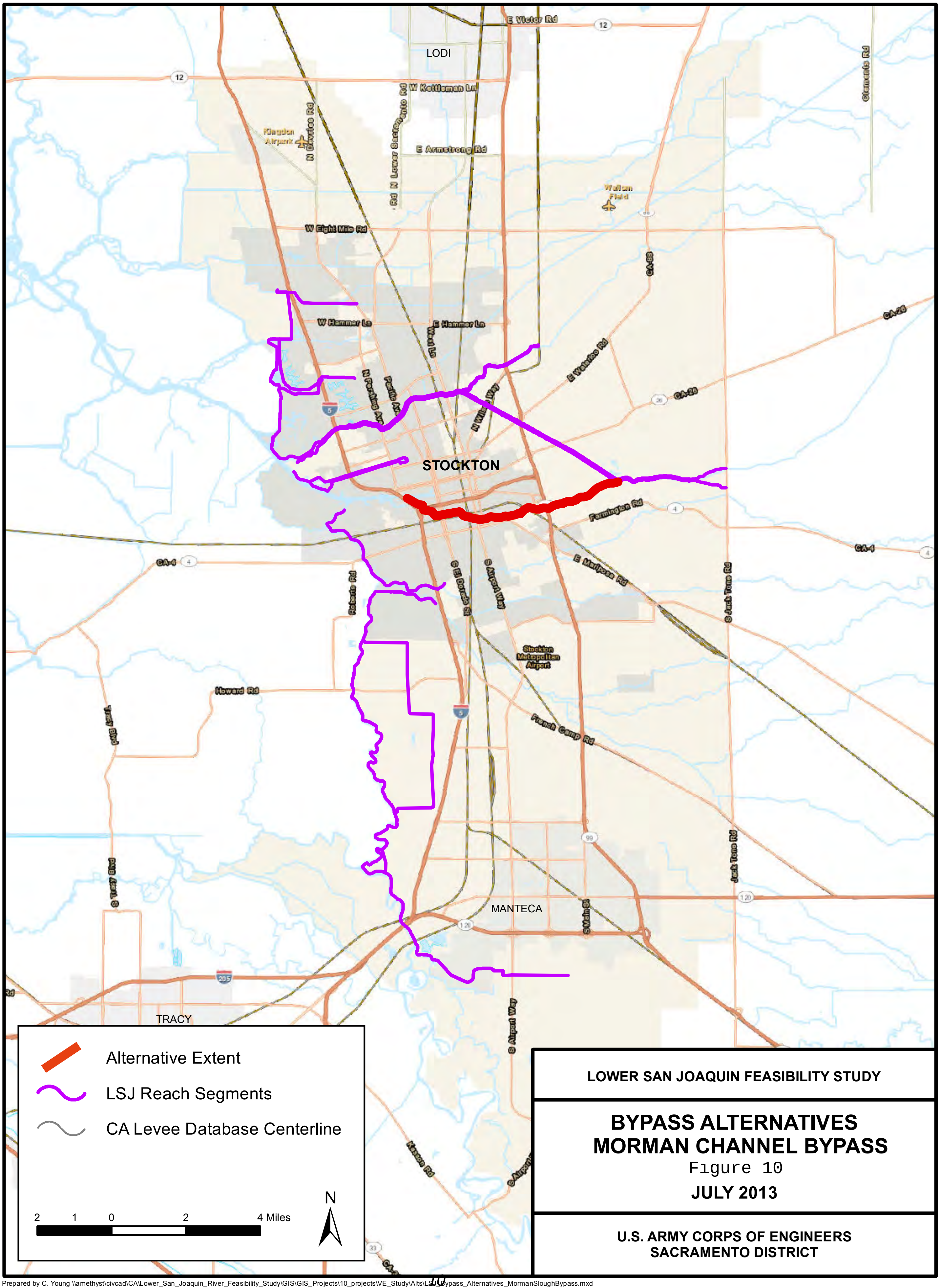
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
WITHOUT PROJECT CONDITION
FIVEMILE SLOUGH
LOCATION B-D3

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

Figure 9



LOWER SAN JOAQUIN FEASIBILITY STUDY

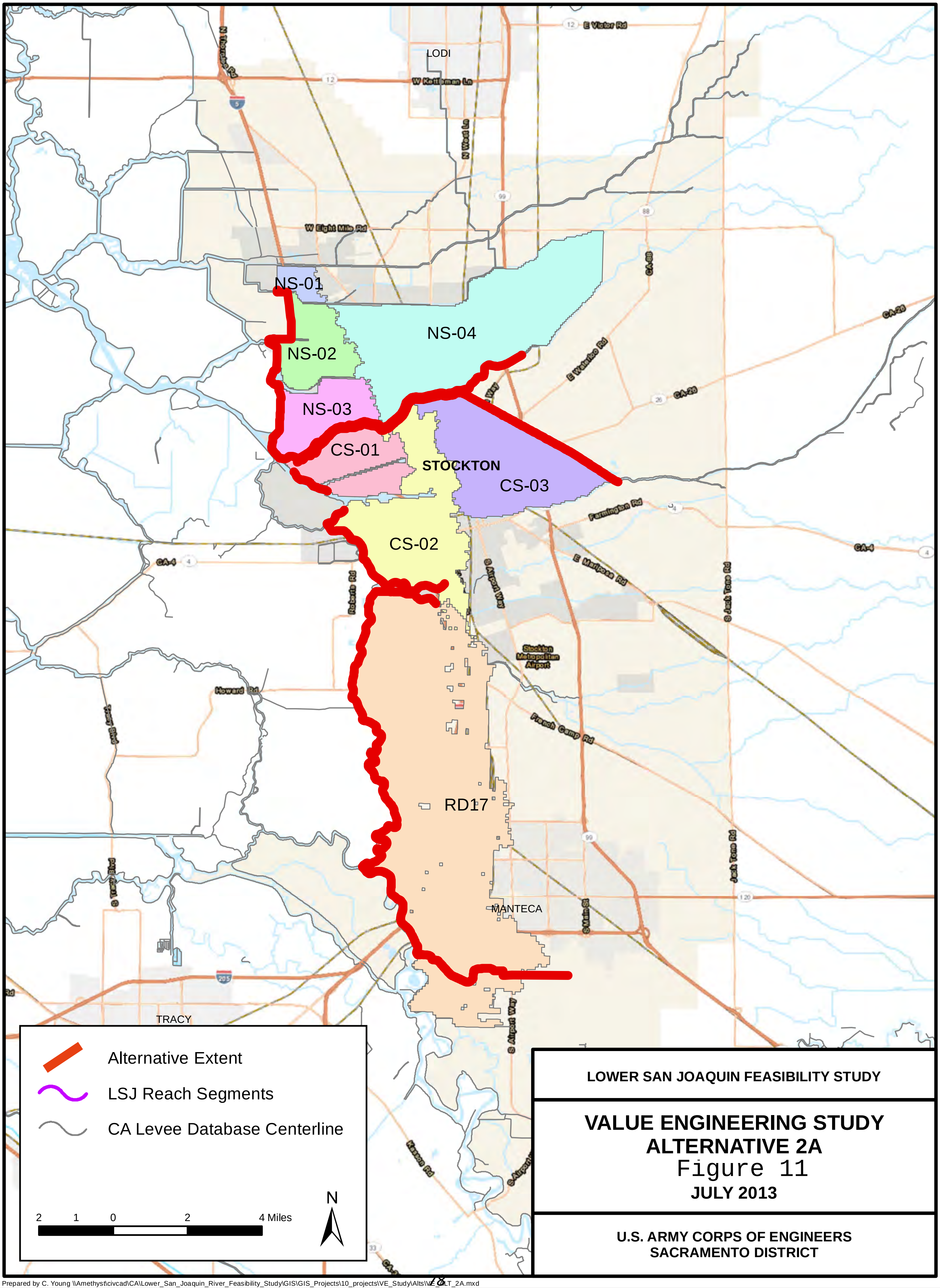
BYPASS ALTERNATIVES
MORMAN CHANNEL BYPASS

Figure 10

JULY 2013

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young \\amethyst\civcad\CA\Lower_San_Joaquin_River_Feasibility_Study\GIS\GIS_Projects\10_projects\VE_Study\Alts\LSJ_Bypass_Alternatives_MormanSloughBypass.mxd



LOWER SAN JOAQUIN FEASIBILITY STUDY

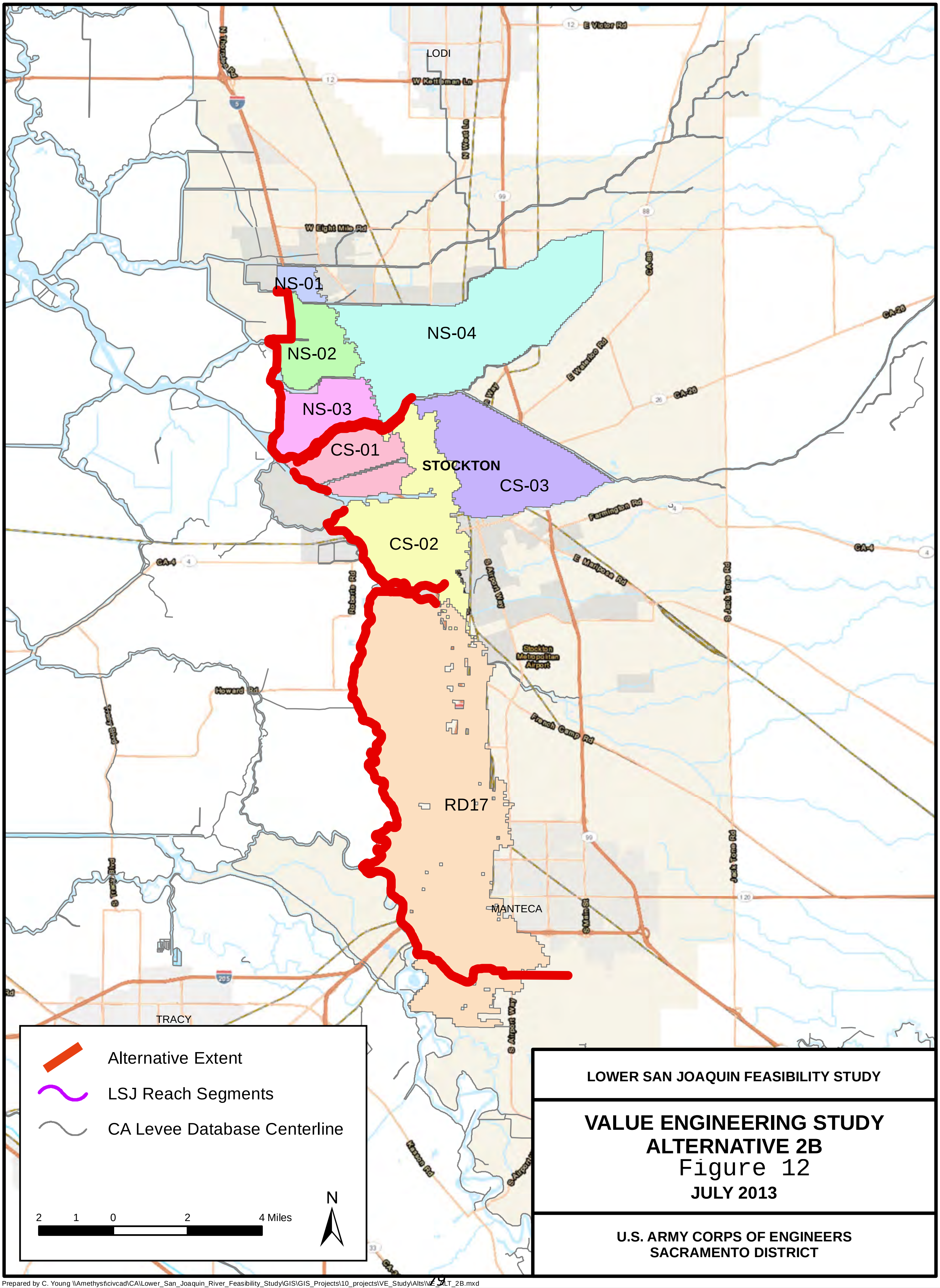
VALUE ENGINEERING STUDY ALTERNATIVE 2A

Figure 11

JULY 2013

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young \\Amethyst\civcad\CA\Lower_San_Joaquin_River_Feasibility_Study\GIS\GIS_Projects\10_projects\VE_Study\Alts\VE_ALT_2A.mxd

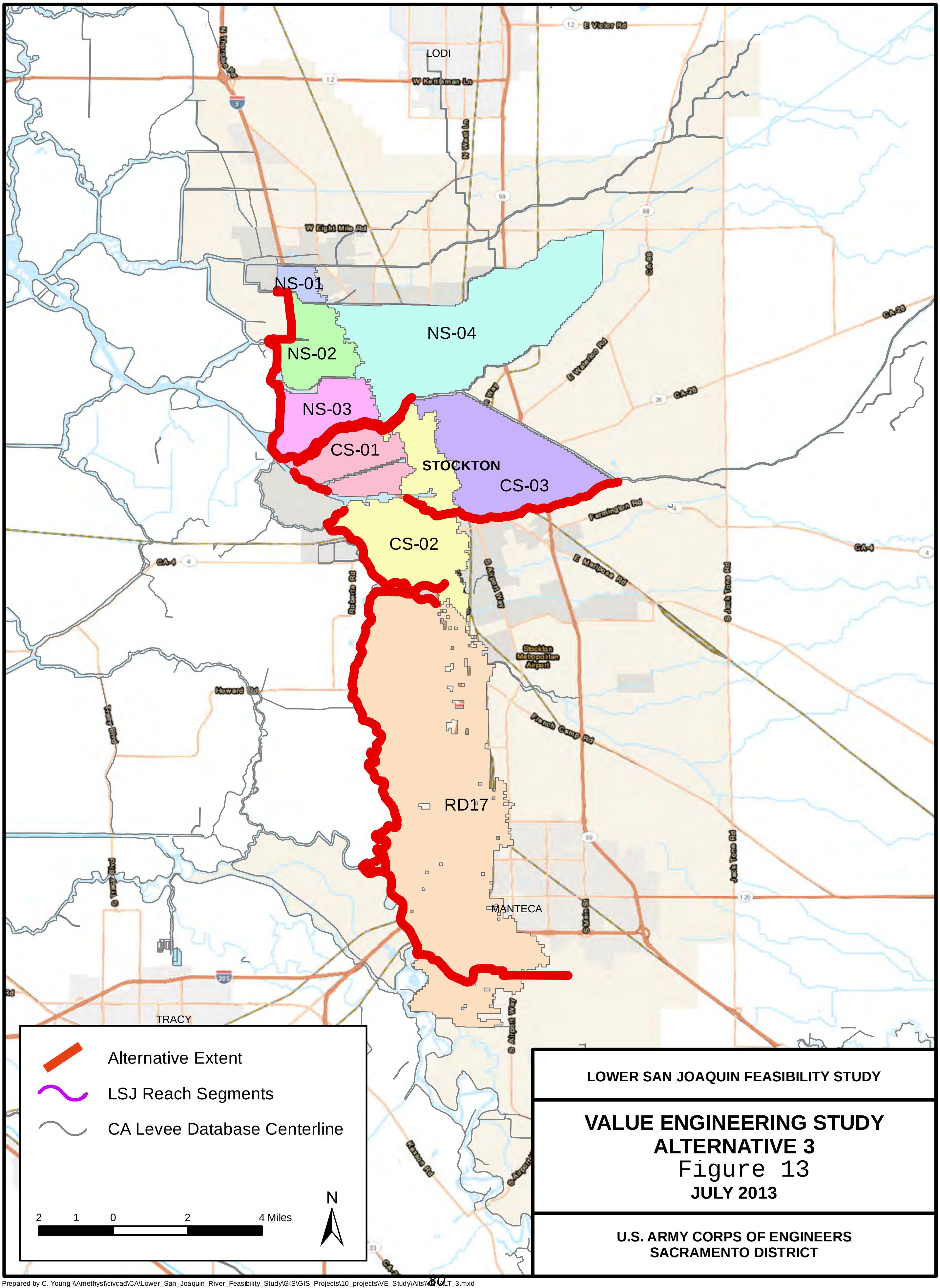


LOWER SAN JOAQUIN FEASIBILITY STUDY

VALUE ENGINEERING STUDY
ALTERNATIVE 2B
Figure 12
JULY 2013

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young \\Amethyst\civcad\CA\Lower_San_Joaquin_River_Feasibility_Study\GIS\GIS_Projects\10_projects\VE_Study\Alts\VE_ALT_2B.mxd

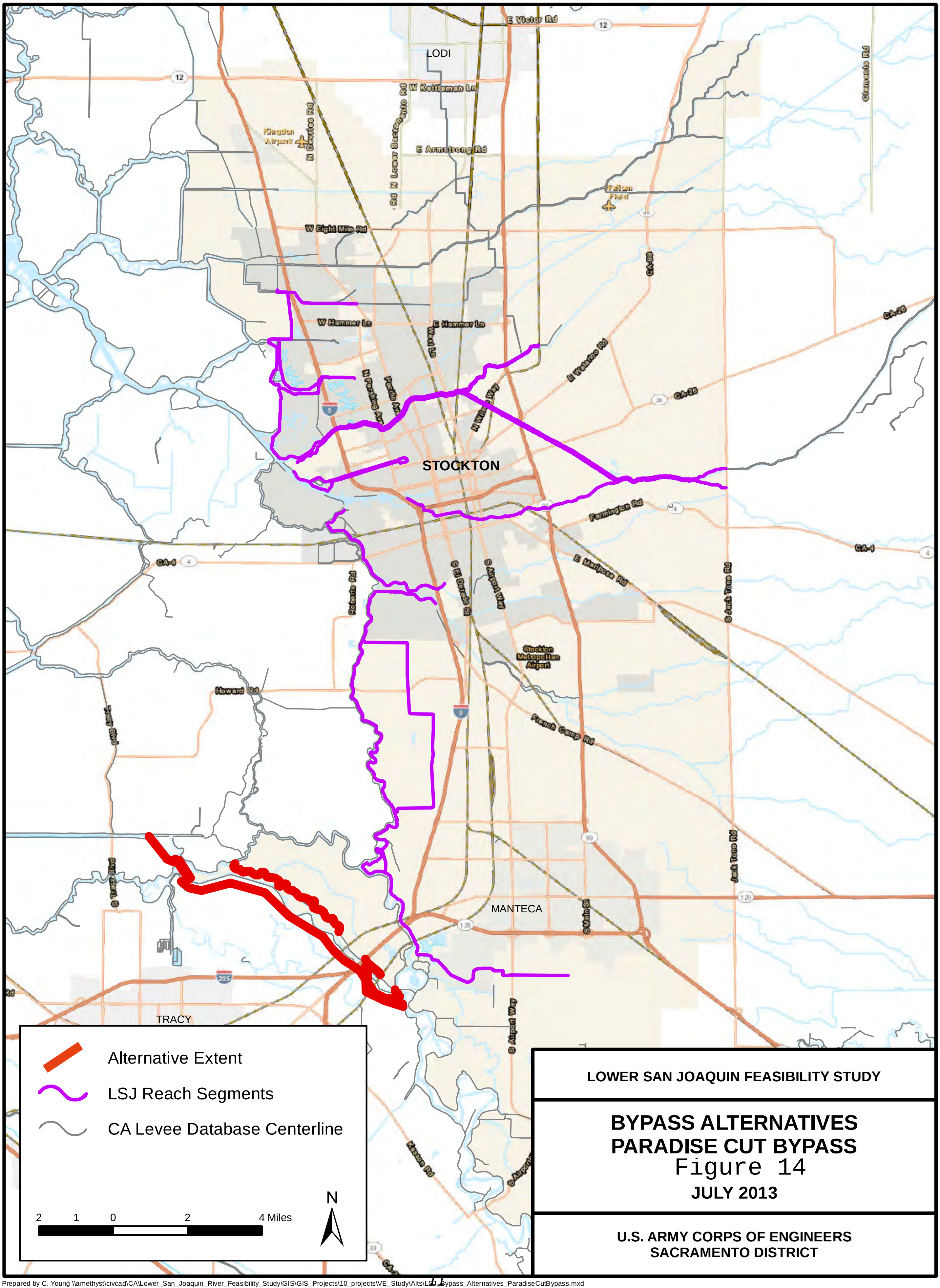


LOWER SAN JOAQUIN FEASIBILITY STUDY

VALUE ENGINEERING STUDY
ALTERNATIVE 3
Figure 13
JULY 2013

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young \\Amethyst\civcad\CA\Lower_San_Joaquin_River_Feasibility_Study\GIS\GIS_Projects\10_projects\VE_Study\Alts\VE_ALT_3.mxd



LOWER SAN JOAQUIN FEASIBILITY STUDY

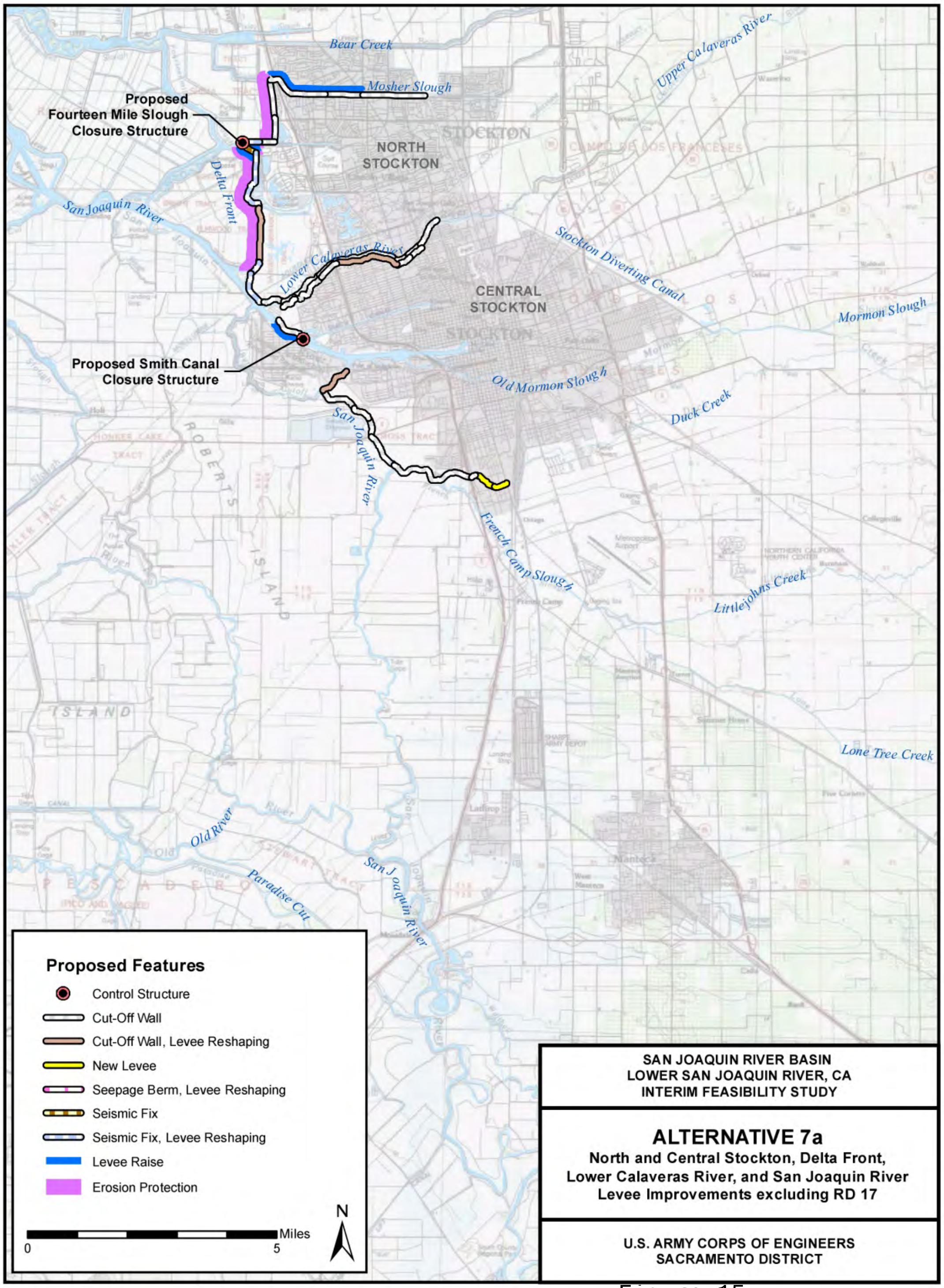
BYPASS ALTERNATIVES
PARADISE CUT BYPASS

Figure 14

JULY 2013

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young \\amethyst\civcad\CA\Lower_San_Joaquin_River_Feasibility_Study\GIS\GIS_Projects\10_projects\VE_Study\Alts\LSJ_Bypass_Alternatives_ParadiseCutBypass.mxd



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

ALTERNATIVE 7a
North and Central Stockton, Delta Front, Lower Calaveras River, and San Joaquin River Levee Improvements excluding RD 17

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Figure 15

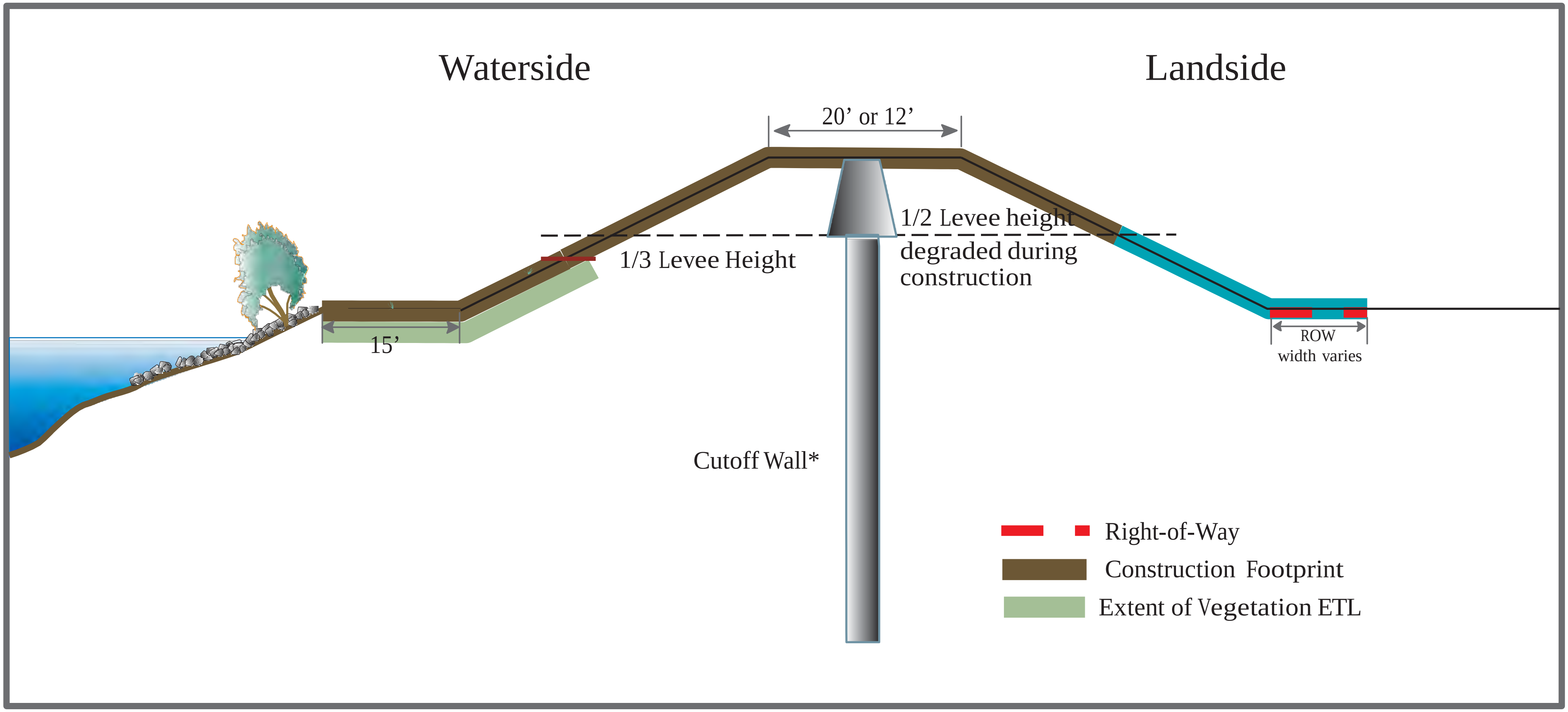


Figure 16. Lower San Joaquin Feasibility Study - Typical Cross Section Repair for the Tentatively Selected Plan

* ~ 22 miles of levee repair would resemble that shown with cutoff wall for the TSP

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 7a, N Stockton, Fix B**
Project Development Stage: **Feasibility (Alternatives)**
Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 325,811,013**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total	
	01 LANDS AND DAMAGES	Real Estate	**$ 79,569,213**		**28.72%**	$ 22,851,571	$ 102,420,783.86	
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 33,941,435**		20.94%	$ 7,106,318	$ 41,047,753.09	
2	**11 01 LEVEES**	**Earthwork**	**$ 57,240,029**		21.04%	$ 12,046,053	$ 69,286,082.12	
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 26,171,400**		20.71%	$ 5,420,188	$ 31,591,588.17	
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ 130,949,900**		25.76%	$ 33,732,220	$ 164,682,120.36	
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 8,213,271**		45.52%	$ 3,739,091	$ 11,952,362.11	
6	**15 FLOODWAY CONTROL AND DIVERSION STRUCTURES**	**Control Structure**	**$ 15,598,000**		26.45%	$ 4,126,008	$ 19,724,007.90	
12		**Remaining Construction Items**	**$ 53,696,978**	*16.5%*	25.82%	$ 13,865,527	$ 67,562,505.50	
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 48,871,652**		28.87%	$ 14,109,845	$ 62,981,496.99	
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 32,581,101**		24.56%	$ 8,003,541	$ 40,584,641.93	*

Totals				
Real Estate	$ 79,569,213	28.72%	$ 22,851,571	$ 102,420,783.86
Total Construction Estimate	$ 325,811,013	24.56%	$ 80,035,406	$ 405,846,419
Total Planning, Engineering & Design	$ 48,871,652	28.87%	$ 14,109,845	$ 62,981,497
Total Construction Management	$ 32,581,101	24.56%	$ 8,003,541	$ 40,584,642
Total	$ 486,832,979	25.08%	$ 125,000,363	$ 611,833,342

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 7a, C Stockton, Fixes B & C plus Duck Cr**
Project Development Stage: **Feasibility (Alternatives)**
Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 108,302,310**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total
	01 LANDS AND DAMAGES	Real Estate	**$ 22,577,987**		**26.45%**	$ 5,972,949	$ 28,550,936.14
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 6,104,019**		23.31%	$ 1,422,752	$ 7,526,770.55
2	**11 01 LEVEES**	**Earthwork**	**$ 38,085,725**		11.28%	$ 4,294,201	$ 42,379,925.95
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 22,525,000**		20.71%	$ 4,665,006	$ 27,190,006.02
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ -**		0.00%	$ -	$ -
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 13,400**		15.94%	$ 2,136	$ 15,535.72
6	**15 FLOODWAY CONTROL AND DIVERSION STRUCTURES**	**Control Structure**	**$ 14,187,000**		26.45%	$ 3,752,768	$ 17,939,767.92
12		**Remaining Construction Items**	**$ 27,387,166**	*25.3%*	25.82%	$ 7,071,860	$ 34,459,026.13
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 16,245,346**		28.87%	$ 4,690,231	$ 20,935,577.11
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 10,830,231**		19.58%	$ 2,120,872	$ 12,951,103.23 *

Totals				
Real Estate	$ 22,577,987	26.45%	$ 5,972,949	$ 28,550,936.14
Total Construction Estimate	$ 108,302,310	19.58%	$ 21,208,723	$ 129,511,032
Total Planning, Engineering & Design	$ 16,245,346	28.87%	$ 4,690,231	$ 20,935,577
Total Construction Management	$ 10,830,231	19.58%	$ 2,120,872	$ 12,951,103
Total	$ 157,955,874	20.70%	$ 33,992,775	$ 191,948,649

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 7b, N Stockton, Fix B**
Project Development Stage: **Feasibility (Alternatives)**
Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 325,798,700**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total
	01 LANDS AND DAMAGES	Real Estate	**$ 78,909,904**		**28.72%**	$ 22,662,267	$ 101,572,170.76
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 33,941,435**		20.94%	$ 7,106,318	$ 41,047,753.09
2	**11 01 LEVEES**	**Earthwork**	**$ 57,240,029**		21.04%	$ 12,046,053	$ 69,286,082.12
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 26,171,400**		20.71%	$ 5,420,188	$ 31,591,588.17
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ 130,949,900**		25.76%	$ 33,732,220	$ 164,682,120.36
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 8,213,271**		45.52%	$ 3,739,091	$ 11,952,362.11
6	**15 FLOODWAY CONTROL AND DIVERSION STRUCTURES**	**Control Structure**	**$ 15,598,000**		26.45%	$ 4,126,008	$ 19,724,007.90
12		**Remaining Construction Items**	**$ 53,684,665**	*16.5%*	25.82%	$ 13,862,348	$ 67,547,013.06
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 48,869,805**		28.87%	$ 14,109,312	$ 62,979,116.80
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 32,579,870**		24.56%	$ 8,003,223	$ 40,583,092.68 *

Totals				
Real Estate	$ 78,909,904	28.72%	$ 22,662,267	$ 101,572,170.76
Total Construction Estimate	$ 325,798,700	24.56%	$ 80,032,227	$ 405,830,927
Total Planning, Engineering & Design	$ 48,869,805	28.87%	$ 14,109,312	$ 62,979,117
Total Construction Management	$ 32,579,870	24.56%	$ 8,003,223	$ 40,583,093
Total	$ 486,158,279	25.08%	$ 124,807,028	$ 610,965,307

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 7b, C Stockton, Fixes B & C**
Project Development Stage: **Feasibility (Alternatives)**
Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 102,187,062**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total
	01 LANDS AND DAMAGES	Real Estate	**$ 21,622,368**		**26.52%**	$ 5,734,874	$ 27,357,242.69
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 5,810,984**		23.31%	$ 1,354,450	$ 7,165,434.02
2	**11 01 LEVEES**	**Earthwork**	**$ 35,034,483**		11.28%	$ 3,950,171	$ 38,984,654.08
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 20,998,600**		20.71%	$ 4,348,883	$ 25,347,483.26
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ -**		0.00%	$ -	$ -
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 13,400**		15.94%	$ 2,136	$ 15,535.72
6	**15 FLOODWAY CONTROL AND DIVERSION STRUCTURES**	**Control Structure**	**$ 14,187,000**		26.45%	$ 3,752,768	$ 17,939,767.92
12		**Remaining Construction Items**	**$ 26,142,595**	*25.6%*	25.82%	$ 6,750,489	$ 32,893,083.54
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 15,328,059**		28.87%	$ 4,425,399	$ 19,753,457.86
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 10,218,706**		19.73%	$ 2,015,890	$ 12,234,595.85 *

Totals				
Real Estate	$ 21,622,368	26.52%	$ 5,734,874	$ 27,357,242.69
Total Construction Estimate	$ 102,187,062	19.73%	$ 20,158,897	$ 122,345,959
Total Planning, Engineering & Design	$ 15,328,059	28.87%	$ 4,425,399	$ 19,753,458
Total Construction Management	$ 10,218,706	19.73%	$ 2,015,890	$ 12,234,596
Total	$ 149,356,195	20.82%	$ 32,335,060	$ 181,691,255

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 7b, RD 17**
Project Development Stage: **Feasibility (Alternatives)**
Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 257,527,099**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total
	01 LANDS AND DAMAGES	Real Estate	**$ 14,442,728**		**23.96%**	$ 3,460,609	$ 17,903,336.54
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 11,298,265**		24.28%	$ 2,742,785	$ 14,041,050.50
2	**11 01 LEVEES**	**Earthwork**	**$ 140,674,376**		20.39%	$ 28,688,582	$ 169,362,958.24
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 43,491,800**		19.52%	$ 8,490,634	$ 51,982,433.97
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ -**		0.00%	$ -	$ -
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 6,946,167**		10.62%	$ 737,954	$ 7,684,120.72
12		**Remaining Construction Items**	**$ 55,116,491**	*21.4%*	7.00%	$ 3,858,154	$ 58,974,645.87
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 38,629,065**		7.00%	$ 2,704,035	$ 41,333,099.47
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 25,752,710**		17.29%	$ 4,451,811	$ 30,204,520.93 *

Totals				
Real Estate	$ 14,442,728	23.96%	$ 3,460,609	$ 17,903,336.54
Total Construction Estimate	$ 257,527,099	17.29%	$ 44,518,110	$ 302,045,209
Total Planning, Engineering & Design	$ 38,629,065	7.00%	$ 2,704,035	$ 41,333,099
Total Construction Management	$ 25,752,710	17.29%	$ 4,451,811	$ 30,204,521
Total	$ 336,351,602	16.05%	$ 55,134,564	$ 391,486,166

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 8a, N Stockton, Fix F**
Project Development Stage: **Feasibility (Alternatives)**
Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 350,564,416**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total	
	01 LANDS AND DAMAGES	Real Estate	**$ 88,700,915**		**28.08%**	$ 24,905,073	$ 113,605,988.01	
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 36,818,361**		20.94%	$ 7,708,661	$ 44,527,021.38	
2	**11 01 LEVEES**	**Earthwork**	**$ 67,277,633**		21.04%	$ 14,158,447	$ 81,436,080.33	
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 30,178,200**		20.71%	$ 6,250,010	$ 36,428,210.42	
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ 130,949,900**		25.76%	$ 33,732,220	$ 164,682,120.36	
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 8,213,271**		45.52%	$ 3,739,091	$ 11,952,362.11	
6	**15 FLOODWAY CONTROL AND DIVERSION STRUCTURES**	**Control Structure**	**$ 15,598,000**		26.45%	$ 4,126,008	$ 19,724,007.90	
12		**Remaining Construction Items**	**$ 61,529,051**	*17.6%*	25.82%	$ 15,887,910	$ 77,416,960.66	
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 52,584,662**		28.87%	$ 15,181,837	$ 67,766,499.08	
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 35,056,442**		24.42%	$ 8,560,235	$ 43,616,676.32	*

Totals

	Contract Cost	% Contingency	$ Contingency	Total
Real Estate	$ 88,700,915	28.08%	$ 24,905,073	$ 113,605,988.01
Total Construction Estimate	$ 350,564,416	24.42%	$ 85,602,347	$ 436,166,763
Total Planning, Engineering & Design	$ 52,584,662	28.87%	$ 15,181,837	$ 67,766,499
Total Construction Management	$ 35,056,442	24.42%	$ 8,560,235	$ 43,616,676
Total	$ 526,906,435	24.95%	$ 134,249,491	$ 661,155,927

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 8a, C Stockton, Fix D + Duck Cr**
Project Development Stage: **Feasibility (Alternatives)**
Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 158,945,400**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total
	01 LANDS AND DAMAGES	Real Estate	**$ 27,465,085**		**25.59%**	$ 7,027,527	$ 34,492,611.99
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 8,381,698**		23.31%	$ 1,953,643	$ 10,335,340.80
2	**11 01 LEVEES**	**Earthwork**	**$ 63,754,473**		13.31%	$ 8,486,525	$ 72,240,998.29
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 32,383,000**		20.71%	$ 6,706,632	$ 39,089,632.19
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ -**		0.00%	$ -	$ -
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 13,400**		15.94%	$ 2,136	$ 15,535.72
6	**15 FLOODWAY CONTROL AND DIVERSION STRUCTURES**	**Control Structure**	**$ 14,187,000**		26.45%	$ 3,752,768	$ 17,939,767.92
12		**Remaining Construction Items**	**$ 40,225,830**	*25.3%*	25.82%	$ 10,387,034	$ 50,612,863.96
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 23,841,810**		28.87%	$ 6,883,423	$ 30,725,232.91
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 15,894,540**		19.69%	$ 3,128,874	$ 19,023,413.89 *

Totals				
Real Estate	$ 27,465,085	25.59%	$ 7,027,527	$ 34,492,611.99
Total Construction Estimate	$ 158,945,400	19.69%	$ 31,288,738	$ 190,234,139
Total Planning, Engineering & Design	$ 23,841,810	28.87%	$ 6,883,423	$ 30,725,233
Total Construction Management	$ 15,894,540	19.69%	$ 3,128,874	$ 19,023,414
Total	$ 226,146,836	20.79%	$ 48,328,562	$ 274,475,398

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 8b, N Stockton, Fix F**
Project Development Stage: **Feasibility (Alternatives)**
Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 350,546,087**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total	
	01 LANDS AND DAMAGES	Real Estate	**$ 87,719,448**		**28.07%**	$ 24,623,520	$ 112,342,968.27	
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 36,818,361**		20.94%	$ 7,708,661	$ 44,527,021.38	
2	**11 01 LEVEES**	**Earthwork**	**$ 67,277,633**		21.04%	$ 14,158,447	$ 81,436,080.33	
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 30,178,200**		20.71%	$ 6,250,010	$ 36,428,210.42	
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ 130,949,900**		25.76%	$ 33,732,220	$ 164,682,120.36	
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 8,213,271**		45.52%	$ 3,739,091	$ 11,952,362.11	
6	**15 FLOODWAY CONTROL AND DIVERSION STRUCTURES**	**Control Structure**	**$ 15,598,000**		26.45%	$ 4,126,008	$ 19,724,007.90	
12		**Remaining Construction Items**	**$ 61,510,722**	*17.5%*	25.82%	$ 15,883,177	$ 77,393,898.78	
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 52,581,913**		28.87%	$ 15,181,043	$ 67,762,955.96	
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 35,054,609**		24.42%	$ 8,559,761	$ 43,614,370.13	*

Totals				
Real Estate	$ 87,719,448	28.07%	$ 24,623,520	$ 112,342,968.27
Total Construction Estimate	$ 350,546,087	24.42%	$ 85,597,614	$ 436,143,701
Total Planning, Engineering & Design	$ 52,581,913	28.87%	$ 15,181,043	$ 67,762,956
Total Construction Management	$ 35,054,609	24.42%	$ 8,559,761	$ 43,614,370
Total	$ 525,902,057	24.95%	$ 133,961,938	$ 659,863,996

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 8b, C Stockton, Fix D**
Project Development Stage: **Feasibility (Alternatives)**
Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 152,543,543**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total	
	01 LANDS AND DAMAGES	Real Estate	**$ 26,149,697**		**25.57%**	$ 6,686,584	$ 32,836,280.57	
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 8,119,996**		23.31%	$ 1,892,645	$ 10,012,640.71	
2	**11 01 LEVEES**	**Earthwork**	**$ 60,707,783**		11.28%	$ 6,844,860	$ 67,552,642.38	
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 30,856,600**		20.71%	$ 6,390,509	$ 37,247,109.43	
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ -**		0.00%	$ -	$ -	
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 13,400**		15.94%	$ 2,136	$ 15,535.72	
6	**15 FLOODWAY CONTROL AND DIVERSION STRUCTURES**	**Control Structure**	**$ 14,187,000**		26.45%	$ 3,752,768	$ 17,939,767.92	
12		**Remaining Construction Items**	**$ 38,658,764**	*25.3%*	25.82%	$ 9,982,390	$ 48,641,153.40	
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 22,881,531**		28.87%	$ 6,606,179	$ 29,487,709.99	
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 15,254,354**		18.92%	$ 2,886,531	$ 18,140,884.95	*

Totals				
Real Estate	$ 26,149,697	25.57%	$ 6,686,584	$ 32,836,280.57
Total Construction Estimate	$ 152,543,543	18.92%	$ 28,865,307	$ 181,408,850
Total Planning, Engineering & Design	$ 22,881,531	28.87%	$ 6,606,179	$ 29,487,710
Total Construction Management	$ 15,254,354	18.92%	$ 2,886,531	$ 18,140,885
Total	$ 216,829,125	20.12%	$ 45,044,600	$ 261,873,725

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 8b, RD 17**
Project Development Stage: **Feasibility (Alternatives)**
Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 257,536,663**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total	
	01 LANDS AND DAMAGES	Real Estate	**$ 14,954,855**		**24.05%**	$ 3,596,296	$ 18,551,151.16	
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 11,298,265**		24.28%	$ 2,742,785	$ 14,041,050.50	
2	**11 01 LEVEES**	**Earthwork**	**$ 140,674,376**		20.39%	$ 28,688,582	$ 169,362,958.24	
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 43,491,800**		19.52%	$ 8,490,634	$ 51,982,433.97	
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ -**		0.00%	$ -	$ -	
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 6,946,167**		10.62%	$ 737,954	$ 7,684,120.72	
12		**Remaining Construction Items**	**$ 55,126,055**	*21.4%*	7.00%	$ 3,858,824	$ 58,984,879.35	
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 38,630,500**		7.00%	$ 2,704,135	$ 41,334,634.49	
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 25,753,666**		17.29%	$ 4,451,878	$ 30,205,544.28	*

Totals				
Real Estate	$ 14,954,855	24.05%	$ 3,596,296	$ 18,551,151.16
Total Construction Estimate	$ 257,536,663	17.29%	$ 44,518,779	$ 302,055,443
Total Planning, Engineering & Design	$ 38,630,500	7.00%	$ 2,704,135	$ 41,334,634
Total Construction Management	$ 25,753,666	17.29%	$ 4,451,878	$ 30,205,544
Total	$ 336,875,684	16.05%	$ 55,271,088	$ 392,146,773

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 9a, N Stockton, Fix B**
Project Development Stage: **Feasibility (Alternatives)**
Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 325,813,366**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total
	01 LANDS AND DAMAGES	Real Estate	**$ 79,695,198**		**28.71%**	$ 22,883,002	$ 102,578,200.44
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 33,941,435**		20.94%	$ 7,106,318	$ 41,047,753.09
2	**11 01 LEVEES**	**Earthwork**	**$ 57,240,029**		21.04%	$ 12,046,053	$ 69,286,082.12
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 26,171,400**		20.71%	$ 5,420,188	$ 31,591,588.17
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ 130,949,900**		25.76%	$ 33,732,220	$ 164,682,120.36
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 8,213,271**		45.52%	$ 3,739,091	$ 11,952,362.11
6	**15 FLOODWAY CONTROL AND DIVERSION STRUCTURES**	**Control Structure**	**$ 15,598,000**		26.45%	$ 4,126,008	$ 19,724,007.90
12		**Remaining Construction Items**	**$ 53,699,331**	*16.5%*	25.82%	$ 13,866,135	$ 67,565,466.08
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 48,872,005**		28.87%	$ 14,109,947	$ 62,981,951.84
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 32,581,337**		24.56%	$ 8,003,601	$ 40,584,937.98 *

Totals				
Real Estate	$ 79,695,198	28.71%	$ 22,883,002	$ 102,578,200.44
Total Construction Estimate	$ 325,813,366	24.56%	$ 80,036,014	$ 405,849,380
Total Planning, Engineering & Design	$ 48,872,005	28.87%	$ 14,109,947	$ 62,981,952
Total Construction Management	$ 32,581,337	24.56%	$ 8,003,601	$ 40,584,938
Total	$ 486,961,906	25.08%	$ 125,032,564	$ 611,994,470

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 9a, C Stockton, Fixes B & C plus Duck Creek & Mormon Channel**

Project Development Stage: **Feasibility (Alternatives)**

Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 124,760,655**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total	
	01 LANDS AND DAMAGES	Real Estate	**$ 25,485,082**		**25.14%**	$ 6,407,596	$ 31,892,677.56	
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 7,028,675**		23.31%	$ 1,638,275	$ 8,666,949.85	
2	**11 01 LEVEES**	**Earthwork**	**$ 40,284,842**		11.28%	$ 4,542,154	$ 44,826,995.47	
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 22,525,000**		20.71%	$ 4,665,006	$ 27,190,006.02	
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ -**		0.00%	$ -	$ -	
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 13,400**		15.94%	$ 2,136	$ 15,535.72	
6	**15 FLOODWAY CONTROL AND DIVERSION STRUCTURES**	**Control Structures**	**$ 24,370,000**		30.93%	$ 7,538,473	$ 31,908,472.66	
12		**Remaining Construction Items**	**$ 30,538,738**	*24.5%*	25.82%	$ 7,885,652	$ 38,424,390.41	
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 18,714,098**		28.87%	$ 5,402,990	$ 24,117,087.75	
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 12,476,065**		21.06%	$ 2,627,170	$ 15,103,235.01	*

Totals

	Contract Cost	% Contingency	$ Contingency	Total
Real Estate	$ 25,485,082	25.14%	$ 6,407,596	$ 31,892,677.56
Total Construction Estimate	$ 124,760,655	21.06%	$ 26,271,695	$ 151,032,350
Total Planning, Engineering & Design	$ 18,714,098	28.87%	$ 5,402,990	$ 24,117,088
Total Construction Management	$ 12,476,065	21.06%	$ 2,627,170	$ 15,103,235
Total	$ 181,435,900	22.00%	$ 40,709,451	$ 222,145,350

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 9b, N Stockton, Fix B**
Project Development Stage: **Feasibility (Alternatives)**
Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 325,798,986**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total	
	01 LANDS AND DAMAGES	Real Estate	**$ 78,925,237**		**28.71%**	$ 22,663,033	$ 101,588,270.59	
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 33,941,435**		20.94%	$ 7,106,318	$ 41,047,753.09	
2	**11 01 LEVEES**	**Earthwork**	**$ 57,240,029**		21.04%	$ 12,046,053	$ 69,286,082.12	
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 26,171,400**		20.71%	$ 5,420,188	$ 31,591,588.17	
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ 130,949,900**		25.76%	$ 33,732,220	$ 164,682,120.36	
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 8,213,271**		45.52%	$ 3,739,091	$ 11,952,362.11	
6	**15 FLOODWAY CONTROL AND DIVERSION STRUCTURES**	**Control Structure**	**$ 15,598,000**		26.45%	$ 4,126,008	$ 19,724,007.90	
12		**Remaining Construction Items**	**$ 53,684,951**	*16.5%*	25.82%	$ 13,862,422	$ 67,547,372.91	
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 48,869,848**		28.87%	$ 14,109,324	$ 62,979,172.09	
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 32,579,899**		24.56%	$ 8,003,230	$ 40,583,128.67	*

Totals				
Real Estate	$ 78,925,237	28.71%	$ 22,663,033	$ 101,588,270.59
Total Construction Estimate	$ 325,798,986	24.56%	$ 80,032,301	$ 405,831,287
Total Planning, Engineering & Design	$ 48,869,848	28.87%	$ 14,109,324	$ 62,979,172
Total Construction Management	$ 32,579,899	24.56%	$ 8,003,230	$ 40,583,129
Total	$ 486,173,970	25.08%	$ 124,807,888	$ 610,981,858

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 9b, C Stockton, Fixes B & C plus Mormon Channel**

Project Development Stage: **Feasibility (Alternatives)**

Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 118,334,371**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total
	01 LANDS AND DAMAGES	Real Estate	**$ 24,468,872**		**25.15%**	$ 6,152,924	$ 30,621,796.06
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 6,736,134**		23.31%	$ 1,570,088	$ 8,306,222.06
2	**11 01 LEVEES**	**Earthwork**	**$ 37,239,483**		11.28%	$ 4,198,787	$ 41,438,269.92
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 20,998,600**		20.71%	$ 4,348,883	$ 25,347,483.26
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ -**		0.00%	$ -	$ -
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 13,400**		15.94%	$ 2,136	$ 15,535.72
6	**15 FLOODWAY CONTROL AND DIVERSION STRUCTURES**	**Control Structures**	**$ 24,370,000**		30.93%	$ 7,538,473	$ 31,908,472.66
12		**Remaining Construction Items**	**$ 28,976,754**	*24.5%*	25.82%	$ 7,482,320	$ 36,459,073.47
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 17,750,156**		28.87%	$ 5,124,688	$ 22,874,843.12
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 11,833,437**		21.25%	$ 2,514,069	$ 14,347,505.71 *

Totals	Contract Cost	% Contingency	$ Contingency	Total
Real Estate	$ 24,468,872	25.15%	$ 6,152,924	$ 30,621,796.06
Total Construction Estimate	$ 118,334,371	21.25%	$ 25,140,686	$ 143,475,057
Total Planning, Engineering & Design	$ 17,750,156	28.87%	$ 5,124,688	$ 22,874,843
Total Construction Management	$ 11,833,437	21.25%	$ 2,514,069	$ 14,347,506
Total	$ 172,386,836	22.16%	$ 38,932,366	$ 211,319,202

Abbreviated Risk Analysis

Project (less than $40M): **Lower San Joaquin River Feasibility Study, Alt 9b, RD 17**
Project Development Stage: **Feasibility (Alternatives)**
Risk Category: **Moderate Risk: Typical Project or Possible Life Safety**

Total Construction Contract Cost = **$ 257,527,888**

	CWWBS	Feature of Work	Contract Cost		% Contingency	$ Contingency	Total
	01 LANDS AND DAMAGES	Real Estate	**$ 14,484,970**		**23.91%**	$ 3,462,721	$ 17,947,691.35
1	**06 FISH AND WILDLIFE FACILITIES**	**Fish & Wildlife Facilities**	**$ 11,298,265**		24.28%	$ 2,742,785	$ 14,041,050.50
2	**11 01 LEVEES**	**Earthwork**	**$ 140,674,376**		20.39%	$ 28,688,582	$ 169,362,958.24
3	**11 01 LEVEES**	**Cutoff Walls**	**$ 43,491,800**		19.52%	$ 8,490,634	$ 51,982,433.97
4	**11 01 LEVEES**	**DSM (Seismic)**	**$ -**		0.00%	$ -	$ -
5	**11 01 LEVEES**	**Slope/Erosion Protection**	**$ 6,946,167**		10.62%	$ 737,954	$ 7,684,120.72
12		**Remaining Construction Items**	**$ 55,117,280**	*21.4%*	7.00%	$ 3,858,210	$ 58,975,490.10
13	30 PLANNING, ENGINEERING, AND DESIGN	**Planning, Engineering, & Design**	**$ 38,629,183**		7.00%	$ 2,704,043	$ 41,333,226.10
14	31 CONSTRUCTION MANAGEMENT	**Construction Management**	**$ 25,752,789**		17.29%	$ 4,451,817	$ 30,204,605.35 *

Totals				
Real Estate	$ 14,484,970	23.91%	$ 3,462,721	$ 17,947,691.35
Total Construction Estimate	$ 257,527,888	17.29%	$ 44,518,165	$ 302,046,054
Total Planning, Engineering & Design	$ 38,629,183	7.00%	$ 2,704,043	$ 41,333,226
Total Construction Management	$ 25,752,789	17.29%	$ 4,451,817	$ 30,204,605
Total	$ 336,394,831	16.05%	$ 55,136,746	$ 391,531,576

**** TOTAL PROJECT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-7A
PROJECT NO: 105785
LOCATION: Stockton CA

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

Program Year (Budget EC): 2015
Effective Price Level Date: 1 OCT 14

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
WBS NUMBER *A*	Civil Works Feature & Sub-Feature Description *B*	COST ($K) *C*	CNTG ($K) *D*	CNTG (%) *E*	TOTAL ($K) *F*	ESC (%) *G*	COST ($K) *H*	CNTG ($K) *I*	**TOTAL ($K)** *J*	Spent Thru: 1-Oct-14 ($K) *K*	*L*	COST ($K) *M*	CNTG ($K) *N*	FULL ($K) *O*
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE	$0	$0	24%	$0	-	$0	$0	$0	$0		$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$40,045	$9,775	24%	$49,820	0.0%	$40,045	$9,775	$49,820	$0		$40,045	$9,775	$49,820
11	LEVEES & FLOODWALLS	$335,898	$80,860	24%	$416,758	0.0%	$335,898	$80,860	$416,758	$0		$335,898	$80,860	$416,758
15	FLOODWAY CONTROL & DIVERSION STRU	$29,785	$6,846	23%	$36,631	0.0%	$29,785	$6,846	$36,631	$0		$29,785	$6,846	$36,631
18	CULTURAL RESOURCE PRESERVATION	$11,767	$2,824	24%	$14,592	0.0%	$11,767	$2,824	$14,592	$0		$11,767	$2,824	$14,592
	CONSTRUCTION ESTIMATE TOTALS:	$417,496	$100,305		$517,801	0.0%	$417,496	$100,305	$517,801	$0		$417,496	$100,305	$517,801
01	LANDS AND DAMAGES	$102,147	$28,824	28%	$130,971	0.0%	$102,147	$28,824	$130,971	$0		$102,147	$28,824	$130,971
02	RELOCATIONS	$16,618	$3,805	23%	$20,423	0.0%	$16,618	$3,805	$20,423	$0		$16,618	$3,805	$20,423
30	RELOCATIIONS - PED	$2,493	$571	23%	$3,063	0.0%	$2,493	$571	$3,063	$0		$2,493	$571	$3,063
31	RELOCATIONS - CM	$1,662	$381	23%	$2,042	0.0%	$1,662	$381	$2,042	$0		$1,662	$381	$2,042
30	PLANNING, ENGINEERING & DESIGN	$62,624	$15,046	24%	$77,670	0.0%	$62,624	$15,046	$77,670	$0		$62,624	$15,046	$77,670
31	CONSTRUCTION MANAGEMENT	$41,749	$10,030	24%	$51,779	0.0%	$41,749	$10,030	$51,779	$0		$41,749	$10,030	$51,779
	PROJECT COST TOTALS:	$644,788	$158,962	25%	$803,750		$644,788	$158,962	**$803,750**	$0		$644,788	$158,962	$803,750

CHIEF, COST ENGINEERING, Jeremiah Frost

PROJECT MANAGER, Joana Savinon

CHIEF, REAL ESTATE, Sharon Caine

CHIEF, ENGINEERING, Rick Poeppelman

ESTIMATED TOTAL PROJECT COST: **$803,750,000**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-7A
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
				RISK BASED										
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	N Stockton, Fix B													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE)		$0	25%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$33,941	$8,513	25%	$42,454	0.0%	$33,941	$8,513	$42,454	2014Q1	0.0%	$33,941	$8,513	$42,454
11	LEVEES & FLOODWALLS	$259,009	$64,959	25%	$323,969	0.0%	$259,009	$64,959	$323,969	2014Q1	0.0%	$259,009	$64,959	$323,969
15	FLOODWAY CONTROL & DIVERSION STRU	$15,598	$3,912	25%	$19,510	0.0%	$15,598	$3,912	$19,510	2014Q1	0.0%	$15,598	$3,912	$19,510
18	CULTURAL RESOURCE PRESERVATION	$8,885	$2,228	25%	$11,113	0.0%	$8,885	$2,228	$11,113	2014Q1	0.0%	$8,885	$2,228	$11,113
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$317,433	$79,612	25%	**$397,045**		$317,433	$79,612	$397,045			$317,433	$79,612	$397,045
01	LANDS AND DAMAGES	$79,569	$22,852	29%	$102,421	0.0%	$79,569	$22,852	$102,421	2014Q1	0.0%	$79,569	$22,852	$102,421
02	RELOCATIONS	$8,378	$2,101	25%	$10,479	0.0%	$8,378	$2,101	$10,479	2014Q1	0.0%	$8,378	$2,101	$10,479
30	RELOCATIIONS - PED	$1,257	$315	25%	$1,572	0.0%	$1,257	$315	$1,572	2014Q1	0.0%	$1,257	$315	$1,572
31	RELOCATIONS - CM	$838	$210	25%	$1,048	0.0%	$838	$210	$1,048	2014Q1	0.0%	$838	$210	$1,048
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
1.0%	Planning & Environmental Compliance	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
6.0%	Engineering & Design	$19,046	$4,777	25%	$23,823	0.0%	$19,046	$4,777	$23,823	2014Q1	0.0%	$19,046	$4,777	$23,823
0.5%	Reviews, ATRs, IEPRs, VE	$1,587	$398	25%	$1,985	0.0%	$1,587	$398	$1,985	2014Q1	0.0%	$1,587	$398	$1,985
0.5%	Life Cycle Updates (cost, schedule, risks)	$1,587	$398	25%	$1,985	0.0%	$1,587	$398	$1,985	2014Q1	0.0%	$1,587	$398	$1,985
1.0%	Contracting & Reprographics	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
3.0%	Engineering During Construction	$9,523	$2,388	25%	$11,911	0.0%	$9,523	$2,388	$11,911	2014Q1	0.0%	$9,523	$2,388	$11,911
1.0%	Planning During Construction	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
1.0%	Project Operations	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$20,633	$5,175	25%	$25,808	0.0%	$20,633	$5,175	$25,808	2014Q1	0.0%	$20,633	$5,175	$25,808
1.5%	Project Operation:	$4,761	$1,194	25%	$5,955	0.0%	$4,761	$1,194	$5,955	2014Q1	0.0%	$4,761	$1,194	$5,955
2.0%	Project Management	$6,349	$1,592	25%	$7,941	0.0%	$6,349	$1,592	$7,941	2014Q1	0.0%	$6,349	$1,592	$7,941
	CONTRACT COST TOTALS:	$486,831	$124,993		**$611,824**		$486,831	$124,993	$611,824			$486,831	$124,993	**$611,824**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-7A
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	C Stockton, Fixes B & C													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE)		$0	21%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$6,104	$1,262	21%	$7,366	0.0%	$6,104	$1,262	$7,366	2014Q1	0.0%	$6,104	$1,262	$7,366
11	LEVEES & FLOODWALLS	$76,889	$15,901	21%	$92,789	0.0%	$76,889	$15,901	$92,789	2014Q1	0.0%	$76,889	$15,901	$92,789
15	FLOODWAY CONTROL & DIVERSION STRU	$14,187	$2,934	21%	$17,121	0.0%	$14,187	$2,934	$17,121	2014Q1	0.0%	$14,187	$2,934	$17,121
18	CULTURAL RESOURCE PRESERVATION	$2,883	$596	21%	$3,479	0.0%	$2,883	$596	$3,479	2014Q1	0.0%	$2,883	$596	$3,479
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$100,063	$20,693	21%	**$120,755**		$100,063	$20,693	$120,755			$100,063	$20,693	$120,755
01	LANDS AND DAMAGES	$22,578	$5,972	26%	$28,550	0.0%	$22,578	$5,972	$28,550	2014Q1	0.0%	$22,578	$5,972	$28,550
02	RELOCATIONS	$8,240	$1,704	21%	$9,944	0.0%	$8,240	$1,704	$9,944	2014Q1	0.0%	$8,240	$1,704	$9,944
30	RELOCATIIONS - PED	$1,236	$256	21%	$1,492	0.0%	$1,236	$256	$1,492	2014Q1	0.0%	$1,236	$256	$1,492
31	RELOCATIONS - CM	$824	$170	21%	$994	0.0%	$824	$170	$994	2014Q1	0.0%	$824	$170	$994
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$1,001	$207	21%	$1,208	0.0%	$1,001	$207	$1,208	2014Q1	0.0%	$1,001	$207	$1,208
1.0%	Planning & Environmental Compliance	$1,001	$207	21%	$1,208	0.0%	$1,001	$207	$1,208	2014Q1	0.0%	$1,001	$207	$1,208
6.0%	Engineering & Design	$6,004	$1,242	21%	$7,246	0.0%	$6,004	$1,242	$7,246	2014Q1	0.0%	$6,004	$1,242	$7,246
0.5%	Reviews, ATRs, IEPRs, VE	$500	$103	21%	$603	0.0%	$500	$103	$603	2014Q1	0.0%	$500	$103	$603
0.5%	Life Cycle Updates (cost, schedule, risks)	$500	$103	21%	$603	0.0%	$500	$103	$603	2014Q1	0.0%	$500	$103	$603
1.0%	Contracting & Reprographics	$1,001	$207	21%	$1,208	0.0%	$1,001	$207	$1,208	2014Q1	0.0%	$1,001	$207	$1,208
3.0%	Engineering During Construction	$3,002	$621	21%	$3,623	0.0%	$3,002	$621	$3,623	2014Q1	0.0%	$3,002	$621	$3,623
1.0%	Planning During Construction	$1,001	$207	21%	$1,208	0.0%	$1,001	$207	$1,208	2014Q1	0.0%	$1,001	$207	$1,208
1.0%	Project Operations	$1,001	$207	21%	$1,208	0.0%	$1,001	$207	$1,208	2014Q1	0.0%	$1,001	$207	$1,208
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$6,504	$1,345	21%	$7,849	0.0%	$6,504	$1,345	$7,849	2014Q1	0.0%	$6,504	$1,345	$7,849
1.5%	Project Operation:	$1,501	$310	21%	$1,811	0.0%	$1,501	$310	$1,811	2014Q1	0.0%	$1,501	$310	$1,811
2.0%	Project Management	$2,001	$414	21%	$2,415	0.0%	$2,001	$414	$2,415	2014Q1	0.0%	$2,001	$414	$2,415
	CONTRACT COST TOTALS:	$157,957	$33,968		**$191,926**		$157,957	$33,968	$191,926			$157,957	$33,968	**$191,926**

**** TOTAL PROJECT COST SUMMARY ****

Printed:6/12/2014
Page 1 of 4

PROJECT: Lower San Joaquin River Feas Study - Alt LS-7B
PROJECT NO: 105785
LOCATION: Stockton CA

DISTRICT: SPD South Pacific Division
PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

Civil Works Work Breakdown Structure		ESTIMATED COST (in $1000s)				PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)				TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)				
						Program Year (Budget EC): 2015 Effective Price Level Date: 1 OCT 14								
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	**TOTAL ($K)**	Spent Thru: 1-Oct-14 ($K)		COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*K*	*L*	*M*	*N*	*O*
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE	$0	$0	21%	$0	-	$0	$0	$0	$0		$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$51,051	$11,535	23%	$62,586	0.0%	$51,051	$11,535	$62,586	$0		$51,051	$11,535	$62,586
11	LEVEES & FLOODWALLS	$560,809	$116,789	21%	$677,598	0.0%	$560,809	$116,789	$677,598	$0		$560,809	$116,789	$677,598
15	FLOODWAY CONTROL & DIVERSION STRU	$29,785	$6,864	23%	$36,649	0.0%	$29,785	$6,864	$36,649	$0		$29,785	$6,864	$36,649
18	CULTURAL RESOURCE PRESERVATION	$17,737	$3,778	21%	$21,514	0.0%	$17,737	$3,778	$21,514	$0		$17,737	$3,778	$21,514
	CONSTRUCTION ESTIMATE TOTALS:	$659,382	$138,966		$798,347	0.0%	$659,382	$138,966	$798,347	$0		$659,382	$138,966	$798,347
01	LANDS AND DAMAGES	$114,975	$31,858	28%	$146,833	0.0%	$114,975	$31,858	$146,833	$0		$114,975	$31,858	$146,833
02	RELOCATIONS	$26,131	$5,343	20%	$31,474	0.0%	$26,131	$5,343	$31,474	$0		$26,131	$5,343	$31,474
30	RELOCATIIONS - PED	$3,920	$801	20%	$4,721	0.0%	$3,920	$801	$4,721	$0		$3,920	$801	$4,721
31	RELOCATIONS - CM	$2,613	$534	20%	$3,147	0.0%	$2,613	$534	$3,147	$0		$2,613	$534	$3,147
30	PLANNING, ENGINEERING & DESIGN	$98,903	$20,844	21%	$119,747	0.0%	$98,903	$20,844	$119,747	$0		$98,903	$20,844	$119,747
31	CONSTRUCTION MANAGEMENT	$65,937	$13,896	21%	$79,833	0.0%	$65,937	$13,896	$79,833	$0		$65,937	$13,896	$79,833
	PROJECT COST TOTALS:	$971,861	$212,242	22%	$1,184,103		$971,861	$212,242	**$1,184,103**	$0		$971,861	$212,242	$1,184,103

CHIEF, COST ENGINEERING, Jeremiah Frost

PROJECT MANAGER, Joana Savinon

CHIEF, REAL ESTATE, Sharon Caine

CHIEF, ENGINEERING, Rick Poeppelman

ESTIMATED TOTAL PROJECT COST: **$1,184,103,000**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-7B
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		ESTIMATED COST (in $1000s)				PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)				TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
				RISK BASED										
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	N Stockton, Fix B													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE)		$0	25%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$33,941	$8,513	25%	$42,454	0.0%	$33,941	$8,513	$42,454	2014Q1	0.0%	$33,941	$8,513	$42,454
11	LEVEES & FLOODWALLS	$259,009	$64,959	25%	$323,969	0.0%	$259,009	$64,959	$323,969	2014Q1	0.0%	$259,009	$64,959	$323,969
15	FLOODWAY CONTROL & DIVERSION STRU	$15,598	$3,912	25%	$19,510	0.0%	$15,598	$3,912	$19,510	2014Q1	0.0%	$15,598	$3,912	$19,510
18	CULTURAL RESOURCE PRESERVATION	$8,872	$2,225	25%	$11,098	0.0%	$8,872	$2,225	$11,098	2014Q1	0.0%	$8,872	$2,225	$11,098
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$317,421	$79,609	25%	**$397,030**		$317,421	$79,609	$397,030			$317,421	$79,609	$397,030
01	LANDS AND DAMAGES	$78,910	$22,663	29%	$101,573	0.0%	$78,910	$22,663	$101,573	2014Q1	0.0%	$78,910	$22,663	$101,573
02	RELOCATIONS	$8,378	$2,101	25%	$10,479	0.0%	$8,378	$2,101	$10,479	2014Q1	0.0%	$8,378	$2,101	$10,479
30	RELOCATIIONS - PED	$1,257	$315	25%	$1,572	0.0%	$1,257	$315	$1,572	2014Q1	0.0%	$1,257	$315	$1,572
31	RELOCATIONS - CM	$838	$210	25%	$1,048	0.0%	$838	$210	$1,048	2014Q1	0.0%	$838	$210	$1,048
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
1.0%	Planning & Environmental Compliance	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
6.0%	Engineering & Design	$19,045	$4,776	25%	$23,821	0.0%	$19,045	$4,776	$23,821	2014Q1	0.0%	$19,045	$4,776	$23,821
0.5%	Reviews, ATRs, IEPRs, VE	$1,587	$398	25%	$1,985	0.0%	$1,587	$398	$1,985	2014Q1	0.0%	$1,587	$398	$1,985
0.5%	Life Cycle Updates (cost, schedule, risks)	$1,587	$398	25%	$1,985	0.0%	$1,587	$398	$1,985	2014Q1	0.0%	$1,587	$398	$1,985
1.0%	Contracting & Reprographics	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
3.0%	Engineering During Construction	$9,523	$2,388	25%	$11,911	0.0%	$9,523	$2,388	$11,911	2014Q1	0.0%	$9,523	$2,388	$11,911
1.0%	Planning During Construction	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
1.0%	Project Operations	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$20,632	$5,175	25%	$25,807	0.0%	$20,632	$5,175	$25,807	2014Q1	0.0%	$20,632	$5,175	$25,807
1.5%	Project Operation:	$4,761	$1,194	25%	$5,955	0.0%	$4,761	$1,194	$5,955	2014Q1	0.0%	$4,761	$1,194	$5,955
2.0%	Project Management	$6,348	$1,592	25%	$7,940	0.0%	$6,348	$1,592	$7,940	2014Q1	0.0%	$6,348	$1,592	$7,940
	CONTRACT COST TOTALS:	$486,156	$124,800		**$610,956**		$486,156	$124,800	$610,956			$486,156	$124,800	**$610,956**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-7B
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		ESTIMATED COST (in $1000s)				PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)				TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	C Stockton, Fixes B & C													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE)		$0	21%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$5,811	$1,209	21%	$7,020	0.0%	$5,811	$1,209	$7,020	2014Q1	0.0%	$5,811	$1,209	$7,020
11	LEVEES & FLOODWALLS	$71,224	$14,822	21%	$86,045	0.0%	$71,224	$14,822	$86,045	2014Q1	0.0%	$71,224	$14,822	$86,045
15	FLOODWAY CONTROL & DIVERSION STRU	$14,187	$2,952	21%	$17,139	0.0%	$14,187	$2,952	$17,139	2014Q1	0.0%	$14,187	$2,952	$17,139
18	CULTURAL RESOURCE PRESERVATION	$2,726	$567	21%	$3,293	0.0%	$2,726	$567	$3,293	2014Q1	0.0%	$2,726	$567	$3,293
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$93,947	$19,550	21%	**$113,498**		$93,947	$19,550	$113,498			$93,947	$19,550	$113,498
01	LANDS AND DAMAGES	$21,622	$5,734	27%	$27,357	0.0%	$21,622	$5,734	$27,357	2014Q1	0.0%	$21,622	$5,734	$27,357
02	RELOCATIONS	$8,240	$1,715	21%	$9,955	0.0%	$8,240	$1,715	$9,955	2014Q1	0.0%	$8,240	$1,715	$9,955
30	RELOCATIIONS - PED	$1,236	$257	21%	$1,493	0.0%	$1,236	$257	$1,493	2014Q1	0.0%	$1,236	$257	$1,493
31	RELOCATIONS - CM	$824	$171	21%	$995	0.0%	$824	$171	$995	2014Q1	0.0%	$824	$171	$995
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$939	$195	21%	$1,134	0.0%	$939	$195	$1,134	2014Q1	0.0%	$939	$195	$1,134
1.0%	Planning & Environmental Compliance	$939	$195	21%	$1,134	0.0%	$939	$195	$1,134	2014Q1	0.0%	$939	$195	$1,134
6.0%	Engineering & Design	$5,637	$1,173	21%	$6,810	0.0%	$5,637	$1,173	$6,810	2014Q1	0.0%	$5,637	$1,173	$6,810
0.5%	Reviews, ATRs, IEPRs, VE	$470	$98	21%	$568	0.0%	$470	$98	$568	2014Q1	0.0%	$470	$98	$568
0.5%	Life Cycle Updates (cost, schedule, risks)	$470	$98	21%	$568	0.0%	$470	$98	$568	2014Q1	0.0%	$470	$98	$568
1.0%	Contracting & Reprographics	$939	$195	21%	$1,134	0.0%	$939	$195	$1,134	2014Q1	0.0%	$939	$195	$1,134
3.0%	Engineering During Construction	$2,818	$586	21%	$3,404	0.0%	$2,818	$586	$3,404	2014Q1	0.0%	$2,818	$586	$3,404
1.0%	Planning During Construction	$939	$195	21%	$1,134	0.0%	$939	$195	$1,134	2014Q1	0.0%	$939	$195	$1,134
1.0%	Project Operations	$939	$195	21%	$1,134	0.0%	$939	$195	$1,134	2014Q1	0.0%	$939	$195	$1,134
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$6,107	$1,271	21%	$7,378	0.0%	$6,107	$1,271	$7,378	2014Q1	0.0%	$6,107	$1,271	$7,378
1.5%	Project Operation:	$1,409	$293	21%	$1,702	0.0%	$1,409	$293	$1,702	2014Q1	0.0%	$1,409	$293	$1,702
2.0%	Project Management	$1,879	$391	21%	$2,270	0.0%	$1,879	$391	$2,270	2014Q1	0.0%	$1,879	$391	$2,270
	CONTRACT COST TOTALS:	$149,354	$32,315		**$181,670**		$149,354	$32,315	$181,670			$149,354	$32,315	**$181,670**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-7B
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	***B***	***C***	***D***	***E***	***F***	***G***	***H***	***I***	***J***	***P***	***L***	***M***	***N***	***O***
	RD 17 Fix E													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE	$0	$0	16%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$11,298	$1,813	16%	$13,112	0.0%	$11,298	$1,813	$13,112	2014Q1	0.0%	$11,298	$1,813	$13,112
11	LEVEES & FLOODWALLS	$230,577	$37,008	16%	$267,584	0.0%	$230,577	$37,008	$267,584	2014Q1	0.0%	$230,577	$37,008	$267,584
15	FLOODWAY CONTROL & DIVERSION STRU	$0	$0	16%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
18	CULTURAL RESOURCE PRESERVATION	$6,138	$985	16%	$7,124	0.0%	$6,138	$985	$7,124	2014Q1	0.0%	$6,138	$985	$7,124
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$248,014	$39,806	16%	**$287,820**		$248,014	$39,806	$287,820			$248,014	$39,806	$287,820
01	LANDS AND DAMAGES	$14,443	$3,460	24%	$17,903	0.0%	$14,443	$3,460	$17,903	2014Q1	0.0%	$14,443	$3,460	$17,903
02	RELOCATIONS	$9,514	$1,527	16%	$11,041	0.0%	$9,514	$1,527	$11,041	2014Q1	0.0%	$9,514	$1,527	$11,041
30	RELOCATIIONS - PED	$1,427	$229	16%	$1,656	0.0%	$1,427	$229	$1,656	2014Q1	0.0%	$1,427	$229	$1,656
31	RELOCATIONS - CM	$951	$153	16%	$1,104	0.0%	$951	$153	$1,104	2014Q1	0.0%	$951	$153	$1,104
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
1.0%	Planning & Environmental Compliance	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
6.0%	Engineering & Design	$14,881	$2,388	16%	$17,269	0.0%	$14,881	$2,388	$17,269	2014Q1	0.0%	$14,881	$2,388	$17,269
0.5%	Reviews, ATRs, IEPRs, VE	$1,240	$199	16%	$1,439	0.0%	$1,240	$199	$1,439	2014Q1	0.0%	$1,240	$199	$1,439
0.5%	Life Cycle Updates (cost, schedule, risks)	$1,240	$199	16%	$1,439	0.0%	$1,240	$199	$1,439	2014Q1	0.0%	$1,240	$199	$1,439
1.0%	Contracting & Reprographics	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
3.0%	Engineering During Construction	$7,440	$1,194	16%	$8,634	0.0%	$7,440	$1,194	$8,634	2014Q1	0.0%	$7,440	$1,194	$8,634
1.0%	Planning During Construction	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
1.0%	Project Operations	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$16,121	$2,587	16%	$18,708	0.0%	$16,121	$2,587	$18,708	2014Q1	0.0%	$16,121	$2,587	$18,708
1.5%	Project Operation:	$3,720	$597	16%	$4,317	0.0%	$3,720	$597	$4,317	2014Q1	0.0%	$3,720	$597	$4,317
2.0%	Project Management	$4,960	$796	16%	$5,756	0.0%	$4,960	$796	$5,756	2014Q1	0.0%	$4,960	$796	$5,756
	CONTRACT COST TOTALS:	$336,350	$55,127		**$391,477**		$336,350	$55,127	$391,477			$336,350	$55,127	**$391,477**

**** TOTAL PROJECT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-8A
PROJECT NO: 105785
LOCATION: Stockton CA

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

Civil Works Work Breakdown Structure		ESTIMATED COST (in $1000s)				PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)				TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)				
						Program Year (Budget EC): 2015 Effective Price Level Date: 1 OCT 14								
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	**TOTAL ($K)**	Spent Thru: 1-Oct-14 ($K)		COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*K*	*L*	*M*	*N*	*O*
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE	$0	$0	24%	$0	-	$0	$0	$0	$0		$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$45,200	$10,929	24%	$56,129	0.0%	$45,200	$10,929	$56,129	$0		$45,200	$10,929	$56,129
11	LEVEES & FLOODWALLS	$396,985	$94,040	24%	$491,025	0.0%	$396,985	$94,040	$491,025	$0		$396,985	$94,040	$491,025
15	FLOODWAY CONTROL & DIVERSION STRU	$29,785	$6,841	23%	$36,626	0.0%	$29,785	$6,841	$36,626	$0		$29,785	$6,841	$36,626
18	CULTURAL RESOURCE PRESERVATION	$13,743	$3,257	24%	$17,000	0.0%	$13,743	$3,257	$17,000	$0		$13,743	$3,257	$17,000
	CONSTRUCTION ESTIMATE TOTALS:	$485,713	$115,067		$600,781	0.0%	$485,713	$115,067	$600,781	$0		$485,713	$115,067	$600,781
01	LANDS AND DAMAGES	$116,166	$31,936	27%	$148,102	0.0%	$116,166	$31,936	$148,102	$0		$116,166	$31,936	$148,102
02	RELOCATIONS	$23,797	$5,443	23%	$29,240	0.0%	$23,797	$5,443	$29,240	$0		$23,797	$5,443	$29,240
30	RELOCATIONS - PED	$3,569	$816	23%	$4,386	0.0%	$3,569	$816	$4,386	$0		$3,569	$816	$4,386
31	RELOCATIONS - CM	$2,380	$544	23%	$2,924	0.0%	$2,380	$544	$2,924	$0		$2,380	$544	$2,924
30	PLANNING, ENGINEERING & DESIGN	$72,855	$17,260	24%	$90,115	0.0%	$72,855	$17,260	$90,115	$0		$72,855	$17,260	$90,115
31	CONSTRUCTION MANAGEMENT	$48,572	$11,507	24%	$60,079	0.0%	$48,572	$11,507	$60,079	$0		$48,572	$11,507	$60,079
	PROJECT COST TOTALS:	$753,052	$182,573	24%	$935,625		$753,052	$182,573	**$935,625**	$0		$753,052	$182,573	$935,625

CHIEF, COST ENGINEERING, Jeremiah Frost

PROJECT MANAGER, Joana Savinon

CHIEF, REAL ESTATE, Sharon Caine

CHIEF, ENGINEERING, Rick Poeppelman

ESTIMATED TOTAL PROJECT COST: **$935,625,000**

Filename: LSJRFS_PCS-Alt8a_20140612.xlsx
TPCS

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-8A
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
				RISK BASED										
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	N Stockton, Fix F													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE)		$0	25%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$36,818	$9,186	25%	$46,005	0.0%	$36,818	$9,186	$46,005	2014Q1	0.0%	$36,818	$9,186	$46,005
11	LEVEES & FLOODWALLS	$276,611	$69,014	25%	$345,626	0.0%	$276,611	$69,014	$345,626	2014Q1	0.0%	$276,611	$69,014	$345,626
15	FLOODWAY CONTROL & DIVERSION STRU	$15,598	$3,892	25%	$19,490	0.0%	$15,598	$3,892	$19,490	2014Q1	0.0%	$15,598	$3,892	$19,490
18	CULTURAL RESOURCE PRESERVATION	$9,616	$2,399	25%	$12,015	0.0%	$9,616	$2,399	$12,015	2014Q1	0.0%	$9,616	$2,399	$12,015
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$338,644	$84,492	25%	**$423,135**		$338,644	$84,492	$423,135			$338,644	$84,492	$423,135
01	LANDS AND DAMAGES	$88,701	$24,907	28%	$113,608	0.0%	$88,701	$24,907	$113,608	2014Q1	0.0%	$88,701	$24,907	$113,608
02	RELOCATIONS	$11,921	$2,974	25%	$14,895	0.0%	$11,921	$2,974	$14,895	2014Q1	0.0%	$11,921	$2,974	$14,895
30	RELOCATIIONS - PED	$1,788	$446	25%	$2,234	0.0%	$1,788	$446	$2,234	2014Q1	0.0%	$1,788	$446	$2,234
31	RELOCATIONS - CM	$1,192	$297	25%	$1,490	0.0%	$1,192	$297	$1,490	2014Q1	0.0%	$1,192	$297	$1,490
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$3,386	$845	25%	$4,231	0.0%	$3,386	$845	$4,231	2014Q1	0.0%	$3,386	$845	$4,231
1.0%	Planning & Environmental Compliance	$3,386	$845	25%	$4,231	0.0%	$3,386	$845	$4,231	2014Q1	0.0%	$3,386	$845	$4,231
6.0%	Engineering & Design	$20,319	$5,070	25%	$25,389	0.0%	$20,319	$5,070	$25,389	2014Q1	0.0%	$20,319	$5,070	$25,389
0.5%	Reviews, ATRs, IEPRs, VE	$1,693	$422	25%	$2,115	0.0%	$1,693	$422	$2,115	2014Q1	0.0%	$1,693	$422	$2,115
0.5%	Life Cycle Updates (cost, schedule, risks)	$1,693	$422	25%	$2,115	0.0%	$1,693	$422	$2,115	2014Q1	0.0%	$1,693	$422	$2,115
1.0%	Contracting & Reprographics	$3,386	$845	25%	$4,231	0.0%	$3,386	$845	$4,231	2014Q1	0.0%	$3,386	$845	$4,231
3.0%	Engineering During Construction	$10,159	$2,535	25%	$12,694	0.0%	$10,159	$2,535	$12,694	2014Q1	0.0%	$10,159	$2,535	$12,694
1.0%	Planning During Construction	$3,386	$845	25%	$4,231	0.0%	$3,386	$845	$4,231	2014Q1	0.0%	$3,386	$845	$4,231
1.0%	Project Operations	$3,386	$845	25%	$4,231	0.0%	$3,386	$845	$4,231	2014Q1	0.0%	$3,386	$845	$4,231
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$22,012	$5,492	25%	$27,504	0.0%	$22,012	$5,492	$27,504	2014Q1	0.0%	$22,012	$5,492	$27,504
1.5%	Project Operation:	$5,080	$1,267	25%	$6,347	0.0%	$5,080	$1,267	$6,347	2014Q1	0.0%	$5,080	$1,267	$6,347
2.0%	Project Management	$6,773	$1,690	25%	$8,463	0.0%	$6,773	$1,690	$8,463	2014Q1	0.0%	$6,773	$1,690	$8,463
	CONTRACT COST TOTALS:	$526,905	$134,239		**$661,144**		$526,905	$134,239	$661,144			$526,905	$134,239	**$661,144**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-8A
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	C Stockton, Fix D + Duck Cr													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE	$0	$0	21%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$8,382	$1,743	21%	$10,124	0.0%	$8,382	$1,743	$10,124	2014Q1	0.0%	$8,382	$1,743	$10,124
11	LEVEES & FLOODWALLS	$120,374	$25,026	21%	$145,399	0.0%	$120,374	$25,026	$145,399	2014Q1	0.0%	$120,374	$25,026	$145,399
15	FLOODWAY CONTROL & DIVERSION STRU	$14,187	$2,949	21%	$17,136	0.0%	$14,187	$2,949	$17,136	2014Q1	0.0%	$14,187	$2,949	$17,136
18	CULTURAL RESOURCE PRESERVATION	$4,127	$858	21%	$4,985	0.0%	$4,127	$858	$4,985	2014Q1	0.0%	$4,127	$858	$4,985
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$147,070	$30,576	21%	**$177,645**		$147,070	$30,576	$177,645			$147,070	$30,576	$177,645
01	LANDS AND DAMAGES	$27,465	$7,028	26%	$34,493	0.0%	$27,465	$7,028	$34,493	2014Q1	0.0%	$27,465	$7,028	$34,493
02	RELOCATIONS	$11,876	$2,469	21%	$14,345	0.0%	$11,876	$2,469	$14,345	2014Q1	0.0%	$11,876	$2,469	$14,345
30	RELOCATIIONS - PED	$1,781	$370	21%	$2,152	0.0%	$1,781	$370	$2,152	2014Q1	0.0%	$1,781	$370	$2,152
31	RELOCATIONS - CM	$1,188	$247	21%	$1,434	0.0%	$1,188	$247	$1,434	2014Q1	0.0%	$1,188	$247	$1,434
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$1,471	$306	21%	$1,777	0.0%	$1,471	$306	$1,777	2014Q1	0.0%	$1,471	$306	$1,777
1.0%	Planning & Environmental Compliance	$1,471	$306	21%	$1,777	0.0%	$1,471	$306	$1,777	2014Q1	0.0%	$1,471	$306	$1,777
6.0%	Engineering & Design	$8,824	$1,835	21%	$10,659	0.0%	$8,824	$1,835	$10,659	2014Q1	0.0%	$8,824	$1,835	$10,659
0.5%	Reviews, ATRs, IEPRs, VE	$735	$153	21%	$888	0.0%	$735	$153	$888	2014Q1	0.0%	$735	$153	$888
0.5%	Life Cycle Updates (cost, schedule, risks)	$735	$153	21%	$888	0.0%	$735	$153	$888	2014Q1	0.0%	$735	$153	$888
1.0%	Contracting & Reprographics	$1,471	$306	21%	$1,777	0.0%	$1,471	$306	$1,777	2014Q1	0.0%	$1,471	$306	$1,777
3.0%	Engineering During Construction	$4,412	$917	21%	$5,329	0.0%	$4,412	$917	$5,329	2014Q1	0.0%	$4,412	$917	$5,329
1.0%	Planning During Construction	$1,471	$306	21%	$1,777	0.0%	$1,471	$306	$1,777	2014Q1	0.0%	$1,471	$306	$1,777
1.0%	Project Operations	$1,471	$306	21%	$1,777	0.0%	$1,471	$306	$1,777	2014Q1	0.0%	$1,471	$306	$1,777
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$9,560	$1,988	21%	$11,548	0.0%	$9,560	$1,988	$11,548	2014Q1	0.0%	$9,560	$1,988	$11,548
1.5%	Project Operation:	$2,206	$459	21%	$2,665	0.0%	$2,206	$459	$2,665	2014Q1	0.0%	$2,206	$459	$2,665
2.0%	Project Management	$2,941	$611	21%	$3,552	0.0%	$2,941	$611	$3,552	2014Q1	0.0%	$2,941	$611	$3,552
	CONTRACT COST TOTALS:	$226,147	$48,334		**$274,482**		$226,147	$48,334	$274,482			$226,147	$48,334	**$274,482**

**** TOTAL PROJECT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-8B
PROJECT NO: 105785
LOCATION: Stockton CA

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
						Program Year (Budget EC):			2015					
						Effective Price Level Date:			1 OCT 14					
										Spent Thru:				
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	**TOTAL ($K)**	1-Oct-14 ($K)		COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*K*	*L*	*M*	*N*	*O*
02	RELOCATIONS	$0	$0	21%	$0	-	$0	$0	$0	$0		$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$56,237	$12,633	22%	$68,870	0.0%	$56,237	$12,633	$68,870	$0		$56,237	$12,633	$68,870
11	LEVEES & FLOODWALLS	$621,902	$129,102	21%	$751,004	0.0%	$621,902	$129,102	$751,004	$0		$621,902	$129,102	$751,004
15	FLOODWAY CONTROL & DIVERSION STRU	$29,785	$6,746	23%	$36,531	0.0%	$29,785	$6,746	$36,531	$0		$29,785	$6,746	$36,531
18	CULTURAL RESOURCE PRESERVATION	$19,703	$4,178	21%	$23,880	0.0%	$19,703	$4,178	$23,880	$0		$19,703	$4,178	$23,880
	CONSTRUCTION ESTIMATE TOTALS:	$727,626	$152,659		$880,286	0.0%	$727,626	$152,659	$880,286	$0		$727,626	$152,659	$880,286
01	LANDS AND DAMAGES	$128,824	$34,906	27%	$163,730	0.0%	$128,824	$34,906	$163,730	$0		$128,824	$34,906	$163,730
02	RELOCATIONS	$33,000	$6,828	21%	$39,828	0.0%	$33,000	$6,828	$39,828	$0		$33,000	$6,828	$39,828
30	RELOCATIIONS - PED	$4,950	$1,024	21%	$5,974	0.0%	$4,950	$1,024	$5,974	$0		$4,950	$1,024	$5,974
31	RELOCATIONS - CM	$3,300	$683	21%	$3,983	0.0%	$3,300	$683	$3,983	$0		$3,300	$683	$3,983
30	PLANNING, ENGINEERING & DESIGN	$109,143	$22,899	21%	$132,042	0.0%	$109,143	$22,899	$132,042	$0		$109,143	$22,899	$132,042
31	CONSTRUCTION MANAGEMENT	$72,764	$15,266	21%	$88,030	0.0%	$72,764	$15,266	$88,030	$0		$72,764	$15,266	$88,030
	PROJECT COST TOTALS:	$1,079,607	$234,266	22%	$1,313,873		$1,079,607	$234,266	**$1,313,873**	$0		$1,079,607	$234,266	$1,313,873

CHIEF, COST ENGINEERING, Jeremiah Frost

PROJECT MANAGER, Joana Savinon

CHIEF, REAL ESTATE, Sharon Caine

CHIEF, ENGINEERING, Rick Poeppelman

ESTIMATED TOTAL PROJECT COST: **$1,313,873,000**

Filename: LSJRFS_PCS-Alt8b_20140612.xlsx
TPCS

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-8B
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
WBS NUMBER *A*	Civil Works Feature & Sub-Feature Description *B*	COST ($K) *C*	CNTG ($K) *D*	RISK BASED CNTG (%) *E*	TOTAL ($K) *F*	ESC (%) *G*	COST ($K) *H*	CNTG ($K) *I*	TOTAL ($K) *J*	Mid-Point Date *P*	ESC (%) *L*	COST ($K) *M*	CNTG ($K) *N*	FULL ($K) *O*
	N Stockton, Fix F													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE)		$0	25%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$36,818	$9,186	25%	$46,005	0.0%	$36,818	$9,186	$46,005	2014Q1	0.0%	$36,818	$9,186	$46,005
11	LEVEES & FLOODWALLS	$276,611	$69,014	25%	$345,626	0.0%	$276,611	$69,014	$345,626	2014Q1	0.0%	$276,611	$69,014	$345,626
15	FLOODWAY CONTROL & DIVERSION STRU	$15,598	$3,892	25%	$19,490	0.0%	$15,598	$3,892	$19,490	2014Q1	0.0%	$15,598	$3,892	$19,490
18	CULTURAL RESOURCE PRESERVATION	$9,598	$2,395	25%	$11,992	0.0%	$9,598	$2,395	$11,992	2014Q1	0.0%	$9,598	$2,395	$11,992
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$338,625	$84,487	25%	**$423,112**		$338,625	$84,487	$423,112			$338,625	$84,487	$423,112
01	LANDS AND DAMAGES	$87,719	$24,623	28%	$112,342	0.0%	$87,719	$24,623	$112,342	2014Q1	0.0%	$87,719	$24,623	$112,342
02	RELOCATIONS	$11,921	$2,974	25%	$14,895	0.0%	$11,921	$2,974	$14,895	2014Q1	0.0%	$11,921	$2,974	$14,895
30	RELOCATIIONS - PED	$1,788	$446	25%	$2,234	0.0%	$1,788	$446	$2,234	2014Q1	0.0%	$1,788	$446	$2,234
31	RELOCATIONS - CM	$1,192	$297	25%	$1,490	0.0%	$1,192	$297	$1,490	2014Q1	0.0%	$1,192	$297	$1,490
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$3,386	$845	25%	$4,231	0.0%	$3,386	$845	$4,231	2014Q1	0.0%	$3,386	$845	$4,231
1.0%	Planning & Environmental Compliance	$3,386	$845	25%	$4,231	0.0%	$3,386	$845	$4,231	2014Q1	0.0%	$3,386	$845	$4,231
6.0%	Engineering & Design	$20,318	$5,069	25%	$25,387	0.0%	$20,318	$5,069	$25,387	2014Q1	0.0%	$20,318	$5,069	$25,387
0.5%	Reviews, ATRs, IEPRs, VE	$1,693	$422	25%	$2,115	0.0%	$1,693	$422	$2,115	2014Q1	0.0%	$1,693	$422	$2,115
0.5%	Life Cycle Updates (cost, schedule, risks)	$1,693	$422	25%	$2,115	0.0%	$1,693	$422	$2,115	2014Q1	0.0%	$1,693	$422	$2,115
1.0%	Contracting & Reprographics	$3,386	$845	25%	$4,231	0.0%	$3,386	$845	$4,231	2014Q1	0.0%	$3,386	$845	$4,231
3.0%	Engineering During Construction	$10,159	$2,535	25%	$12,694	0.0%	$10,159	$2,535	$12,694	2014Q1	0.0%	$10,159	$2,535	$12,694
1.0%	Planning During Construction	$3,386	$845	25%	$4,231	0.0%	$3,386	$845	$4,231	2014Q1	0.0%	$3,386	$845	$4,231
1.0%	Project Operations	$3,386	$845	25%	$4,231	0.0%	$3,386	$845	$4,231	2014Q1	0.0%	$3,386	$845	$4,231
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$22,011	$5,492	25%	$27,503	0.0%	$22,011	$5,492	$27,503	2014Q1	0.0%	$22,011	$5,492	$27,503
1.5%	Project Operation:	$5,079	$1,267	25%	$6,346	0.0%	$5,079	$1,267	$6,346	2014Q1	0.0%	$5,079	$1,267	$6,346
2.0%	Project Management	$6,773	$1,690	25%	$8,463	0.0%	$6,773	$1,690	$8,463	2014Q1	0.0%	$6,773	$1,690	$8,463
	CONTRACT COST TOTALS:	$525,902	$133,949		**$659,851**		$525,902	$133,949	$659,851			$525,902	$133,949	**$659,851**

Filename: LSJRFS_PCS-Alt8b_20140612.xlsx
TPCS

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-8B
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	C Stockton, Fix D													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE	$0	$0	20%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$8,120	$1,634	20%	$9,754	0.0%	$8,120	$1,634	$9,754	2014Q1	0.0%	$8,120	$1,634	$9,754
11	LEVEES & FLOODWALLS	$114,714	$23,080	20%	$137,794	0.0%	$114,714	$23,080	$137,794	2014Q1	0.0%	$114,714	$23,080	$137,794
15	FLOODWAY CONTROL & DIVERSION STRU	$14,187	$2,854	20%	$17,041	0.0%	$14,187	$2,854	$17,041	2014Q1	0.0%	$14,187	$2,854	$17,041
18	CULTURAL RESOURCE PRESERVATION	$3,957	$796	20%	$4,753	0.0%	$3,957	$796	$4,753	2014Q1	0.0%	$3,957	$796	$4,753
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$140,978	$28,365	20%	**$169,342**		$140,978	$28,365	$169,342			$140,978	$28,365	$169,342
01	LANDS AND DAMAGES	$26,150	$6,686	26%	$32,836	0.0%	$26,150	$6,686	$32,836	2014Q1	0.0%	$26,150	$6,686	$32,836
02	RELOCATIONS	$11,566	$2,327	20%	$13,893	0.0%	$11,566	$2,327	$13,893	2014Q1	0.0%	$11,566	$2,327	$13,893
30	RELOCATIIONS - PED	$1,735	$349	20%	$2,084	0.0%	$1,735	$349	$2,084	2014Q1	0.0%	$1,735	$349	$2,084
31	RELOCATIONS - CM	$1,157	$233	20%	$1,389	0.0%	$1,157	$233	$1,389	2014Q1	0.0%	$1,157	$233	$1,389
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$1,410	$284	20%	$1,694	0.0%	$1,410	$284	$1,694	2014Q1	0.0%	$1,410	$284	$1,694
1.0%	Planning & Environmental Compliance	$1,410	$284	20%	$1,694	0.0%	$1,410	$284	$1,694	2014Q1	0.0%	$1,410	$284	$1,694
6.0%	Engineering & Design	$8,459	$1,702	20%	$10,161	0.0%	$8,459	$1,702	$10,161	2014Q1	0.0%	$8,459	$1,702	$10,161
0.5%	Reviews, ATRs, IEPRs, VE	$705	$142	20%	$847	0.0%	$705	$142	$847	2014Q1	0.0%	$705	$142	$847
0.5%	Life Cycle Updates (cost, schedule, risks)	$705	$142	20%	$847	0.0%	$705	$142	$847	2014Q1	0.0%	$705	$142	$847
1.0%	Contracting & Reprographics	$1,410	$284	20%	$1,694	0.0%	$1,410	$284	$1,694	2014Q1	0.0%	$1,410	$284	$1,694
3.0%	Engineering During Construction	$4,229	$851	20%	$5,080	0.0%	$4,229	$851	$5,080	2014Q1	0.0%	$4,229	$851	$5,080
1.0%	Planning During Construction	$1,410	$284	20%	$1,694	0.0%	$1,410	$284	$1,694	2014Q1	0.0%	$1,410	$284	$1,694
1.0%	Project Operations	$1,410	$284	20%	$1,694	0.0%	$1,410	$284	$1,694	2014Q1	0.0%	$1,410	$284	$1,694
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$9,164	$1,844	20%	$11,008	0.0%	$9,164	$1,844	$11,008	2014Q1	0.0%	$9,164	$1,844	$11,008
1.5%	Project Operation:	$2,115	$426	20%	$2,541	0.0%	$2,115	$426	$2,541	2014Q1	0.0%	$2,115	$426	$2,541
2.0%	Project Management	$2,820	$567	20%	$3,387	0.0%	$2,820	$567	$3,387	2014Q1	0.0%	$2,820	$567	$3,387
	CONTRACT COST TOTALS:	$216,832	$45,052		**$261,883**		$216,832	$45,052	$261,883			$216,832	$45,052	**$261,883**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-8B
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	RD 17 Fix E													
02	RELOCATIONS		$0	16%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$11,298	$1,813	16%	$13,112	0.0%	$11,298	$1,813	$13,112	2014Q1	0.0%	$11,298	$1,813	$13,112
11	LEVEES & FLOODWALLS	$230,577	$37,008	16%	$267,584	0.0%	$230,577	$37,008	$267,584	2014Q1	0.0%	$230,577	$37,008	$267,584
15	FLOODWAY CONTROL & DIVERSION STRUCTURE		$0	16%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
18	CULTURAL RESOURCE PRESERVATION	$6,148	$987	16%	$7,135	0.0%	$6,148	$987	$7,135	2014Q1	0.0%	$6,148	$987	$7,135
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$248,023	$39,808	16%	**$287,831**		$248,023	$39,808	$287,831			$248,023	$39,808	$287,831
01	LANDS AND DAMAGES	$14,955	$3,597	24%	$18,551	0.0%	$14,955	$3,597	$18,551	2014Q1	0.0%	$14,955	$3,597	$18,551
02	RELOCATIONS	$9,514	$1,527	16%	$11,041	0.0%	$9,514	$1,527	$11,041	2014Q1	0.0%	$9,514	$1,527	$11,041
30	RELOCATIIONS - PED	$1,427	$229	16%	$1,656	0.0%	$1,427	$229	$1,656	2014Q1	0.0%	$1,427	$229	$1,656
31	RELOCATIONS - CM	$951	$153	16%	$1,104	0.0%	$951	$153	$1,104	2014Q1	0.0%	$951	$153	$1,104
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
1.0%	Planning & Environmental Compliance	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
6.0%	Engineering & Design	$14,881	$2,388	16%	$17,269	0.0%	$14,881	$2,388	$17,269	2014Q1	0.0%	$14,881	$2,388	$17,269
0.5%	Reviews, ATRs, IEPRs, VE	$1,240	$199	16%	$1,439	0.0%	$1,240	$199	$1,439	2014Q1	0.0%	$1,240	$199	$1,439
0.5%	Life Cycle Updates (cost, schedule, risks)	$1,240	$199	16%	$1,439	0.0%	$1,240	$199	$1,439	2014Q1	0.0%	$1,240	$199	$1,439
1.0%	Contracting & Reprographics	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
3.0%	Engineering During Construction	$7,441	$1,194	16%	$8,635	0.0%	$7,441	$1,194	$8,635	2014Q1	0.0%	$7,441	$1,194	$8,635
1.0%	Planning During Construction	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
1.0%	Project Operations	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$16,122	$2,588	16%	$18,710	0.0%	$16,122	$2,588	$18,710	2014Q1	0.0%	$16,122	$2,588	$18,710
1.5%	Project Operation:	$3,720	$597	16%	$4,317	0.0%	$3,720	$597	$4,317	2014Q1	0.0%	$3,720	$597	$4,317
2.0%	Project Management	$4,960	$796	16%	$5,756	0.0%	$4,960	$796	$5,756	2014Q1	0.0%	$4,960	$796	$5,756
	CONTRACT COST TOTALS:	$336,874	$55,265		**$392,139**		$336,874	$55,265	$392,139			$336,874	$55,265	**$392,139**

**** TOTAL PROJECT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-9A
PROJECT NO: 105785
LOCATION: Stockton CA

DISTRICT: SPD South Pacific Division
PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

Civil Works Work Breakdown Structure		ESTIMATED COST (in $1000s)				PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)				TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)				
						Program Year (Budget EC): 2015 Effective Price Level Date: 1 OCT 14								
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	**TOTAL ($K)**	Spent Thru: 1-Oct-14 ($K)		COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*K*	*L*	*M*	*N*	*O*
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE	$0	$0	24%	$0	-	$0	$0	$0	$0		$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$40,970	$10,059	25%	$51,029	0.0%	$40,970	$10,059	$51,029	$0		$40,970	$10,059	$51,029
11	LEVEES & FLOODWALLS	$340,510	$82,890	24%	$423,400	0.0%	$340,510	$82,890	$423,400	$0		$340,510	$82,890	$423,400
15	FLOODWAY CONTROL & DIVERSION STRU	$39,968	$9,273	23%	$49,241	0.0%	$39,968	$9,273	$49,241	$0		$39,968	$9,273	$49,241
18	CULTURAL RESOURCE PRESERVATION	$12,198	$2,957	24%	$15,156	0.0%	$12,198	$2,957	$15,156	$0		$12,198	$2,957	$15,156
	CONSTRUCTION ESTIMATE TOTALS:	$433,646	$105,179		$538,826	0.0%	$433,646	$105,179	$538,826	$0		$433,646	$105,179	$538,826
01	LANDS AND DAMAGES	$105,180	$29,287	28%	$134,468	0.0%	$105,180	$29,287	$134,468	$0		$105,180	$29,287	$134,468
02	RELOCATIONS	$16,928	$3,982	24%	$20,910	0.0%	$16,928	$3,982	$20,910	$0		$16,928	$3,982	$20,910
30	RELOCATIONS - PED	$2,539	$597	24%	$3,136	0.0%	$2,539	$597	$3,136	$0		$2,539	$597	$3,136
31	RELOCATIONS - CM	$1,693	$398	24%	$2,091	0.0%	$1,693	$398	$2,091	$0		$1,693	$398	$2,091
30	PLANNING, ENGINEERING & DESIGN	$65,044	$15,776	24%	$80,820	0.0%	$65,044	$15,776	$80,820	$0		$65,044	$15,776	$80,820
31	CONSTRUCTION MANAGEMENT	$43,365	$10,518	24%	$53,883	0.0%	$43,365	$10,518	$53,883	$0		$43,365	$10,518	$53,883
	PROJECT COST TOTALS:	$668,395	$165,738	25%	$834,134		$668,395	$165,738	**$834,134**	$0		$668,395	$165,738	$834,134

CHIEF, COST ENGINEERING, Jeremiah Frost

PROJECT MANAGER, Joana Savinon

CHIEF, REAL ESTATE, Sharon Caine

CHIEF, ENGINEERING, Rick Poeppelman

ESTIMATED TOTAL PROJECT COST: **$834,134,000**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-9A
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
				RISK BASED										
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	N Stockton, Fix B													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE	$0	$0	25%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$33,941	$8,513	25%	$42,454	0.0%	$33,941	$8,513	$42,454	2014Q1	0.0%	$33,941	$8,513	$42,454
11	LEVEES & FLOODWALLS	$259,009	$64,959	25%	$323,969	0.0%	$259,009	$64,959	$323,969	2014Q1	0.0%	$259,009	$64,959	$323,969
15	FLOODWAY CONTROL & DIVERSION STRU	$15,598	$3,912	25%	$19,510	0.0%	$15,598	$3,912	$19,510	2014Q1	0.0%	$15,598	$3,912	$19,510
18	CULTURAL RESOURCE PRESERVATION	$8,887	$2,229	25%	$11,116	0.0%	$8,887	$2,229	$11,116	2014Q1	0.0%	$8,887	$2,229	$11,116
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$317,436	$79,613	25%	**$397,048**		$317,436	$79,613	$397,048			$317,436	$79,613	$397,048
01	LANDS AND DAMAGES	$79,695	$22,880	29%	$102,576	0.0%	$79,695	$22,880	$102,576	2014Q1	0.0%	$79,695	$22,880	$102,576
02	RELOCATIONS	$8,378	$2,101	25%	$10,479	0.0%	$8,378	$2,101	$10,479	2014Q1	0.0%	$8,378	$2,101	$10,479
30	RELOCATIIONS - PED	$1,257	$315	25%	$1,572	0.0%	$1,257	$315	$1,572	2014Q1	0.0%	$1,257	$315	$1,572
31	RELOCATIONS - CM	$838	$210	25%	$1,048	0.0%	$838	$210	$1,048	2014Q1	0.0%	$838	$210	$1,048
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
1.0%	Planning & Environmental Compliance	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
6.0%	Engineering & Design	$19,046	$4,777	25%	$23,823	0.0%	$19,046	$4,777	$23,823	2014Q1	0.0%	$19,046	$4,777	$23,823
0.5%	Reviews, ATRs, IEPRs, VE	$1,587	$398	25%	$1,985	0.0%	$1,587	$398	$1,985	2014Q1	0.0%	$1,587	$398	$1,985
0.5%	Life Cycle Updates (cost, schedule, risks)	$1,587	$398	25%	$1,985	0.0%	$1,587	$398	$1,985	2014Q1	0.0%	$1,587	$398	$1,985
1.0%	Contracting & Reprographics	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
3.0%	Engineering During Construction	$9,523	$2,388	25%	$11,911	0.0%	$9,523	$2,388	$11,911	2014Q1	0.0%	$9,523	$2,388	$11,911
1.0%	Planning During Construction	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
1.0%	Project Operations	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$20,633	$5,175	25%	$25,808	0.0%	$20,633	$5,175	$25,808	2014Q1	0.0%	$20,633	$5,175	$25,808
1.5%	Project Operation:	$4,762	$1,194	25%	$5,956	0.0%	$4,762	$1,194	$5,956	2014Q1	0.0%	$4,762	$1,194	$5,956
2.0%	Project Management	$6,349	$1,592	25%	$7,941	0.0%	$6,349	$1,592	$7,941	2014Q1	0.0%	$6,349	$1,592	$7,941
	CONTRACT COST TOTALS:	$486,960	$125,023		**$611,983**		$486,960	$125,023	$611,983			$486,960	$125,023	**$611,983**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-9A
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	C Stockton, Fixes B & C + Duck Cr & M Ch													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE	$0	$0	22%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$7,029	$1,546	22%	$8,575	0.0%	$7,029	$1,546	$8,575	2014Q1	0.0%	$7,029	$1,546	$8,575
11	LEVEES & FLOODWALLS	$81,501	$17,930	22%	$99,431	0.0%	$81,501	$17,930	$99,431	2014Q1	0.0%	$81,501	$17,930	$99,431
15	FLOODWAY CONTROL & DIVERSION STRU	$24,370	$5,361	22%	$29,731	0.0%	$24,370	$5,361	$29,731	2014Q1	0.0%	$24,370	$5,361	$29,731
18	CULTURAL RESOURCE PRESERVATION	$3,311	$728	22%	$4,040	0.0%	$3,311	$728	$4,040	2014Q1	0.0%	$3,311	$728	$4,040
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$116,211	$25,566	22%	**$141,777**		$116,211	$25,566	$141,777			$116,211	$25,566	$141,777
01	LANDS AND DAMAGES	$25,485	$6,407	25%	$31,892	0.0%	$25,485	$6,407	$31,892	2014Q1	0.0%	$25,485	$6,407	$31,892
02	RELOCATIONS	$8,550	$1,881	22%	$10,431	0.0%	$8,550	$1,881	$10,431	2014Q1	0.0%	$8,550	$1,881	$10,431
30	RELOCATIIONS - PED	$1,282	$282	22%	$1,565	0.0%	$1,282	$282	$1,565	2014Q1	0.0%	$1,282	$282	$1,565
31	RELOCATIONS - CM	$855	$188	22%	$1,043	0.0%	$855	$188	$1,043	2014Q1	0.0%	$855	$188	$1,043
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$1,162	$256	22%	$1,418	0.0%	$1,162	$256	$1,418	2014Q1	0.0%	$1,162	$256	$1,418
1.0%	Planning & Environmental Compliance	$1,162	$256	22%	$1,418	0.0%	$1,162	$256	$1,418	2014Q1	0.0%	$1,162	$256	$1,418
6.0%	Engineering & Design	$6,973	$1,534	22%	$8,507	0.0%	$6,973	$1,534	$8,507	2014Q1	0.0%	$6,973	$1,534	$8,507
0.5%	Reviews, ATRs, IEPRs, VE	$581	$128	22%	$709	0.0%	$581	$128	$709	2014Q1	0.0%	$581	$128	$709
0.5%	Life Cycle Updates (cost, schedule, risks)	$581	$128	22%	$709	0.0%	$581	$128	$709	2014Q1	0.0%	$581	$128	$709
1.0%	Contracting & Reprographics	$1,162	$256	22%	$1,418	0.0%	$1,162	$256	$1,418	2014Q1	0.0%	$1,162	$256	$1,418
3.0%	Engineering During Construction	$3,486	$767	22%	$4,253	0.0%	$3,486	$767	$4,253	2014Q1	0.0%	$3,486	$767	$4,253
1.0%	Planning During Construction	$1,162	$256	22%	$1,418	0.0%	$1,162	$256	$1,418	2014Q1	0.0%	$1,162	$256	$1,418
1.0%	Project Operations	$1,162	$256	22%	$1,418	0.0%	$1,162	$256	$1,418	2014Q1	0.0%	$1,162	$256	$1,418
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$7,554	$1,662	22%	$9,216	0.0%	$7,554	$1,662	$9,216	2014Q1	0.0%	$7,554	$1,662	$9,216
1.5%	Project Operation:	$1,743	$383	22%	$2,126	0.0%	$1,743	$383	$2,126	2014Q1	0.0%	$1,743	$383	$2,126
2.0%	Project Management	$2,324	$511	22%	$2,835	0.0%	$2,324	$511	$2,835	2014Q1	0.0%	$2,324	$511	$2,835
	CONTRACT COST TOTALS:	$181,435	$40,716		**$222,151**		$181,435	$40,716	$222,151			$181,435	$40,716	**$222,151**

Filename: LSJRFS_PCS-Alt9a_20140612.xlsx
TPCS

**** TOTAL PROJECT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-9B
PROJECT NO: 105785
LOCATION: Stockton CA

DISTRICT: SPD South Pacific Division
PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

Program Year (Budget EC): 2015
Effective Price Level Date: 1 OCT 14

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	**TOTAL ($K)**	Spent Thru: 1-Oct-14 ($K)		COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*K*	*L*	*M*	*N*	*O*
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE	$0	$0	21%	$0	-	$0	$0	$0	$0		$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$51,976	$11,819	23%	$63,794	0.0%	$51,976	$11,819	$63,794	$0		$51,976	$11,819	$63,794
11	LEVEES & FLOODWALLS	$565,428	$118,774	21%	$684,202	0.0%	$565,428	$118,774	$684,202	$0		$565,428	$118,774	$684,202
15	FLOODWAY CONTROL & DIVERSION STRU	$39,968	$9,312	23%	$49,280	0.0%	$39,968	$9,312	$49,280	$0		$39,968	$9,312	$49,280
18	CULTURAL RESOURCE PRESERVATION	$18,158	$3,908	22%	$22,066	0.0%	$18,158	$3,908	$22,066	$0		$18,158	$3,908	$22,066
	CONSTRUCTION ESTIMATE TOTALS:	$675,530	$143,812		$819,343	0.0%	$675,530	$143,812	$819,343	$0		$675,530	$143,812	$819,343
01	LANDS AND DAMAGES	$117,879	$32,277	27%	$150,156	0.0%	$117,879	$32,277	$150,156	$0		$117,879	$32,277	$150,156
02	RELOCATIONS	$26,131	$5,454	21%	$31,585	0.0%	$26,131	$5,454	$31,585	$0		$26,131	$5,454	$31,585
30	RELOCATIONS - PED	$3,920	$818	21%	$4,738	0.0%	$3,920	$818	$4,738	$0		$3,920	$818	$4,738
31	RELOCATIONS - CM	$2,613	$545	21%	$3,159	0.0%	$2,613	$545	$3,159	$0		$2,613	$545	$3,159
30	PLANNING, ENGINEERING & DESIGN	$101,327	$21,571	21%	$122,898	0.0%	$101,327	$21,571	$122,898	$0		$101,327	$21,571	$122,898
31	CONSTRUCTION MANAGEMENT	$67,551	$14,381	21%	$81,932	0.0%	$67,551	$14,381	$81,932	$0		$67,551	$14,381	$81,932
	PROJECT COST TOTALS:	$994,951	$218,859	22%	$1,213,810		$994,951	$218,859	**$1,213,810**	$0		$994,951	$218,859	$1,213,810

CHIEF, COST ENGINEERING, Jeremiah Frost

PROJECT MANAGER, Joana Savinon

CHIEF, REAL ESTATE, Sharon Caine

CHIEF, ENGINEERING, Rick Poeppelman

ESTIMATED TOTAL PROJECT COST: **$1,213,810,000**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-9B
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Estimate Prepared: 6/9/2014
Effective Price Level: 41913

Program Year (Budget EC): 2015
Effective Price Level Date: 1 OCT 14

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
				RISK BASED										
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	N Stockton, Fix B													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE	$0	$0	25%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$33,941	$8,513	25%	$42,454	0.0%	$33,941	$8,513	$42,454	2014Q1	0.0%	$33,941	$8,513	$42,454
11	LEVEES & FLOODWALLS	$259,009	$64,959	25%	$323,969	0.0%	$259,009	$64,959	$323,969	2014Q1	0.0%	$259,009	$64,959	$323,969
15	FLOODWAY CONTROL & DIVERSION STRU	$15,598	$3,912	25%	$19,510	0.0%	$15,598	$3,912	$19,510	2014Q1	0.0%	$15,598	$3,912	$19,510
18	CULTURAL RESOURCE PRESERVATION	$8,873	$2,225	25%	$11,098	0.0%	$8,873	$2,225	$11,098	2014Q1	0.0%	$8,873	$2,225	$11,098
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$317,421	$79,609	25%	**$397,030**		$317,421	$79,609	$397,030			$317,421	$79,609	$397,030
01	LANDS AND DAMAGES	$78,925	$22,659	29%	$101,585	0.0%	$78,925	$22,659	$101,585	2014Q1	0.0%	$78,925	$22,659	$101,585
02	RELOCATIONS	$8,378	$2,101	25%	$10,479	0.0%	$8,378	$2,101	$10,479	2014Q1	0.0%	$8,378	$2,101	$10,479
30	RELOCATIIONS - PED	$1,257	$315	25%	$1,572	0.0%	$1,257	$315	$1,572	2014Q1	0.0%	$1,257	$315	$1,572
31	RELOCATIONS - CM	$838	$210	25%	$1,048	0.0%	$838	$210	$1,048	2014Q1	0.0%	$838	$210	$1,048
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
1.0%	Planning & Environmental Compliance	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
6.0%	Engineering & Design	$19,045	$4,776	25%	$23,821	0.0%	$19,045	$4,776	$23,821	2014Q1	0.0%	$19,045	$4,776	$23,821
0.5%	Reviews, ATRs, IEPRs, VE	$1,587	$398	25%	$1,985	0.0%	$1,587	$398	$1,985	2014Q1	0.0%	$1,587	$398	$1,985
0.5%	Life Cycle Updates (cost, schedule, risks)	$1,587	$398	25%	$1,985	0.0%	$1,587	$398	$1,985	2014Q1	0.0%	$1,587	$398	$1,985
1.0%	Contracting & Reprographics	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
3.0%	Engineering During Construction	$9,523	$2,388	25%	$11,911	0.0%	$9,523	$2,388	$11,911	2014Q1	0.0%	$9,523	$2,388	$11,911
1.0%	Planning During Construction	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
1.0%	Project Operations	$3,174	$796	25%	$3,970	0.0%	$3,174	$796	$3,970	2014Q1	0.0%	$3,174	$796	$3,970
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$20,632	$5,175	25%	$25,807	0.0%	$20,632	$5,175	$25,807	2014Q1	0.0%	$20,632	$5,175	$25,807
1.5%	Project Operation:	$4,761	$1,194	25%	$5,955	0.0%	$4,761	$1,194	$5,955	2014Q1	0.0%	$4,761	$1,194	$5,955
2.0%	Project Management	$6,348	$1,592	25%	$7,940	0.0%	$6,348	$1,592	$7,940	2014Q1	0.0%	$6,348	$1,592	$7,940
	CONTRACT COST TOTALS:	$486,172	$124,797		**$610,969**		$486,172	$124,797	$610,969			$486,172	$124,797	**$610,969**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-9B
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		**ESTIMATED COST (in $1000s)**				**PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)**				**TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)**				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
WBS NUMBER *A*	Civil Works Feature & Sub-Feature Description *B*	COST ($K) *C*	CNTG ($K) *D*	CNTG (%) *E*	TOTAL ($K) *F*	ESC (%) *G*	COST ($K) *H*	CNTG ($K) *I*	TOTAL ($K) *J*	Mid-Point Date *P*	ESC (%) *L*	COST ($K) *M*	CNTG ($K) *N*	FULL ($K) *O*
	C Stockton, Fixes B & C + M Ch													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE	$0	$0	22%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$6,736	$1,493	22%	$8,229	0.0%	$6,736	$1,493	$8,229	2014Q1	0.0%	$6,736	$1,493	$8,229
11	LEVEES & FLOODWALLS	$75,842	$16,807	22%	$92,649	0.0%	$75,842	$16,807	$92,649	2014Q1	0.0%	$75,842	$16,807	$92,649
15	FLOODWAY CONTROL & DIVERSION STRU	$24,370	$5,400	22%	$29,770	0.0%	$24,370	$5,400	$29,770	2014Q1	0.0%	$24,370	$5,400	$29,770
18	CULTURAL RESOURCE PRESERVATION	$3,146	$697	22%	$3,843	0.0%	$3,146	$697	$3,843	2014Q1	0.0%	$3,146	$697	$3,843
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$110,095	$24,397	22%	**$134,492**		$110,095	$24,397	$134,492			$110,095	$24,397	$134,492
01	LANDS AND DAMAGES	$24,469	$6,154	25%	$30,623	0.0%	$24,469	$6,154	$30,623	2014Q1	0.0%	$24,469	$6,154	$30,623
02	RELOCATIONS	$8,240	$1,826	22%	$10,066	0.0%	$8,240	$1,826	$10,066	2014Q1	0.0%	$8,240	$1,826	$10,066
30	RELOCATIIONS - PED	$1,236	$274	22%	$1,510	0.0%	$1,236	$274	$1,510	2014Q1	0.0%	$1,236	$274	$1,510
31	RELOCATIONS - CM	$824	$183	22%	$1,007	0.0%	$824	$183	$1,007	2014Q1	0.0%	$824	$183	$1,007
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$1,101	$244	22%	$1,345	0.0%	$1,101	$244	$1,345	2014Q1	0.0%	$1,101	$244	$1,345
1.0%	Planning & Environmental Compliance	$1,101	$244	22%	$1,345	0.0%	$1,101	$244	$1,345	2014Q1	0.0%	$1,101	$244	$1,345
6.0%	Engineering & Design	$6,606	$1,464	22%	$8,070	0.0%	$6,606	$1,464	$8,070	2014Q1	0.0%	$6,606	$1,464	$8,070
0.5%	Reviews, ATRs, IEPRs, VE	$550	$122	22%	$672	0.0%	$550	$122	$672	2014Q1	0.0%	$550	$122	$672
0.5%	Life Cycle Updates (cost, schedule, risks)	$550	$122	22%	$672	0.0%	$550	$122	$672	2014Q1	0.0%	$550	$122	$672
1.0%	Contracting & Reprographics	$1,101	$244	22%	$1,345	0.0%	$1,101	$244	$1,345	2014Q1	0.0%	$1,101	$244	$1,345
3.0%	Engineering During Construction	$3,303	$732	22%	$4,035	0.0%	$3,303	$732	$4,035	2014Q1	0.0%	$3,303	$732	$4,035
1.0%	Planning During Construction	$1,101	$244	22%	$1,345	0.0%	$1,101	$244	$1,345	2014Q1	0.0%	$1,101	$244	$1,345
1.0%	Project Operations	$1,101	$244	22%	$1,345	0.0%	$1,101	$244	$1,345	2014Q1	0.0%	$1,101	$244	$1,345
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$7,156	$1,586	22%	$8,742	0.0%	$7,156	$1,586	$8,742	2014Q1	0.0%	$7,156	$1,586	$8,742
1.5%	Project Operation:	$1,651	$366	22%	$2,017	0.0%	$1,651	$366	$2,017	2014Q1	0.0%	$1,651	$366	$2,017
2.0%	Project Management	$2,202	$488	22%	$2,690	0.0%	$2,202	$488	$2,690	2014Q1	0.0%	$2,202	$488	$2,690
	CONTRACT COST TOTALS:	$172,386	$38,932		**$211,319**		$172,386	$38,932	$211,319			$172,386	$38,932	**$211,319**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Lower San Joaquin River Feas Study - Alt LS-9B
LOCATION: Stockton CA
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (Alternatives)

DISTRICT: SPD South Pacific Division PREPARED: 6/12/2014
POC: CHIEF, COST ENGINEERING, Jeremiah Frost

Civil Works Work Breakdown Structure		ESTIMATED COST (in $1000s)				PROJECT FIRST COST (Constant Dollar Basis) (in $1000s)				TOTAL PROJECT COST (FULLY FUNDED) (in $1000s)				
		Estimate Prepared:			6/9/2014	Program Year (Budget EC):			2015					
		Effective Price Level:			41913	Effective Price Level Date:			1 OCT 14					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	RD 17 Fix E													
ALL	COMPOSITE INDEX (WEIGHTED AVERAGE	$0	$0	16%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$11,298	$1,813	16%	$13,112	0.0%	$11,298	$1,813	$13,112	2014Q1	0.0%	$11,298	$1,813	$13,112
11	LEVEES & FLOODWALLS	$230,577	$37,008	16%	$267,584	0.0%	$230,577	$37,008	$267,584	2014Q1	0.0%	$230,577	$37,008	$267,584
15	FLOODWAY CONTROL & DIVERSION STRU	$0	$0	16%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
18	CULTURAL RESOURCE PRESERVATION	$6,139	$985	16%	$7,125	0.0%	$6,139	$985	$7,125	2014Q1	0.0%	$6,139	$985	$7,125
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$248,014	$39,806	16%	**$287,821**		$248,014	$39,806	$287,821			$248,014	$39,806	$287,821
01	LANDS AND DAMAGES	$14,485	$3,463	24%	$17,948	0.0%	$14,485	$3,463	$17,948	2014Q1	0.0%	$14,485	$3,463	$17,948
02	RELOCATIONS	$9,514	$1,527	16%	$11,041	0.0%	$9,514	$1,527	$11,041	2014Q1	0.0%	$9,514	$1,527	$11,041
30	RELOCATIIONS - PED	$1,427	$229	16%	$1,656	0.0%	$1,427	$229	$1,656	2014Q1	0.0%	$1,427	$229	$1,656
31	RELOCATIONS - CM	$951	$153	16%	$1,104	0.0%	$951	$153	$1,104	2014Q1	0.0%	$951	$153	$1,104
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
1.0%	Planning & Environmental Compliance	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
6.0%	Engineering & Design	$14,881	$2,388	16%	$17,269	0.0%	$14,881	$2,388	$17,269	2014Q1	0.0%	$14,881	$2,388	$17,269
0.5%	Reviews, ATRs, IEPRs, VE	$1,240	$199	16%	$1,439	0.0%	$1,240	$199	$1,439	2014Q1	0.0%	$1,240	$199	$1,439
0.5%	Life Cycle Updates (cost, schedule, risks)	$1,240	$199	16%	$1,439	0.0%	$1,240	$199	$1,439	2014Q1	0.0%	$1,240	$199	$1,439
1.0%	Contracting & Reprographics	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
3.0%	Engineering During Construction	$7,440	$1,194	16%	$8,634	0.0%	$7,440	$1,194	$8,634	2014Q1	0.0%	$7,440	$1,194	$8,634
1.0%	Planning During Construction	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
1.0%	Project Operations	$2,480	$398	16%	$2,878	0.0%	$2,480	$398	$2,878	2014Q1	0.0%	$2,480	$398	$2,878
31	CONSTRUCTION MANAGEMENT													
6.5%	Construction Management	$16,121	$2,587	16%	$18,708	0.0%	$16,121	$2,587	$18,708	2014Q1	0.0%	$16,121	$2,587	$18,708
1.5%	Project Operation:	$3,720	$597	16%	$4,317	0.0%	$3,720	$597	$4,317	2014Q1	0.0%	$3,720	$597	$4,317
2.0%	Project Management	$4,960	$796	16%	$5,756	0.0%	$4,960	$796	$5,756	2014Q1	0.0%	$4,960	$796	$5,756
	CONTRACT COST TOTALS:	$336,393	$55,130		**$391,523**		$336,393	$55,130	$391,523			$336,393	$55,130	**$391,523**

US Army Corps
of Engineers®

Sacramento District
Planning Division

Lower San Joaquin River Feasibility Report

San Joaquin County, California

ECONOMICS APPENDIX

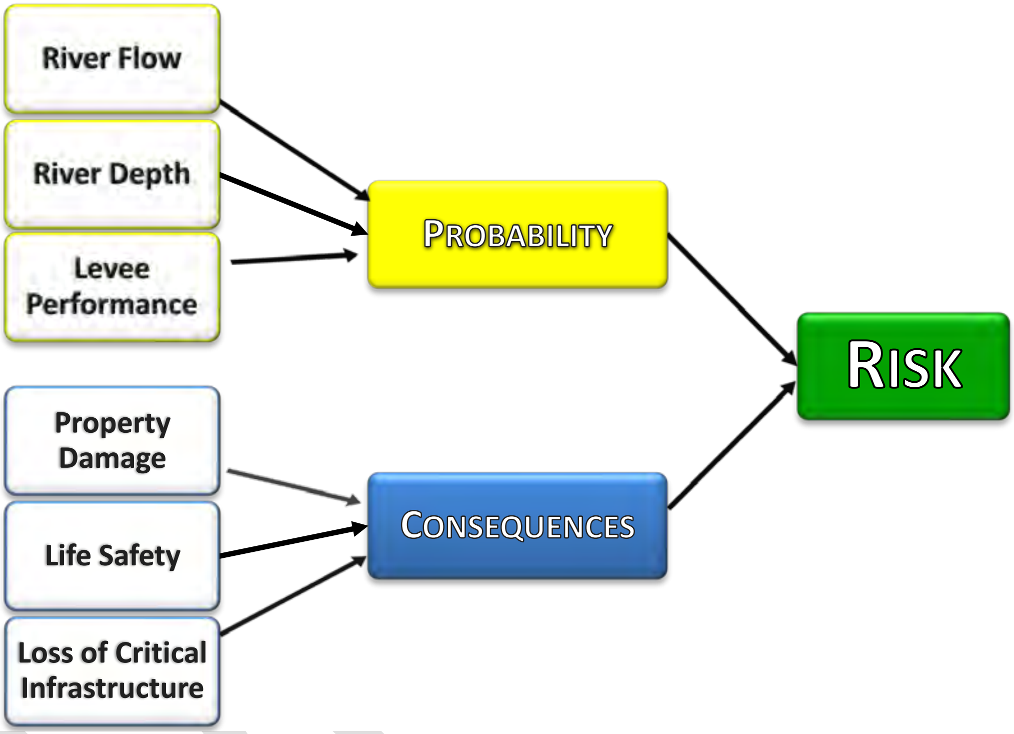
This page intentionally left blank

RISK ANALYSIS OVERVIEW

Risk is defined as the measure of the probabilities and consequences associated with uncertain future events. The objective of this economic analysis is to assess existing flood risk in the Lower San Joaquin River Basin and evaluate potential measures to reduce that risk.

The figure below provides a visual representation of the basic components driving the flood risk analysis summarized in this appendix. Each of these components will be described in detail in subsequent chapters.

COMPONENTS OF FLOOD RISK



Contents

Tables

Figures

Attachments

CHAPTER 1 — INTRODUCTION

1.1 PURPOSE & SCOPE

This Appendix documents the economic analysis conducted in support of the Lower San Joaquin River Feasibility Study (LSJRFS). The purposes of this report are:

- Describe major assumptions, data, methodologies, and tools used in the economic analysis
- Describe the flood risk associated with the without-project condition
- Describe the residual flood risk associated with each alternative.
- Summarize the net benefits and benefit-to-cost ratios of each alternative
- Identify the alternative that reasonably maximizes net benefits

1.2 BACKGROUND

The U.S. Army Corps of Engineers, together with the State of California San Joaquin Area Flood Control Agency (SJAFCA) conducted this feasibility study to select a plan that reduces flood risk. The goal of the study is to identify a cost effective, technically feasible and locally acceptable project that best reduces flood risk and complies with all Federal, State, and local laws and regulations.

The selected flood risk reduction plan may provide ancillary Ecosystem Restoration and Recreation Benefits in the study area. However, these benefits are not included in this economic analysis and will not be discussed further in this appendix.

1.3 HISTORY OF FLOODING

Major flooding has occurred in 1955, 1958, and 1997. The 1955 flood left roughly 1,500 acres of Stockton under six feet of water for as long as eight days. In 1958, approximately 8,500 acres were inundated with up to two feet of water between Bellota and the Diverting Canal with flood durations lasting up to 10 days. The 1997 flood resulted in the evacuation of the Weston Ranch area of Stockton in the northern portion of RD-17. While the 1997 event did not directly damage areas of Stockton, Lathrop, or Manteca, nearly 2,000 residences and businesses were affected in San Joaquin and Stanislaus Counties. The 1997 event caused an estimated $80 million in damage in San Joaquin County.

1.4 PROBLEMS AND OPPORTUNITIES

The purpose of this feasibility study is to recommend a reasonable and implementable plan to address problems and opportunities identified during the planning process. Please refer to Chapter Two of the Main Report for a complete account of the study's problems and opportunities. Brief descriptions of each problem and opportunity identified for the Lower San Joaquin study area are provided below.

PROBLEM — Flooding poses a significant risk to public safety, health, and property in the study area.

OPPORTUNITY — Reduce the risk of flooding from the Calaveras River, San Joaquin River, Mosher Slough, and the Sacramento-San Joaquin Delta.

OPPORTUNITY — Sustain and improve aquatic, riparian, and adjacent terrestrial habitats in conjunction with Flood Risk Management features.

OPPORTUNITY — Integrate a proposed project with other watershed-level initiatives for a holistic approach to flood risk management, ecosystem restoration, and navigation in the San Joaquin River watershed.

OPPORTUNITY — Expand current programs and to continue to educate the public about ongoing residual flood risk.

1.5 STUDY AREA

The Lower San Joaquin study area is located in San Joaquin County, California, approximately 50 miles south of Sacramento. The geographical extent of the economic analysis was established using inundation boundaries of the 0.2% annual chance exceedance (ACE) events from the flooding sources described in Section 1.6 This analysis includes roughly 80 square miles of urban and agricultural lands in the communities of Stockton, Lathrop and Manteca.

A map showing the location of the study area and its relative location within the state of California is shown in Figure 1-1 below. A map delineating urban and agricultural land use is shown in Figure 1-2.

FIGURE 1-1: STUDY AREA MAP

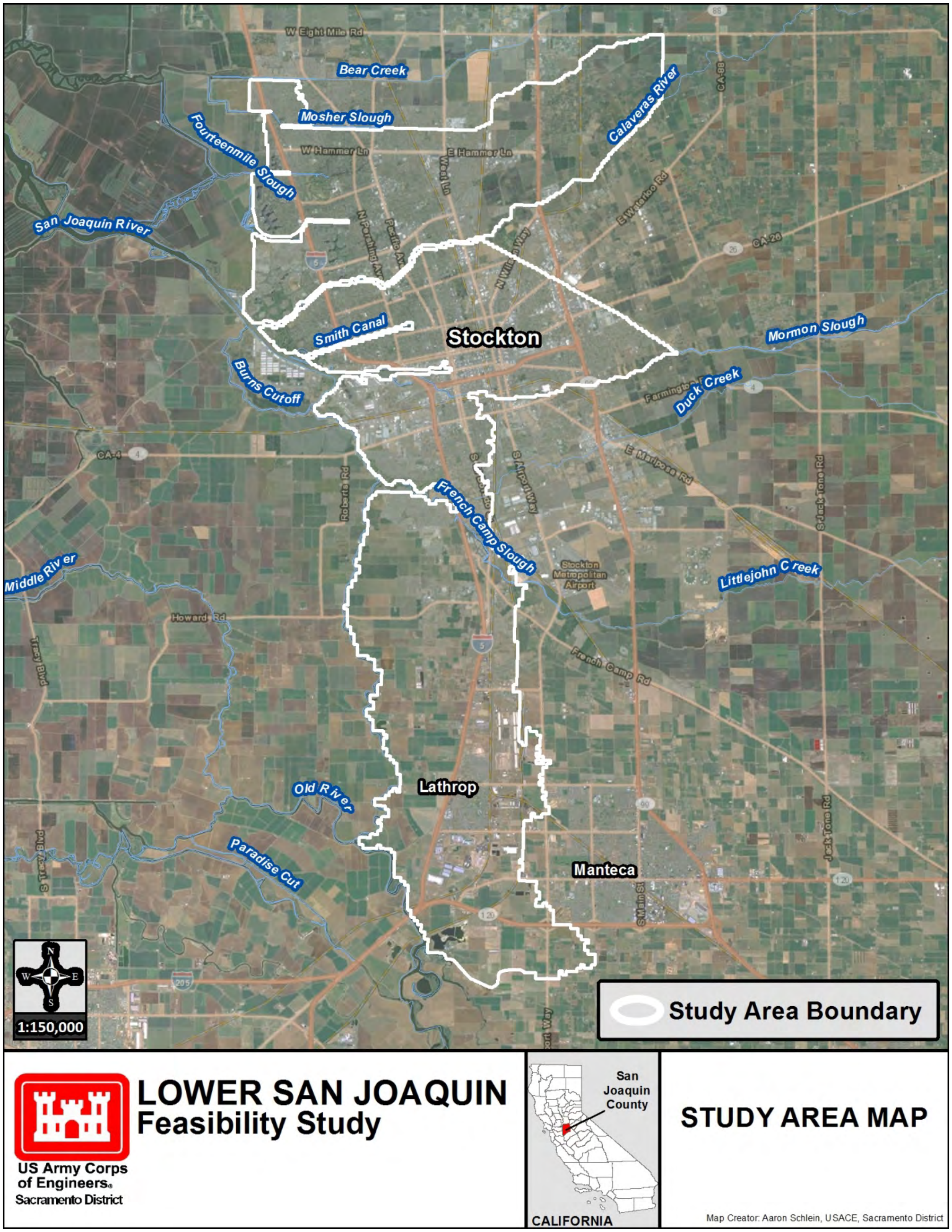
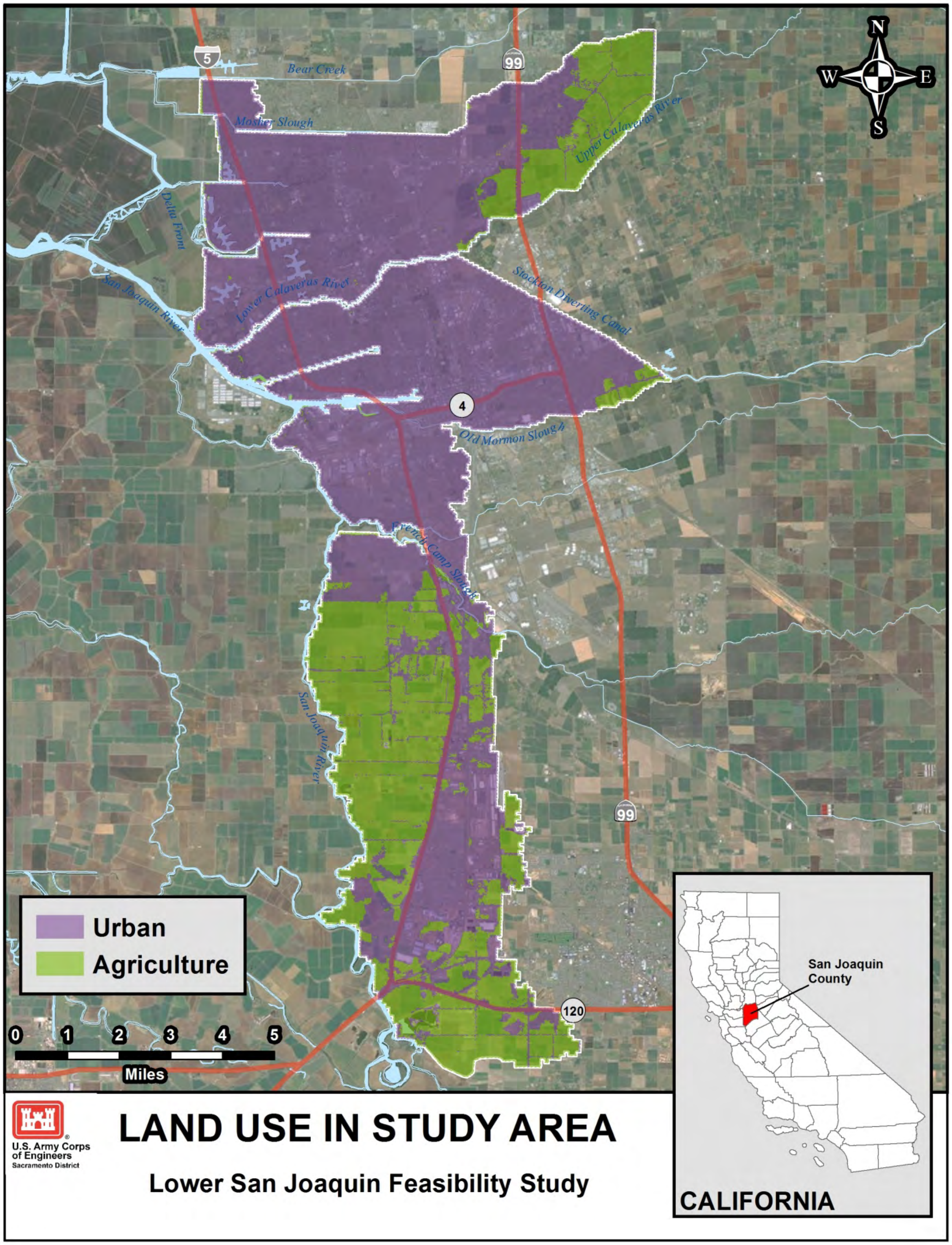


FIGURE 1-2: LAND USE MAP



1.6 SOURCES OF FLOODING

The study area is susceptible to comingled flooding from six principle sources including the Sacramento-San Joaquin Delta, San Joaquin River, Mosher Slough, Calaveras River system, French Camp Slough system, and interior sources. A complete description of each flood source within the study area can be found in Attachment 1.

1.7 RELATED FEDERAL FLOOD RISK MANAGEMENT PROJECTS

Development of water resources in the basin began in the 1850s and currently includes large multiple-purpose reservoirs, extensive levee and channel improvements, bypasses, and local diversion canals (USACE, 1993). Numerous agencies have been involved in water resources development within the study area. Some of these agencies include the USACE, Bureau of Reclamation, the State of California, county irrigation districts, local reclamation districts, and local levee districts.

The following two tables summarize existing Federal Flood Risk Management projects affecting the study area. Design flood flow projects are shown in Table 1-1, and dedicated federal flood storage projects are shown in Table 1-2. A detailed description of each project can be found in Attachment 2 of this appendix.

TABLE 1-1: PROJECT DESIGN FLOOD FLOWS

REACH	DESIGN FLOW (CFS)	DESIGN FREEBOARD (FT)	SOURCE
Mormon Slough			USACE, 1974
Bellota to Potter Creek	12,500	3 w/levee 1.5 w/o levee	USACE, 1974
Potter Creek to Diverting Canal	13,500	3 w/levee 1.5 w/o levee	USACE, 1974
Diverting Canal	13,500	3	USACE, 1974
Lower Calaveras River			
Diverting Canal to San Joaquin River	13,500	3	USACE, 1974
San Joaquin River			
Stanislaus River to Paradise Dam	52,000		USACE, 1993
Paradise Dam to Old River	37,000[1]		USACE, 1993
Old River to Stockton Deep Water Ship Channel	22,000		USACE, 1993
Duck Creek			
Duck Creek Diversion to Mariposa Road	700	Not Available	USACE, 1967
Mariposa Road to French Camp Slough	900	Not Available	USACE, 1967

---

[1] Design diversion capacity of Paradise Cut is 15,000 cfs

TABLE 1-2: PROJECTS WITH FEDERAL DEDICATED FLOOD STORAGE

RESERVOIR	YEAR CONSTRUCTED	GROSS POOL STORAGE (ACRE-FT)	DEDICATED FLOOD STORAGE (ACRE-FT)
Friant	1942	520,500	170,000
Big Dry Creek	1948	30,200	30,200
Farmington	1951	52,000	52,000
Comanche	1963	430,900	200,000
New Hogan	1963	317,100	165,000
Los Banos	1965	34,600	14,000
New Exchequer	1967	1,024,600	350,000
Don Pedro	1971	2,030,000	340,000
Buchanan	1975	150,000	45,000
Hidden	1975	90,000	65,000
New Melones	1979	2,400,000	450,000

1.8 SEPARABLE CONSEQUENCE AREAS

Flood risk in the study area was divided into three separable elements[1], or consequence areas, based on hydrologic and hydraulic characteristics with identifiable and distinct economic benefits. These Consequence areas are described below. A map of the Consequence area boundaries and existing levees is shown in Figure 1-3.

NORTH STOCKTON – The North Stockton area is defined by the right bank levees of the Calaveras River and the levees along the delta front traveling northward along Tenmile Slough, Fourteenmile Slough, crossing Fivemile Creek, and traveling north to tie into the Federal project levee across Mosher Slough at the Atlas Tract.

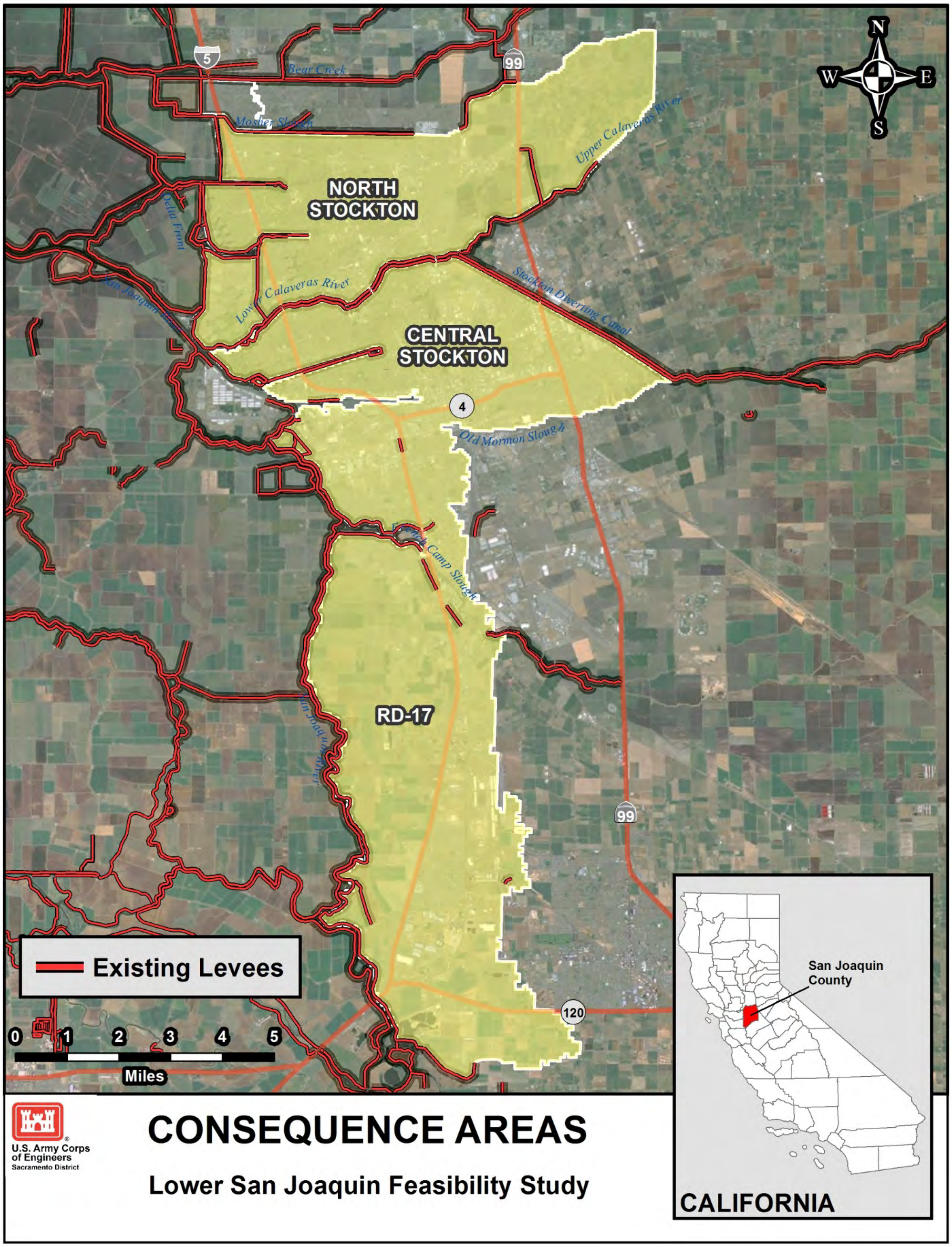
CENTRAL STOCKTON – The Central Stockton Area is defined by the left bank levees of the Stockton Diverting Canal, the left bank levees of the Calaveras River, the right bank levees of the San Joaquin River, and right bank levees of French Camp Slough.

RECLAMATION DISTRICT 17 (RD17) – The RD 17 area is defined by the levees along the right bank of the San Joaquin River, the left bank levees of French Camp Slough, and a dry-land levee at the upstream end of the reclamation district.

---

[1] “Separable element” is defined in 33 United States Code (U.S.C.) Section 2213(f) as a portion of the project that (1) is physically separable from other portions of the project; and (2)(a) achieves hydrologic effects, or (b) produces physical or economic benefits, which are separately identifiable from those produced by other portions of the project.

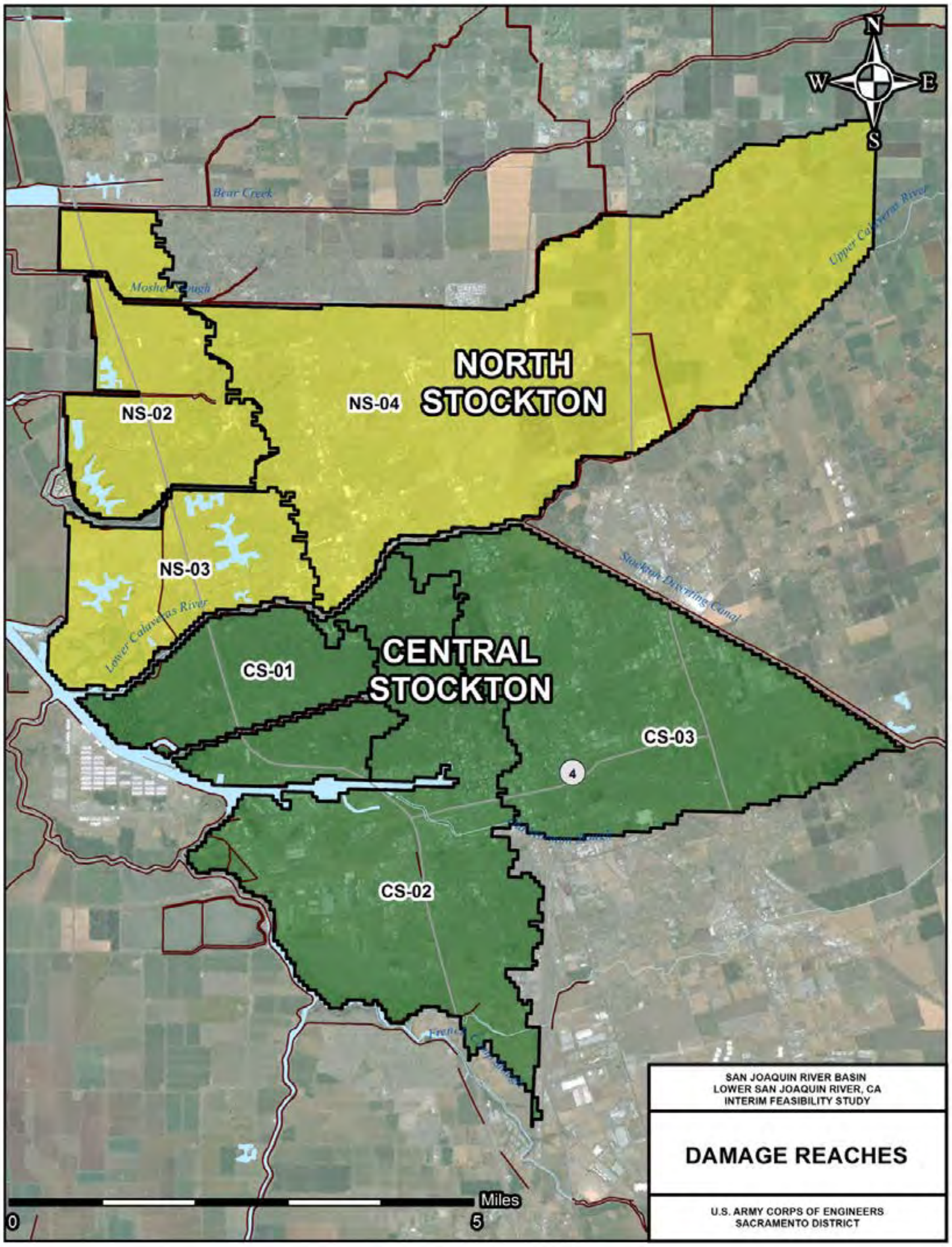
FIGURE 1-3: CONSEQUENCE AREAS



1.8.1 SUBDIVISION OF CONSEQUENCE AREAS

The North Stockton and Central Stockton consequence areas were subdivided for economic analysis purposes. Total damages for each consequence area is the sum of damages in each reach. A map of the subdivided areas is shown in Figure 1-4.

FIGURE 1-4: NORTH AND CENTRAL STOCKTON DAMAGE REACHES



1.9 POPULATION DATA

Population data for this study was obtained using a geographic information systems (GIS) layer containing 2010 census data by census block. This census data reports approximately 235,000 people residing within the study area in 2010. The population at risk by annual chance exceedance is shown in Table 1-3, and the population at risk due to levee overtopping is shown in Table 1-4. The disparity between the two tables illustrates the key role levee performance plays in safeguarding the population of the Lower San Joaquin River basin.

TABLE 1-3: POPULATION AT RISK BY ANNUAL CHANCE EXCEEDANCE

Damage Area	Population at Risk by ACE						
	0.5	**0.10**	**0.04**	**0.02**	**0.01**	**0.005**	**0.002**
NS-02	13,600	18,700	19,400	20,400	21,400	22,800	23,000
NS-03	11,900	16,100	16,700	18,400	18,500	18,800	18,800
NS-04	0	0	0	26,600	32,300	35,900	38,800
CS-01	14,300	19,000	19,900	22,000	22,600	22,900	23,100
CS-02	0	0	0	36,200	42,900	47,300	47,900
CS-03	0	0	0	24,900	28,500	31,000	38,800
RD17	0	0	25,800	38,200	43,600	44,600	44,600
Total	39,800	53,800	81,800	186,700	209,800	223,300	235,000

TABLE 1-4: POPULATION AT RISK DUE TO LEVEE OVERTOPPING

Damage Area	Population at Risk by Overtopping Event						
	0.5	**0.10**	**0.04**	**0.02**	**0.01**	**0.005**	**0.002**
NS-02	0	0	0	0	0	0	0
NS-03	0	0	0	0	0	0	0
NS-04	0	0	0	0	0	0	0
CS-01	0	0	0	0	0	0	23,100
CS-02	0	0	0	0	0	0	47,900
CS-03	0	0	0	0	0	0	0
RD17	0	0	0	0	0	0	44,600
Total	0	0	0	0	0	0	115,600

CHAPTER 2 — ECONOMIC ANALYSIS

2.1 CONSISTENCY WITH CURRENT REGULATIONS & POLICIES

The analysis presented in this document was performed using the most up-to-date guidance and is consistent with current regulations and policies. Various references were used to guide the economic analysis, including:

- The Planning Guidance Notebook (ER 1105-2-100, April 2000, with emphasis on Appendix D, Economic and Social Considerations, Amendment No. 1, June 2004) serves as the primary source for evaluation methods of flood risk management (FRM) studies
- EM 1110-2-1619, Engineering and Design – Risk-Based Analysis for Flood Damage Reduction Studies (August 1996)
- ER 1105-2-101, Planning Risk-Based Analysis for Flood Damage Reduction Studies (Revised January 2006)
- Economic Guidance Memorandum (EGM) 01-03, Generic Depth-Damage Relationships (2000)
- Economic Guidance Memorandum (EGM) 04-01, Generic Depth-Damage Relationships for Residential Structures with Basements (2003)
- Economic Guidance Memorandum (EGM) 09-04, Generic Depth-Damage Relationships for Vehicles (2009)

2.2 PRICE LEVEL, PERIOD OF ANALYSIS, AND DISCOUNT RATE

Values listed in this document are based on an October 2013 price level. Annualized benefits and costs were computed using a 50-year period of analysis and a current federal discount rate of 3.50%. Unless otherwise noted, annualized values are presented in thousands of dollars.

2.3 HYDROLOGIC, HYDRAULIC, AND GEOTECHNICAL DATA

Flood inundation was modeled for eight ACE events at each breach location using FLO-2D software. FLO-2D stores the resulting inundation data for each model using an overlay of uniform grid cells. For this analysis, the maximum water surface elevation at each grid cell was used as an input into HEC-FDA to represent the inundation depth at each structure located within that cell.

The probability of flooding at a given breach location is driven by the following engineering inputs:

UNREGULATED FLOW PROBABILITY — The relationship between natural (unregulated) river flow and the probability of that flow being exceeded

UNREGULATED TO REGULATED FLOW TRANSFORM — The relationship between natural flow and regulated flow resulting from reservoir routing, channel routing, or channel diversion.

DISCHARGE-STAGE RELATIONSHIP — The relationship between regulated flow and corresponding river depth (stage)

GEOTECHNICAL PERFORMANCE — The relationship between river depth and the probability of levee overtopping and/or failure at that depth

2.4 SIMPLIFYING ASSUMPTIONS

Several assumptions were relied upon in order to make best use of scarce resources to reasonably and efficiently identify existing flood risk and evaluate potential solutions.

2.4.1 BREACH LOCATIONS

Existing levees in the study area were divided into 14 levee reaches. Breach and inundation characteristics of each levee reach were modeled using a representative index point. The use of index points is policy compliant and is considered the most reasonable method to efficiently model flood risk over a large geographical area. Index points are summarized geographically from upstream to downstream in Table 2-1 below.

TABLE 2-1: INDEX POINTS BY FLOODING SOURCE

FLOOD SOURCE	INDEX POINT
SAN JOAQUIN RIVER	LRTB
	LR4
	LR3
	LR2
	LR1
FRENCH CAMP SLOUGH	FR1
	FL1
STOCKTON DIVERTING CANAL	SL2
CALAVERAS RIVER	CR2
	CL2
SACRAMENTO-SAN JOAQUIN DELTA FRONT	D3
	D4
	D5
	D-BS

2.4.2 MULTIPLE-SOURCE FLOODING

Throughout this study, multiple sources of flooding exist within a single consequence area, and each source comes with its own unique combination of probabilities and consequences. The simplifying assumption was made that the flood source with the highest economic risk is deemed the lone driver of both without-project and residual risk in each consequence area.

It is acknowledged that overall economic risk may be slightly underestimated, as the combined probabilities and consequences of multiple levee breaches within a single consequence area are not captured by the models. This assumption is considered low risk for two reasons: (1) underestimates of without-project risk are constant across all alternatives; and (2) the probability of multiple levee failures under with-project conditions are extremely low, which causes only negligible underestimates of residual risk.

Figures 2-1 through 2-7 provide a visual representation of the index points chosen for the study. Each figure each contains two graphics. The graphic on the left shows the location of all index points analyzed for a given damage area. The graphic on the right shows the highest risk index point for the damage area and includes an overlay of the flooding associated with a levee breach for each probability-flood event. Each index point label contains the annual exceedance probability (AEP) at the representative breach location. AEP is the likelihood that flooding will occur in a given year considering the probabilities associated with the full range of engineering inputs.

FIGURE 2-1: INDEX POINTS—NORTH STOCKTON 02

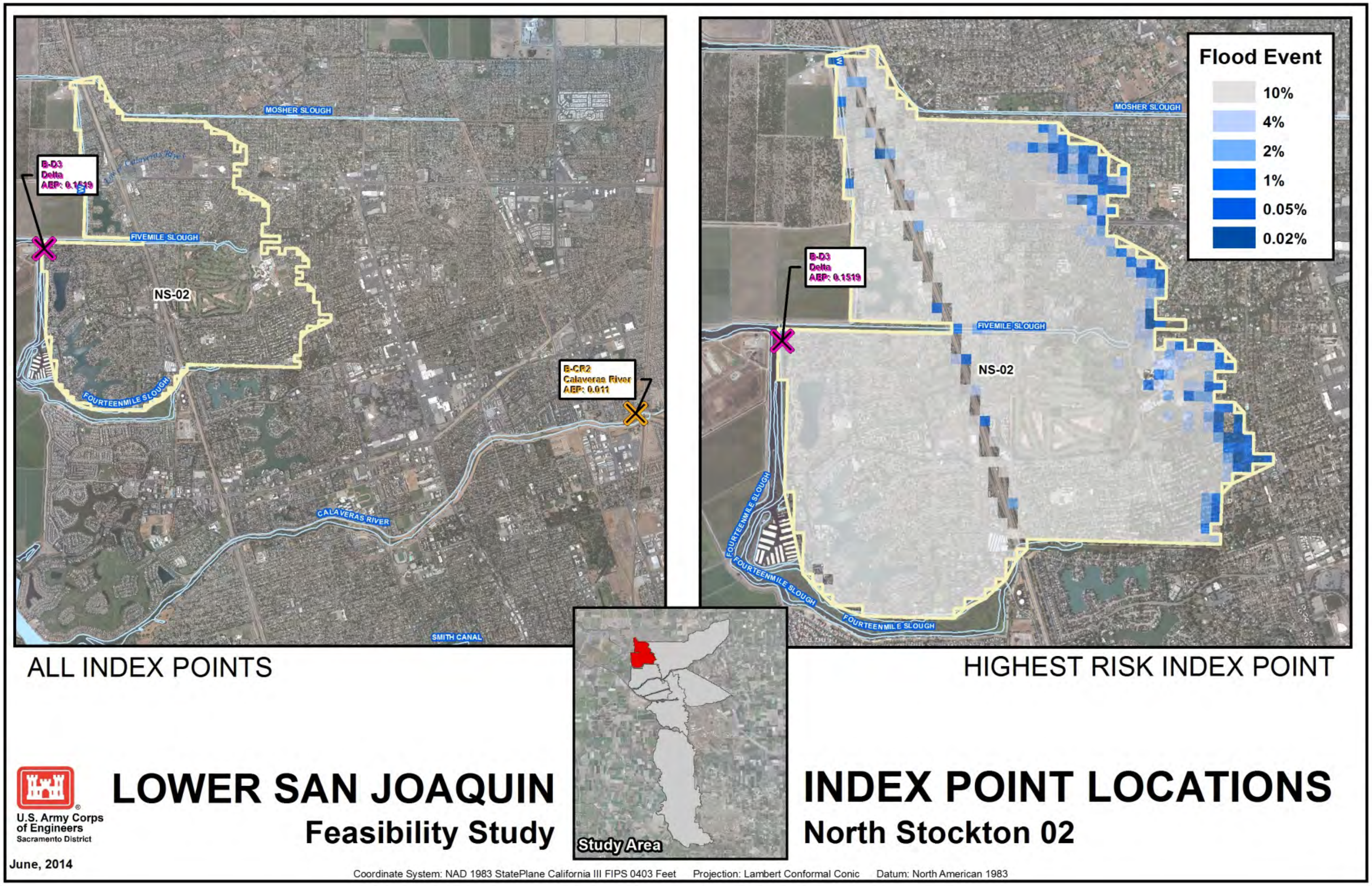


FIGURE 2-2: INDEX POINTS—NORTH STOCKTON 03

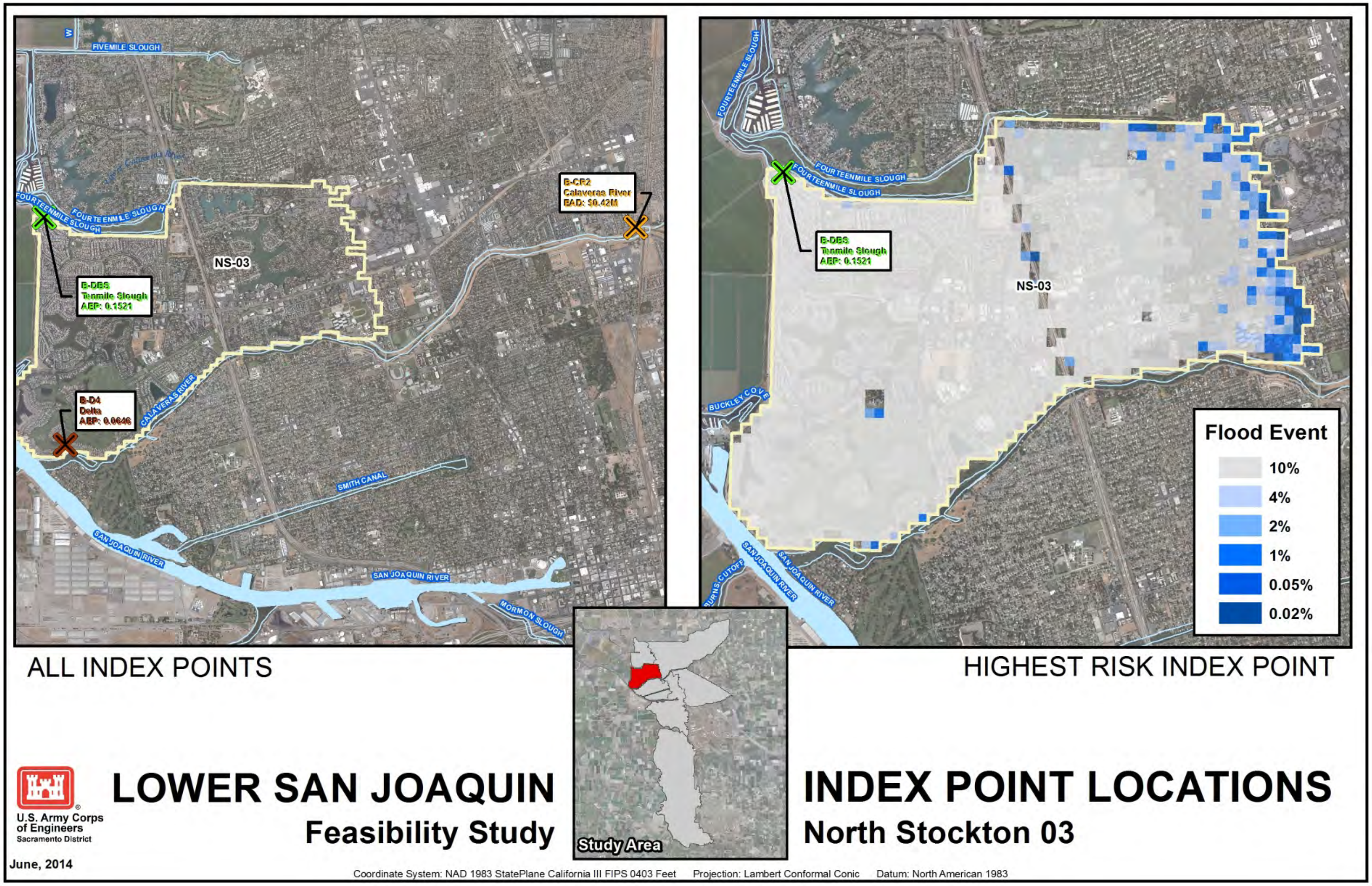


FIGURE 2-3: INDEX POINTS—NORTH STOCKTON 04

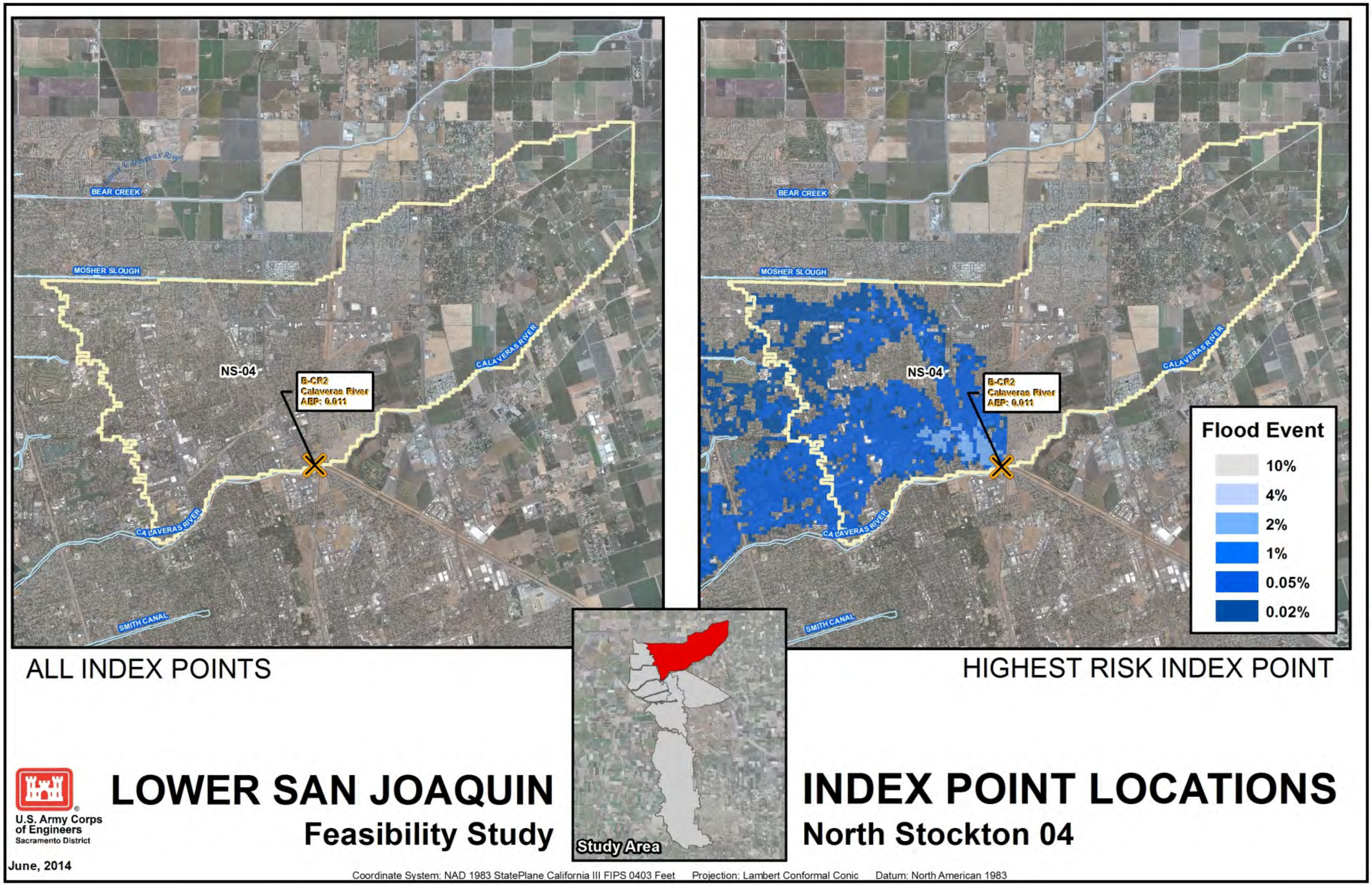


FIGURE 2-4: INDEX POINTS—CENTRAL STOCKTON 01

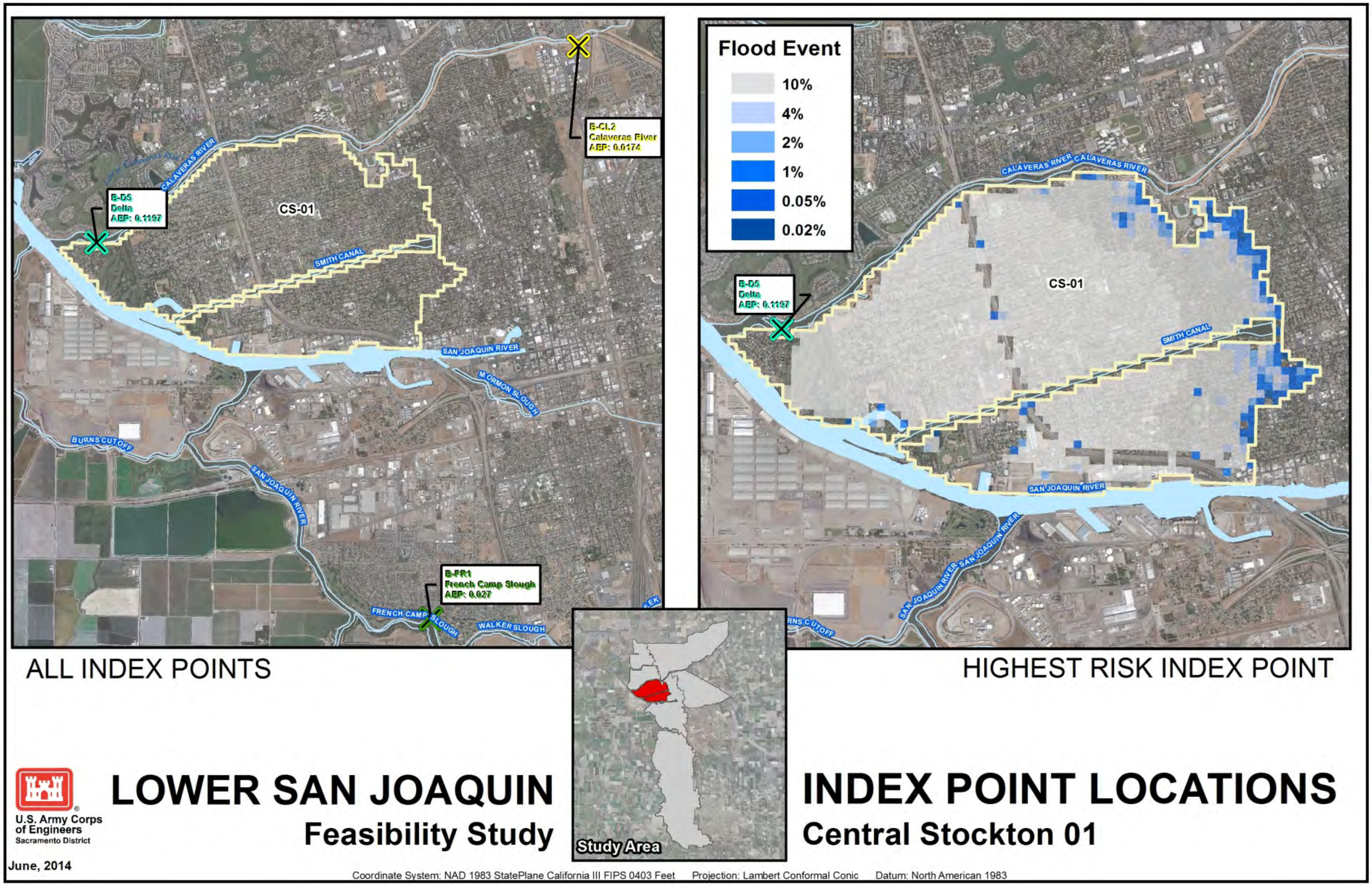


FIGURE 2-5: INDEX POINTS—CENTRAL STOCKTON 02

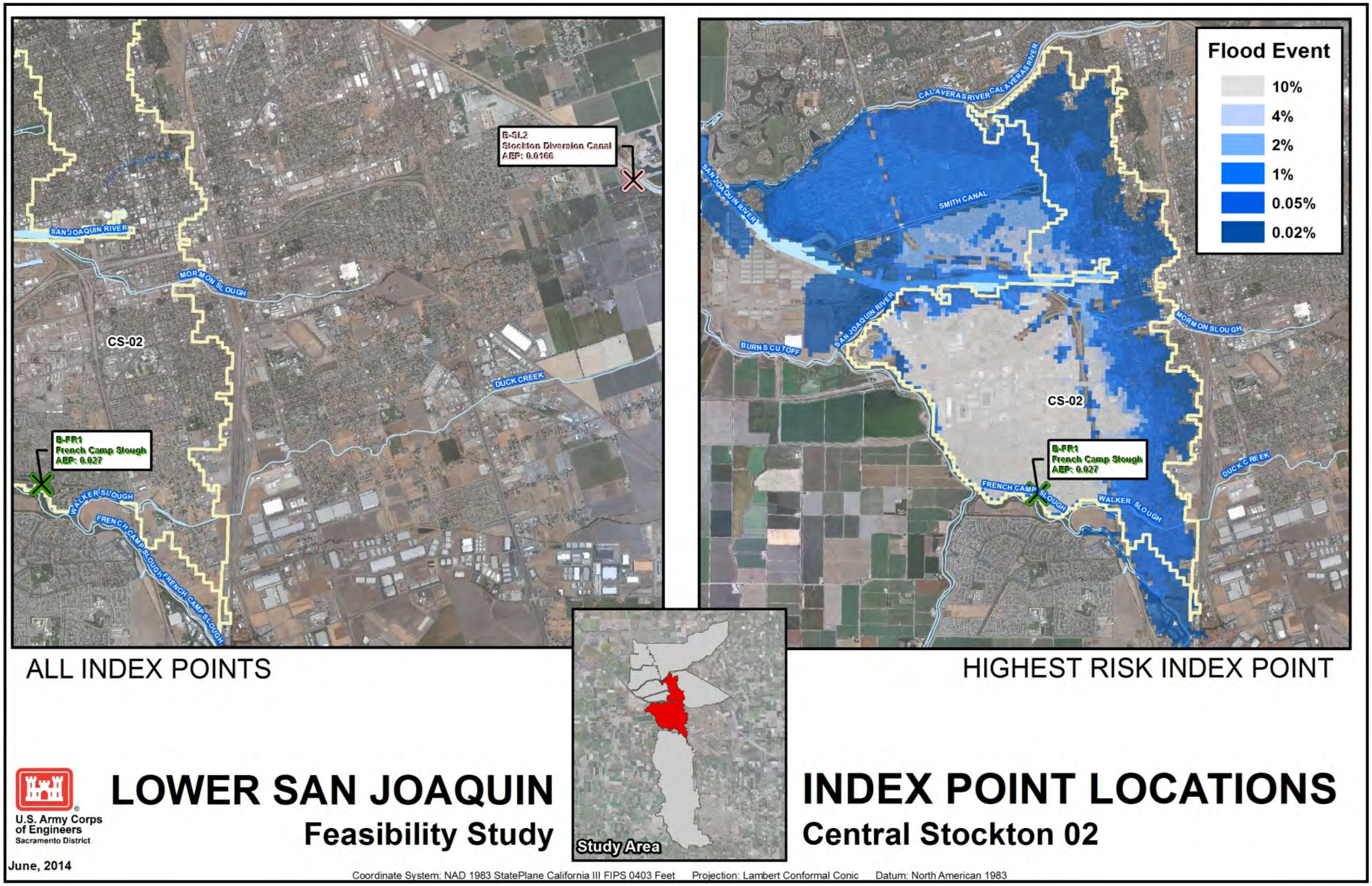


FIGURE 2-6: INDEX POINTS—CENTRAL STOCKTON 03

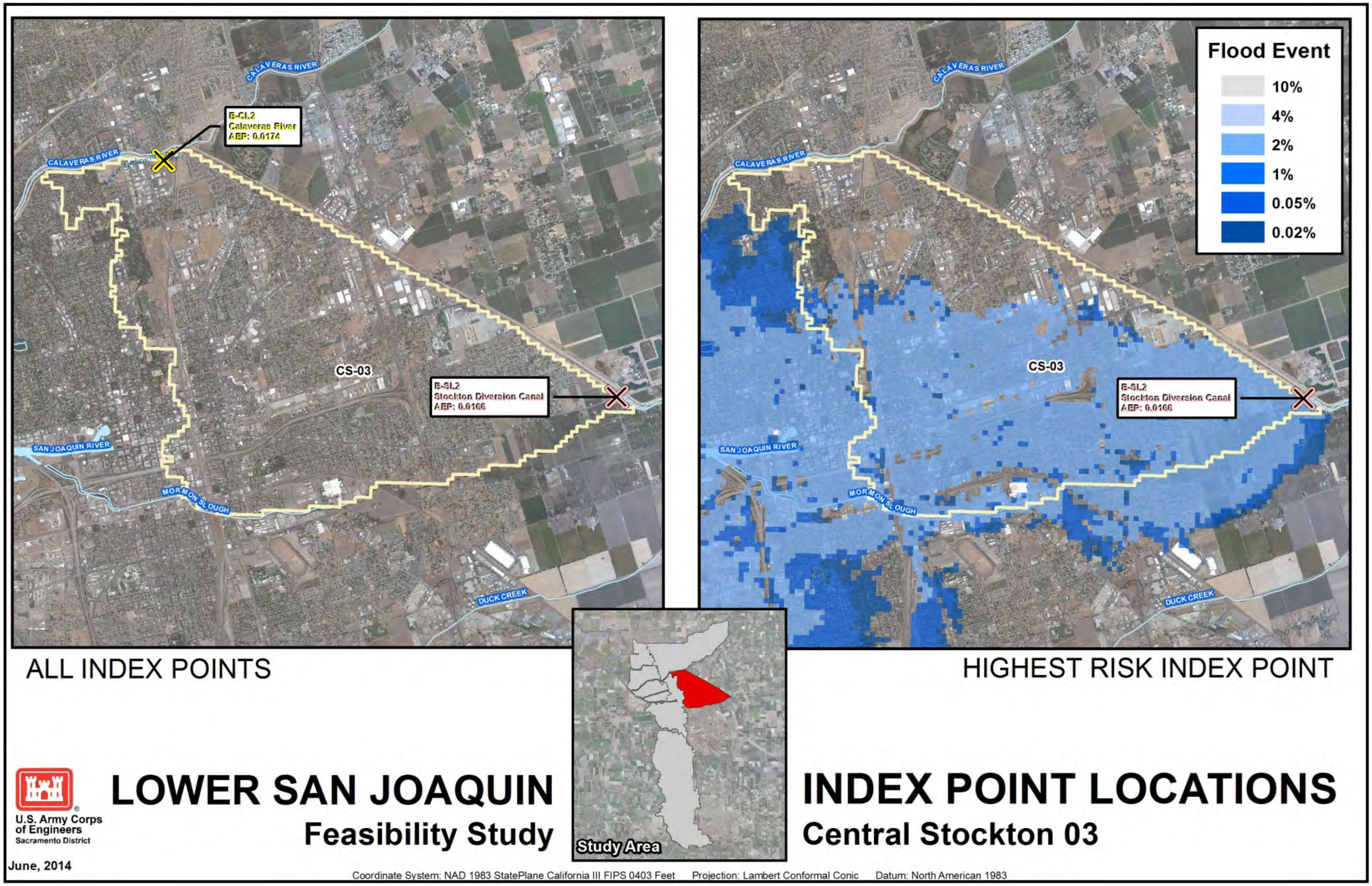
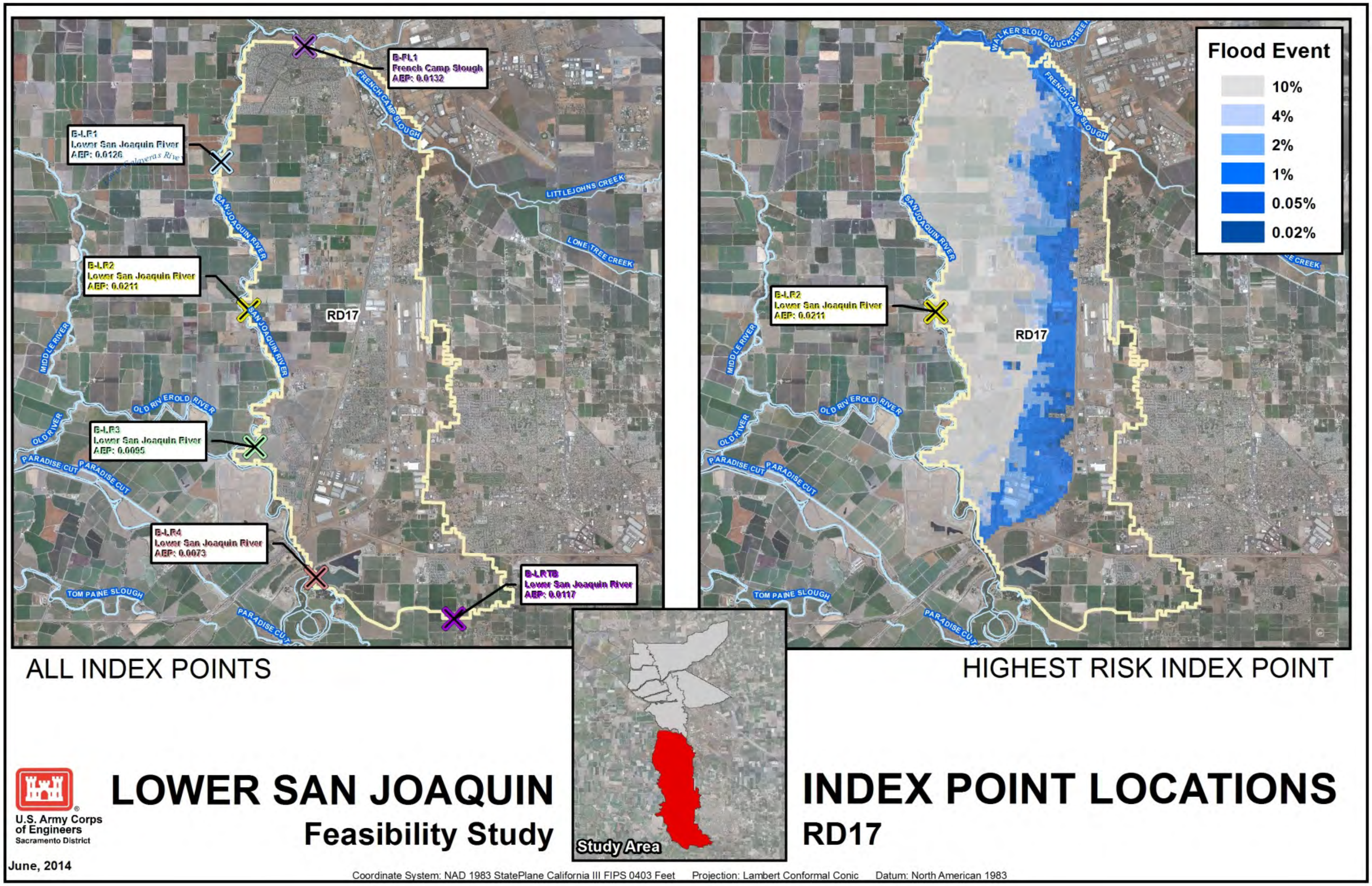


FIGURE 2-7: INDEX POINTS—RD17



2.4.3 FUTURE WITHOUT-PROJECT CONDITION—ECONOMICS

For this feasibility study analysis, the future without-project condition assumes no additional development in the study area. The basis of this assumption is that existing developable land is reasonably built out to its full potential. Additionally, development forecasts were not made for currently undeveloped portions of the study area. This is due to the uncertainty surrounding public policy decisions that may limit or prohibit such development.

2.4.4 SEA LEVEL RISE

Sea level rise is expected to impact stage-frequency at several breach locations in the study area. Hydraulic inputs for all alternatives use 2010 data to represent present-year conditions and forecasted data for the year 2070 to represent the future year. It is acknowledged that using 2010 data presents the risk of failing to capture sea level rise that may have already occurred. This risk is considered acceptable as the result is a slight underestimation of without-project damages and subsequent with-project benefits.

2.4.5 EQUIVALENT ANNUAL DAMAGES

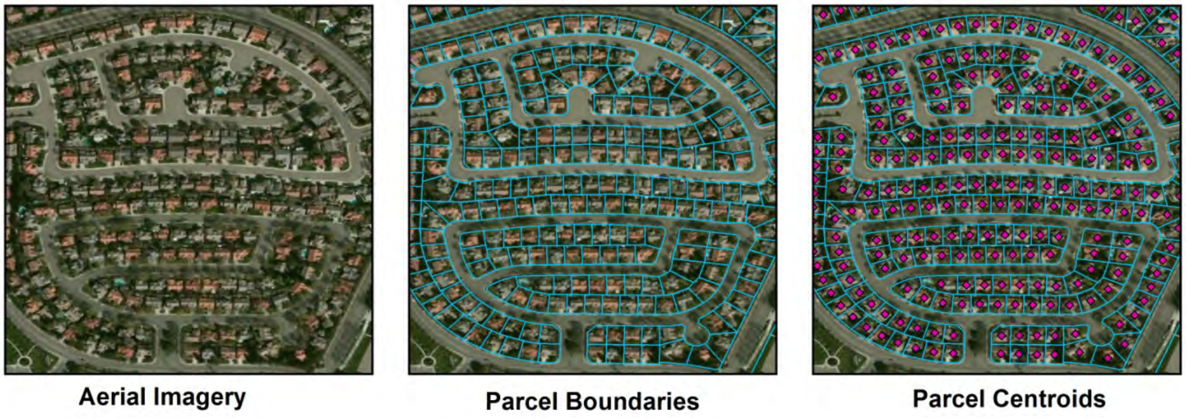
All annual damages in this appendix are reported in average annual equivalent terms. Because sea level rise is expected to lead to an upward shift in the stage-frequency relationship, higher probabilities of flooding are expected in the future, *ceteris paribus*. To capture the consequent increase in expected annual damages, a linear relationship between future damage values was assumed. Future damages are interpolated between the base and future year and discounted back to the base year.

2.4.6 STRUCTURE LOCATIONS

Structure locations were estimated using a geographic information system (GIS) parcel layer containing the boundaries of every parcel of land in the study area. The spatial accuracy of the data was confirmed using aerial imagery. The simplifying assumption was made that structures are to be located at the geometric center, or centroid, of the parcel they are located on. While it is possible to manually place each structure in its precise location using aerial imagery, doing so would provide little return on the resource investment such a task would require.

Figure 2-8 displays this structure placement process visually. It is important to note the location of the centroids in relation to the structures they represent. Any minor spatial discrepancies are believed to be low risk and are justified by the significant resource savings this method offers.

FIGURE 2-8: STRUCTURE PLACEMENT



2.5 STRUCTURE INVENTORY DATA

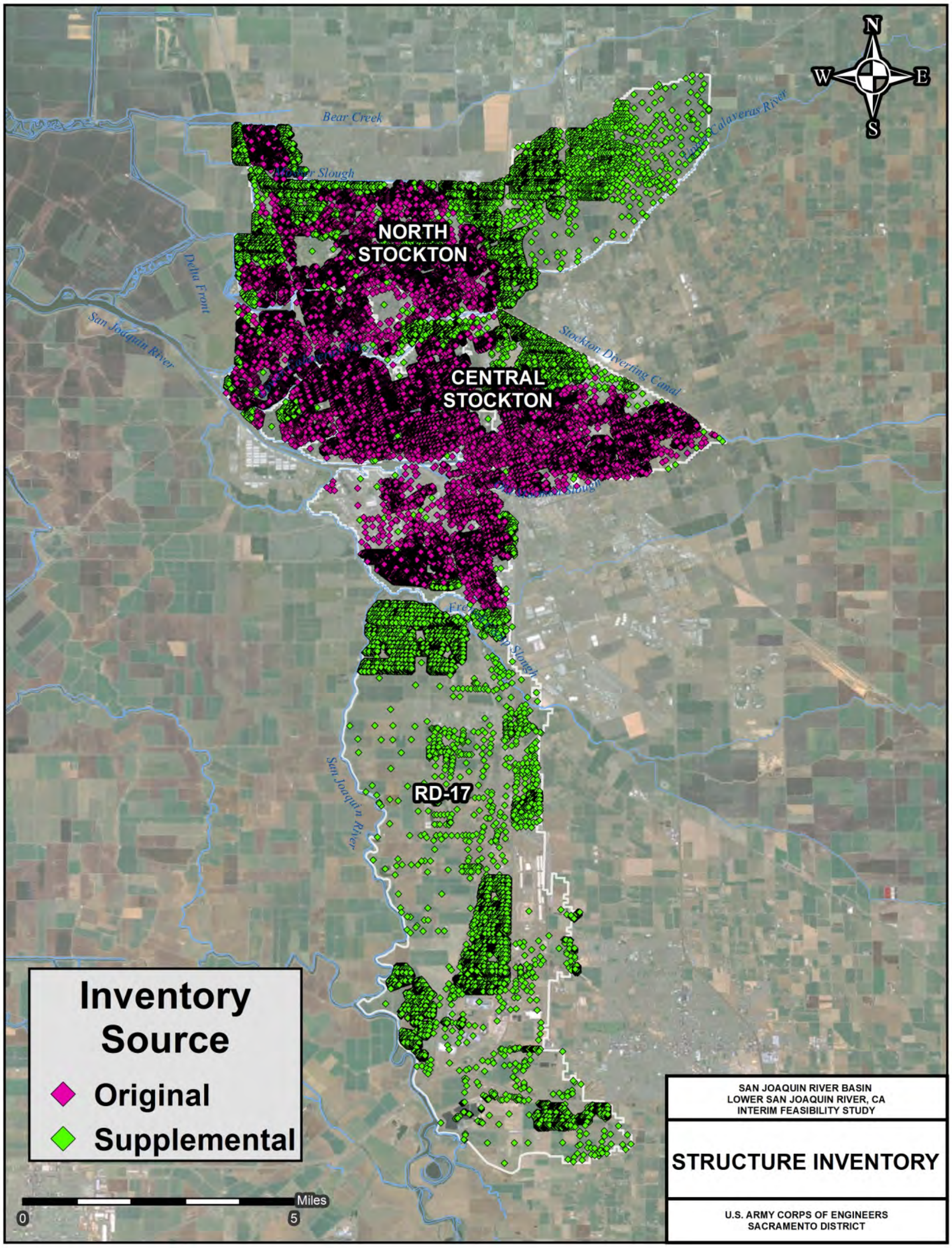
An inventory of damageable property was developed for the study in two parts. The first part was completed in 2011 by USACE Los Angeles District for use in the 2012 preliminary screening analysis. This inventory was based on San Joaquin County Assessor parcel data and included 51,856 structures and covered most of the North and Central Stockton consequence areas. The methodology used to develop the 2011 inventory is provided in Attachment 3 of this appendix.

The second part was developed in 2013 as a supplement to the existing inventory. This was critical to the study as the 2011 inventory did not include structures in RD17. Furthermore, a significant number of structures in North and Central Stockton were missing or inaccurately located. The supplementary inventory was also created using assessor parcel data.

The most notable difference between the two inventories is the valuation method. Structures in the original inventory were valuated using the depreciated replacement value method described in Attachment 3, while the supplemental structures use assessor improvement values. This was due to inadequate time and resources to conduct a proper field survey for the supplemental structures. Assessor improvement values account only for the cost of materials and labor needed to build a structure and do not include land values or trends in the real estate market.

A map of the structure inventory is shown in Figure 2-9. Note that structures from the two inventories are distinguished by color.

FIGURE 2-9: STRUCTURE INVENTORY



2.5.1 CONTENT-STRUCTURE VALUE RATIOS

The content to structure value ratio is the relationship between the value of a structure and the value of its contents. Content to structure value ratios are expressed as a percentage and are based on a structure's occupancy type. Content to structure ratios used in this study area shown in Table 2-2.

TABLE 2-2: CONTENT TO STRUCTURE RATIOS BY OCCUPANCY TYPE

DAMAGE CATEGORY	OCCUPANCY TYPE	CONTENT TO STRUCTURE RATIO
COMMERCIAL	Auto Sales	62%
	Auto Service	193%
	Fast Food Restaurant	42%
	Food Retail	42%
	Full Service Auto Dealership	69%
	Furniture Store -1 story	55%
	Furniture Store -2 story	36%
	General Retail	51%
	Grocery Store	106%
	Hospital - 1 story	92%
	Hospital - 2 story	87%
	Hotel	69%
	Medical - 1 story	148%
	Medical - 2 story	121%
	Office -1 story	34%
	Office -2 story	28%
	Restaurants - 1 story	134%
	Restaurants - 2 story	118%
	Shopping Center - 1 story	67%
	Shopping Center - 2 story	54%
INDUSTRIAL	Heavy Manufacturing - 1 story	31%
	Heavy Manufacturing - 2 story	20%
	Light Manufacturing - 1 story	188%
	Light Manufacturing - 2 story	126%
	Warehouse - 1 story	89%
	Warehouse - 2 story	85%
PUBLIC	Church - 1 story	20%
	Church - 2 story	17%
	Government Building - 1 story	35%
	Government Building - 2 story	26%
	Recreation/Assembly - 1 story	132%
	Recreation/Assembly - 2 story	58%
	School - 1 story	38%
	School - 2 story	32%
RESIDENTIAL	Mobile Home	50%
	Multi-Family Residence	100%
	Single Family Residence	100%

2.6 RISK AND UNCERTAINTY

Uncertainty is especially prevalent in the estimation of flood risk. A list of all the potential sources of uncertainty would be nearly endless. However, primary sources of uncertainty evaluated in this study include: (1) Levels of Storm Water Discharge; (2) Water Surface Elevations; (3) Levee Performance; (4) Depreciated Structure and Structure Content Values; and (5) Flood Damages to Structures and Structure Contents. The section below describes these sources of uncertainty and how each is accounted for in this analysis.

LEVELS OF STORM WATER DISCHARGE – Uncertainty in the level of rainwater discharge associated with a storm event with a given probability of occurrence is driven by a number of inconsistent factors. Storms with equal probabilities of occurrence can differ in the amount of rainfall they produce at various locations throughout the watershed. They can also differ in their intensity, the time that elapses while rain is falling. Ground permeability, soil moisture, ambient temperature and other physical factors at the time of the storm also play an important role in determining when and where rainwater enters the river's channel. All of these natural factors lead to variability in the level of discharge found at a particular location along the river, following any given storm event.

WATER SURFACE ELEVATION – For a given level of discharge, there is uncertainty in the expected water surface elevations for specific locations within the channel. The shape of the riverbed, water temperature, location and amount of debris as well as other obstructions in the channel all add uncertainty to the estimated water surface elevations associated with storms of otherwise equal levels of discharge. To address this uncertainty, engineering data inputs were used to estimate standard deviations for various river stages. These estimated standard deviations are based on level of discharge and location in the floodplain.

LEVEE PERFORMANCE – For a given water surface elevation, there is uncertainty in the ability of the levees and banks to contain flood flows without structural failure. For this report, existing levees and those constructed as part of the SARM project were not assumed to fail prior to being overtopped. Levee and bank elevations were entered into the computer program described in the computer aided analysis section below, to ensure flooding was explicitly limited to those events in which the water surface elevation exceeds the top of bank/levee height.

STRUCTURE ELEVATIONS – The susceptibility of a structure to damage depends on a number of uncertain variables. One key variable, the structure elevation, can be decomposed into two error prone estimates: topographic and first floor elevations. The level of uncertainty in structures' topographic elevations is a function of the accuracy of data used to derive ground elevations. For example, elevation estimates derived from examining a five-foot contour map are likely to contain more error, and therefore have higher levels of uncertainty, than estimates derived using a two-foot aerial survey contour map. The second source of uncertainty in elevation data is the result of error in first floor or foundation height estimates. Foundation height data is important since structures built on land mounds or those with large crawl spaces may sustain little or no damage during floods that inundate surrounding areas and nearby properties. First floor height data error varies according to the precision of the method used to measure foundation heights. In practice, these methods range from best-guess estimates to windshield and professional surveys.

DEPRECIATED STRUCTURE AND CONTENT REPLACEMENT VALUES – The magnitude of damages to a particular structure following a given flood event is a function of its current, depreciated replacement value and the value of its contents. The current or depreciated value of a structure is uncertain for several reasons. First, per square foot structure values are calculated by estimating the construction type, quality and condition of structures during field surveys. These estimates are subject to human error associated with incorrectly classifying a structure within each category. The type, construction quality and condition classifications themselves may further induce error if they do not adequately account for the proper range of possible per square foot values. Further detail on structure valuation for this study can be found in Attachment 3.

FLOOD DAMAGES TO STRUCTURES AND STRUCTURE CONTENTS – Finally, there is considerable uncertainty in evaluating structure and content damages that would occur given a particular level of flooding. The value of damage to non-residential structures' contents was estimated using a method developed during an expert-opinion elicitation process, conducted by the Sacramento District USACE and published in Technical Report: Content Valuation and Depth Damage Curves for Nonresidential Structures, May 2007. Using this methodology, the structure's use (retail, agricultural, residential, etc…) and depreciation is correlated with the value of its contents. Damages to these contents during a hypothetical flooding event are then estimated using depth-damage functions published in the report. Residential structures' content values and damages were evaluated using depth-damage functions and associated standard error estimates developed by the IWR. Hypothetical damages to residential and non-residential structures during various flood events were also evaluated using IWR depth-damage curves. These depth-damage functions and standard error estimates are based upon the damages that actually occurred during previous flood events in the United States.

2.7 HEC-FDA SOFTWARE

The primary analytical tool used to perform the economic analysis was the Hydrologic Engineering Center's Flood Damage Analysis (HEC-FDA) software, version 1.2.5a. This program uses engineering and economic data to model flood risk with uncertainty and evaluate potential solutions in the study area.

By relating the economic inventory data to floodplain data, HEC-FDA computes economic stage-damage curves. Through integration of stage-damage curves and the engineering variables described in Section 2.3, HEC-FDA computes project performance statistics and expected annual damages.

The figure below demonstrates how risk and uncertainty parameters are utilized by HEC-FDA to develop point estimates used in Monte Carlo simulations. In step one, a frequency-discharge function with risk and uncertainty parameters is entered into HEC-FDA. This frequency-discharge function relates storm events with a given probability of occurrence in any given year to storm discharge flows. The solid black line, next to number one in the figure below, represents the expected values of this function;[1] the dotted black lines represent risk and uncertainty parameters entered into HEC-FDA.[2] These risk and uncertainty parameters at various points along the graphed line form the foundation of probability distribution functions, like the one shown to the right of point one.[3] Within a single iteration of a Monte Carlo simulation, the HEC-FDA program first selects a probabilistic event. Given an event with the

---

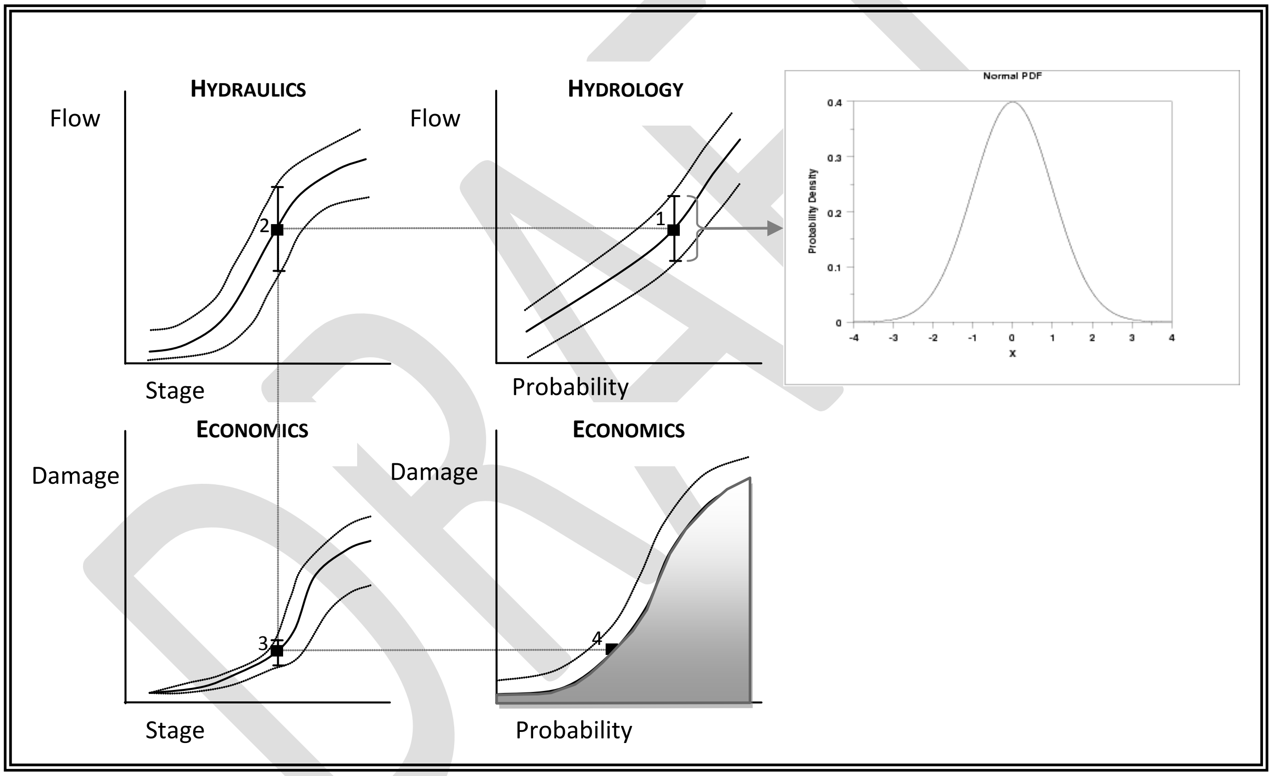
[1] In other words, the "most likely" level of storm discharge resulting from a storm event with a particular probability of occurrence (i.e. 1 percent ACE)

[2] For instance, a 95 percent confidence interval, indicating the range of storm discharge flows that 95 percent of 1 percent ACE would generate.

[3] In this case, the probability distribution function assigns probabilistic values to each potential storm discharge flow that could result from a particular storm event (i.e. the 1 percent ACE).

probability of occurrence represented by point one, a storm flow is drawn from this event's storm flow probability distribution function. The possible storm flow values for this probability event are symbolized with the candlesticks above and below point one in the figure below. Next, the storm discharge is linked to a river stage via a stage-discharge function, entered into HEC-FDA with risk and uncertainty parameters. Again, uncertainty parameters characterize probability distribution functions along the stage-discharge function, graphed about point two. In step three, damages are associated with the river stage *selected* in step two, via a third probability function.[4] This damage estimate, generated within a single Monte Carlo iteration is represented by point four along the cumulative distribution function below, which relates damages to storm events with a particular probability of occurrence in any given year. The damage results, produced in successive iterations of the Monte Carlo process, complete the cumulative distribution function and provide expected annual damage values with uncertainty.

FIGURE 2-10: DAMAGE ANALYSIS IN HEC-FDA WITH MONTE CARLO SIMULATIONS



2.8 PROJECT BENEFIT CALCULATION

Benefits for each alternative are based on the reduction in economic damages as compared to the without-project condition.

---

[4] Levels of storm discharge, river stage and damages are *selected* in the sense that they are drawn at random from a probability distribution function.

The benefits of all alternatives are based on a 50-year period of analysis beginning the year that a federal project would likely be completed. It is possible that differing construction schedules will result in varying base years among the alternatives.

CHAPTER 3 — EXISTING CONDITIONS ANALYSIS

3.1 CONSEQUENCE VARIABLES

Consequences in this study are defined as property damage, life-loss, and loss of critical infrastructure due to levee breach for a given annual chance exceedance (ACE) event. The variables that factor into consequence estimation are described in the following sections.

3.1.1 STRUCTURES AND CONTENTS

Structures were categorized by land use and classified as residential, commercial, industrial, or public. Structure counts by land use and consequence area are shown in Table 3-1 below. The total value of structures, contents, and automobiles within the Lower San Joaquin study area is estimated at $25 billion. Structure and content values by consequence area and occupancy type are summarized in Table 3-2.

TABLE 3-1: STRUCTURES IN THE 0.2% ACE FLOODPLAIN

CONSEQUENCE AREA	NUMBER OF STRUCTURES				
	COMMERCIAL	INDUSTRIAL	PUBLIC	RESIDENTIAL	TOTAL
North Stockton	1,273	68	113	32,322	33,776
Central Stockton	1,593	605	360	30,843	33,401
RD 17	253	238	50	12,147	12,688
Total	3,119	911	523	75,312	79,865

TABLE 3-2: VALUE OF DAMAGEABLE PROPERTY

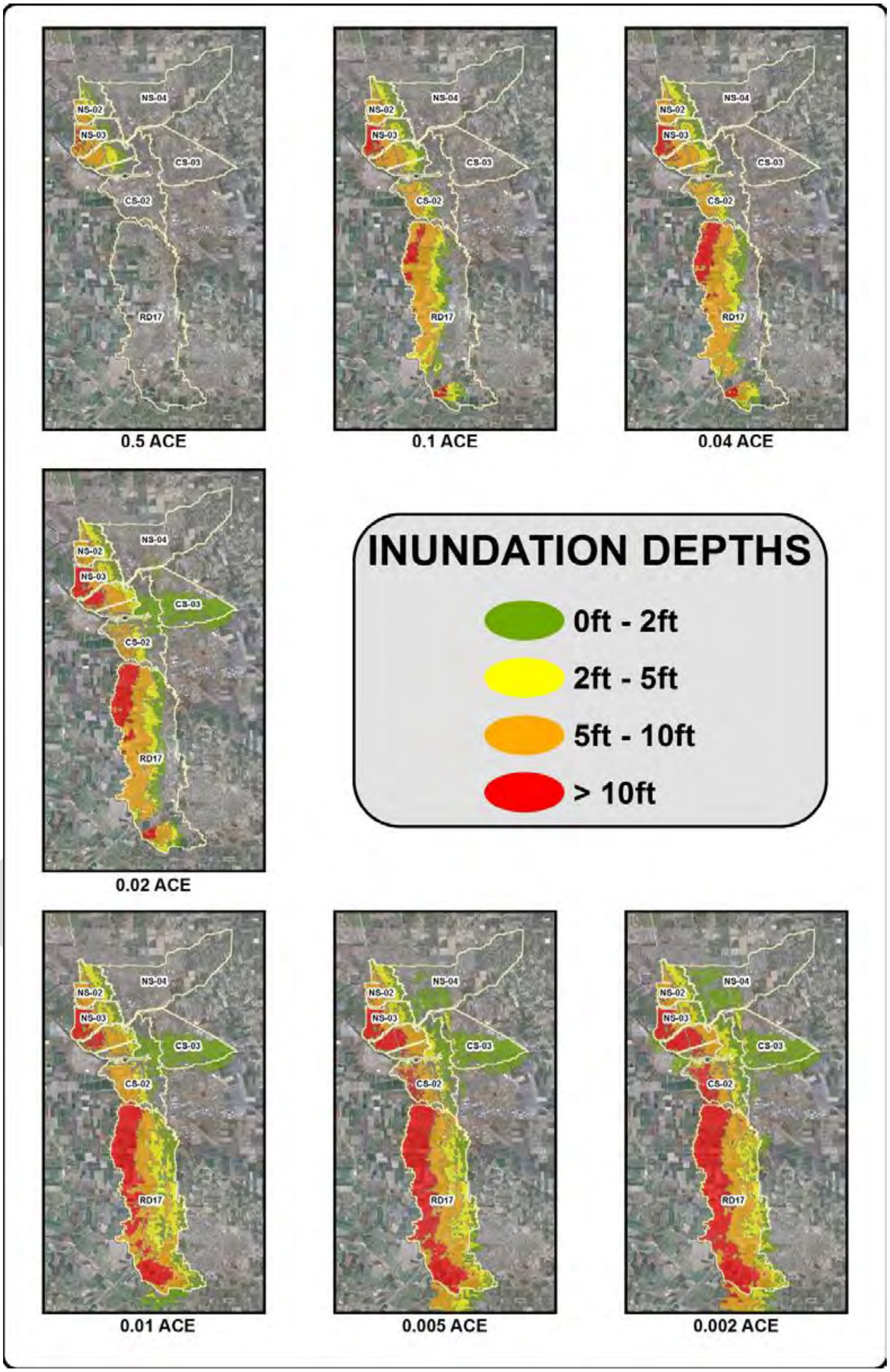
CONSEQUENCE AREA	STRUCTURE AND CONTENT VALUES					
	AUTOS	COMMERCIAL	INDUSTRIAL	PUBLIC	RESIDENTIAL	TOTAL
North Stockton	384,000	2,158,000	107,000	391,000	8,220,000	11,260,000
Central Stockton	301,000	1,751,000	1,784,000	729,000	3,976,000	8,541,000
RD 17	110,000	290,000	1,803,000	104,000	2,944,000	5,251,000
Total	795,000	4,199,000	3,694,000	1,224,000	15,140,000	25,052,000

3.1.2 DEPTH OF FLOODING

As discussed in section 2.3, hydraulic models estimate the flooding depths that would occur following a levee breach for a given ACE event. The results of these models are used to estimate single-event consequences of a levee failure and do not account for the probability of the breach actually occurring. Flood depths are a critical component of consequence estimation, as there is a positive correlation between depth of flooding and property damage, life-loss, and loss of critical infrastructure. Please refer to Chapter 3 of the hydraulic design appendix for an in-depth description of potential flooding conditions.

Figure 3-1 contains inundation maps with corresponding depths for each ACE event in the study area.

FIGURE 3-1: EXISTING CONDITION INUNDATION MAPS BY ACE EVENT



3.1.3 DEPTH-PERCENT DAMAGE FUNCTIONS

Depth-percent damage functions represent the relationship between inundation depth at a structure and the percentage of damage caused by that depth. Economic damage is calculated as a percentage of damage specified by the depth-percent damage function multiplied by the total value of structure and contents. Depth-percent damage functions for structures and contents by occupancy type can be found in Attachment 4.

3.1.4 SINGLE EVENT DAMAGES

Single-event damages are the total damages resulting from a levee breach during a given ACE event. Single-event damages lie solely on the consequences side of the risk equation, as none of the variables driving flood probability are considered. Single-event damages were calculated for the 0.5, 0.1, 0.04, 0.02, 0.01, 0.005 and 0.002 ACE flood events using the HEC-FDA model.

TABLE 3-3: SINGLE-EVENT DAMAGES—NORTH STOCKTON 02—INDEX POINT CR2

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	0	0	0	0	0	13	2,558
Residential	0	0	0	0	0	0	3,447	60,551
Public	0	0	0	0	0	0	0	125
Industrial	0	0	0	0	0	0	0	558
Commercial	0	0	0	0	0	0	35	2,860
TOTAL	0	0	0	0	0	0	3,495	66,652

TABLE 3-4: SINGLE-EVENT DAMAGES—NORTH STOCKTON 02—INDEX POINT D3

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	44,576	63,824	66,901	68,382	69,816	82,547	84,287
Residential	0	311,222	483,803	512,504	527,899	542,980	665,389	695,458
Public	0	7,168	19,081	20,375	20,919	22,761	35,910	32,064
Industrial	0	3,911	7,133	7,558	7,761	7,958	9,979	10,773
Commercial	0	23,715	59,346	68,273	73,124	77,363	101,605	104,974
TOTAL	0	390,593	633,188	675,610	698,086	720,878	895,430	927,555

TABLE 3-5: SINGLE-EVENT DAMAGES—NORTH STOCKTON 03—INDEX POINT CR2

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	0	0	0	0	0	5,082	16,621
Residential	0	0	0	0	0	2,788	146,403	296,136
Public	0	0	0	0	0	0	3,474	9,323
Industrial	0	0	0	0	0	0	0	1,189
Commercial	0	0	0	0	0	0	44,056	72,551
TOTAL	0	0	0	0	0	2,788	199,015	395,820

TABLE 3-6: SINGLE-EVENT DAMAGES—NORTH STOCKTON 03—INDEX POINT D4

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	30,412	43,370	45,716	46,922	48,126	49,399	50,678
Residential	2,788	489,094	643,334	669,529	684,099	696,674	709,927	722,847
Public	0	12,967	18,191	18,423	18,583	18,721	20,841	24,322
Industrial	0	3,057	3,102	3,109	3,112	3,114	3,117	3,120
Commercial	0	128,753	188,878	195,702	199,042	202,937	206,799	210,673
TOTAL	2,788	664,283	896,874	932,479	951,758	969,572	990,083	1,011,640

TABLE 3-7: SINGLE-EVENT DAMAGES—NORTH STOCKTON 03—INDEX POINT D-BS

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	0	0	0	0	0	59,269	61,810
Residential	0	0	0	0	0	2,788	808,034	839,206
Public	0	0	0	0	0	0	31,875	33,241
Industrial	0	0	0	0	0	0	3,139	3,146
Commercial	0	0	0	0	0	0	231,713	238,356
TOTAL	0	0	0	0	0	2,788	1,134,030	1,175,759

Table 3-8: Single-Event Damages—North Stockton 04—Index Point CR2

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	0	0	0	0	298	3,728	6,316
Residential	0	0	0	0	0	11,643	121,614	191,283
Public	0	0	0	0	0	221	2,090	5,021
Industrial	0	0	0	0	0	1,076	4,755	7,968
Commercial	0	0	0	0	0	4,866	65,561	96,570
TOTAL	0	0	0	0	0	18,104	197,748	307,159

Table 3-9: Single-Event Damages—Central Stockton 01—Index Point CL2

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	0	0	0	0	0	3,333	15,981
Residential	0	0	0	0	0	0	101,268	224,512
Public	0	0	0	0	0	0	1,616	10,804
Industrial	0	0	0	0	0	0	0	23
Commercial	0	0	0	0	0	0	3,856	20,078
TOTAL	0	0	0	0	0	0	110,074	271,397

Table 3-10: Single-Event Damages—Central Stockton 01—Index Point D5

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	24,688	40,985	43,998	45,530	47,025	59,932	63,126
Residential	0	253,349	415,541	446,103	461,402	477,533	602,215	642,667
Public	0	14,277	18,241	22,830	24,850	26,854	43,068	46,872
Industrial	0	22,723	49,764	54,160	55,681	57,139	69,870	74,811
Commercial	0	27,993	39,997	42,054	42,879	43,687	52,537	54,732
TOTAL	0	343,030	564,528	609,145	630,342	652,238	827,623	882,208

TABLE 3-11: SINGLE-EVENT DAMAGES—CENTRAL STOCKTON 01—INDEX POINT FR1

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	0	0	69	2,972	19,634	63,965	72,288
Residential	0	0	0	8,378	52,530	250,798	640,500	769,660
Public	0	0	0	22	371	6,933	44,679	59,821
Industrial	0	0	0	1,138	23,967	67,982	78,006	83,936
Commercial	0	0	0	719	3,431	17,002	56,084	61,323
TOTAL	0	0	0	10,325	83,271	362,348	883,235	1,047,027

TABLE 3-12: SINGLE-EVENT DAMAGES—CENTRAL STOCKTON 02—INDEX POINT FR1

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	0	16,350	18,824	20,189	25,535	39,743	61,422
Residential	0	0	156,349	182,111	202,332	262,389	425,446	635,801
Public	0	0	20,256	23,388	25,423	32,678	54,534	158,083
Industrial	0	0	302,314	345,973	375,807	429,568	536,035	590,564
Commercial	0	0	33,912	42,956	52,596	100,516	241,158	400,367
TOTAL	0	0	529,181	613,253	676,347	850,685	1,296,917	1,846,237

TABLE 3-13: SINGLE-EVENT DAMAGES—CENTRAL STOCKTON 02—INDEX POINT SL2

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	0	0	0	0	16,638	43,321	58,022
Residential	0	0	0	0	0	235,595	476,609	607,263
Public	0	0	0	0	0	39,716	133,758	163,413
Industrial	0	0	0	0	0	168,774	288,437	324,328
Commercial	0	0	0	0	0	334,027	470,743	554,720
TOTAL	0	0	0	0	0	794,749	1,412,868	1,707,746

Table 3-14: Single-Event Damages—Central Stockton 03—Index Point CL2

DAMAGE CATEGORY	ACE EVENT							
	0.5	**0.2**	**0.1**	**0.04**	**0.02**	**0.01**	**0.004**	**0.002**
Autos	0	0	0	0	0	90	960	1,126
Residential	0	0	0	0	0	2,700	11,637	15,769
Public	0	0	0	0	0	293	1,529	2,882
Industrial	0	0	0	0	0	3,483	23,786	28,285
Commercial	0	0	0	0	0	1,499	13,863	15,988
TOTAL	0	0	0	0	0	8,065	51,776	64,049

Table 3-15: Single-Event Damages—Central Stockton 03—Index Point SL2

DAMAGE CATEGORY	ACE EVENT							
	0.5	**0.2**	**0.1**	**0.04**	**0.02**	**0.01**	**0.004**	**0.002**
Autos	0	0	0	0	0	7,382	9,916	12,027
Residential	0	0	0	0	0	172,022	192,442	206,606
Public	0	0	0	0	0	13,746	16,246	17,932
Industrial	0	0	0	0	0	76,209	93,210	107,673
Commercial	0	0	0	0	0	63,282	81,267	90,955
TOTAL	0	0	0	0	0	332,640	393,082	435,194

Table 3-16: Single-Event Damages—RD17—Index Point LR1

DAMAGE CATEGORY	ACE EVENT							
	0.5	**0.2**	**0.1**	**0.04**	**0.02**	**0.01**	**0.004**	**0.002**
Autos	0	0	33,578	42,612	47,087	57,965	65,735	70,001
Residential	0	0	366,262	473,042	528,203	672,276	816,939	883,516
Public	0	0	17,040	22,230	24,254	28,657	37,852	44,495
Industrial	0	0	24,071	47,576	56,008	198,458	483,184	545,915
Commercial	0	0	7,866	28,282	30,388	41,555	70,837	83,526
TOTAL	0	0	448,817	613,742	685,939	998,911	1,474,547	1,627,453

TABLE 3-17: SINGLE-EVENT DAMAGES—RD17—INDEX POINT LR2

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	0	42,425	53,429	58,331	66,996	76,685	80,754
Residential	0	0	473,019	604,291	676,789	834,854	965,234	1,015,574
Public	0	0	21,960	26,399	28,858	39,117	52,583	55,118
Industrial	0	0	46,154	65,921	254,358	503,061	607,093	644,994
Commercial	0	0	28,255	33,462	45,273	74,068	96,669	104,332
TOTAL	0	0	611,813	783,502	1,063,609	1,518,096	1,798,265	1,900,773

TABLE 3-18: SINGLE-EVENT DAMAGES—RD17—INDEX POINT LR3

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	0	46,209	65,970	71,114	82,174	89,536	92,183
Residential	0	0	506,696	813,104	884,985	1,036,997	1,133,851	1,176,047
Public	0	0	19,428	38,045	43,887	55,598	59,215	60,408
Industrial	0	0	79,179	482,405	537,398	655,376	733,803	771,011
Commercial	0	0	16,089	70,679	85,058	105,819	116,220	119,125
TOTAL	0	0	667,601	1,470,203	1,622,442	1,935,964	2,132,625	2,218,773

TABLE 3-19: SINGLE-EVENT DAMAGES—RD17—INDEX POINT LR4

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	0	504	1,284	2,336	51,273	72,292	71,936
Residential	0	0	13,118	26,081	45,458	586,136	872,140	879,043
Public	0	0	0	277	1,098	43,793	50,832	49,303
Industrial	0	0	13	118	162	882,024	573,260	597,639
Commercial	0	0	315	932	1,300	85,251	74,816	76,824
TOTAL	0	0	13,949	28,692	50,355	1,648,478	1,643,341	1,674,744

TABLE 3-20: SINGLE-EVENT DAMAGES—RD17—INDEX POINT LRTB

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	0	504	1,284	2,336	51,273	72,292	71,936
Residential	0	0	13,118	26,081	45,458	586,136	872,140	879,043
Public	0	0	0	277	1,098	43,793	50,832	49,303
Industrial	0	0	13	118	162	882,024	573,260	597,639
Commercial	0	0	315	932	1,300	85,251	74,816	76,824
TOTAL	0	0	13,949	28,692	50,355	1,648,478	1,643,341	1,674,744

TABLE 3-21: SINGLE-EVENT DAMAGES—RD17—INDEX POINT FL1

DAMAGE CATEGORY	ACE EVENT							
	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Autos	0	0	33,578	42,612	47,087	57,965	65,735	70,001
Residential	0	0	366,262	473,042	528,203	672,276	816,939	883,516
Public	0	0	17,040	22,230	24,254	28,657	37,852	44,495
Industrial	0	0	24,071	47,576	56,008	198,458	483,184	545,915
Commercial	0	0	7,866	28,282	30,388	41,555	70,837	83,526
TOTAL	0	0	448,817	613,742	685,939	998,911	1,474,547	1,627,453

3.2 PROBABILITY VARIABLES

The overall likelihood that flooding will occur in a given year is dependent on the probabilities associated with the engineering inputs described in section 2.3. Tables 3-22 through 3-28 summarize the engineering inputs for the highest risk index point for each damage area. Engineering inputs for all index points are provided in Attachment 5.

TABLE 3-22: ENGINEERING INPUTS—NORTH STOCKTON 02—2010 WITHOUT PROJECT

INDEX POINT D3						
ANNUAL CHANCE EXCEEDANCE	**UNREGULATED-REGULATED TRANSFORM**		**DISCHARGE-STAGE RATING**		**FRAGILITY CURVE**	
	Unregulated Flow (CFS)	**Regulated Flow (CFS)**	**Regulated Flow (CFS)**	**Regulated Stage (ft)**	**Stage (ft)**	***P* Of Failure**
0.999	0	0			2.00	0.0000
0.95	0	0	0	3.18	6.00	0.0928
0.50	21,899	2,424	2,424	7.70	8.50	0.2098
0.10	79,122	9,864	9,864	9.30	11.00	0.3419
0.04	124,892	11,158	11,158	9.70	13.20[*]	0.4593
0.02	167,074	12,298	12,298	9.90	13.21	1.0000
0.01	216,499	15,920	15,920	10.10		
0.005	273,861	28,712	28,712	12.12		
0.002	363,117	33,013	33,013	13.01		

---

[*] Top of levee stage

TABLE 3-23: ENGINEERING INPUTS—NORTH STOCKTON 03—2010 WITHOUT PROJECT

INDEX POINT D-BS						
ANNUAL CHANCE EXCEEDANCE	**UNREGULATED-REGULATED TRANSFORM**		**DISCHARGE-STAGE RATING**		**FRAGILITY CURVE**	
	Unregulated Flow (CFS)	**Regulated Flow (CFS)**	**Regulated Flow (CFS)**	**Regulated Stage (ft)**	**Stage (ft)**	***P* Of Failure**
0.999	0	0	0	3.18	-3.50	0.0000
0.50	21,899	2,424	2,424	7.70	6.00	0.0743
0.10	79,122	9,864	9,864	9.29	10.00	0.2006
0.04	124,892	11,158	11,158	9.70	14.00	0.5153
0.02	167,074	12,298	12,298	9.90	18.00[*]	0.8532
0.01	216,499	15,920	15,920	10.10	18.01	1.0000
0.005	273,861	28,712	28,712	12.12		
0.002	363,117	33,013	33,013	13.01		

---

[*] Top of levee stage

TABLE 3-24: ENGINEERING INPUTS—NORTH STOCKTON 04—2010 WITHOUT PROJECT

INDEX POINT CR2						
ANNUAL CHANCE EXCEEDANCE	**UNREGULATED-REGULATED TRANSFORM**		**DISCHARGE-STAGE RATING**		**FRAGILITY CURVE**	
	Unregulated Flow (CFS)	**Regulated Flow (CFS)**	**Regulated Flow (CFS)**	**Regulated Stage (ft)**	**Stage (ft)**	***P* Of Failure**
0.999	0	0	0	6.60	23.80	0.0000
0.50	6,901	3,848	3,848	19.13	25.30	0.0892
0.20	15,360	9,496	9,496	23.35	26.90	0.1783
0.10	21,654	9,861	9,861	23.58	28.20	0.3036
0.04	29,659	12,282	12,282	24.81	29.66[*]	0.4846
0.02	35,396	12,846	12,846	25.11	29.76	1.0000
0.01	40,815	15,359	15,359	26.29		
0.005	45,896	15,750	15,750	26.46		
0.002	52,080	19,126	19,126	27.98		

---

[*] Top of levee stage

TABLE 3-25: ENGINEERING INPUTS—CENTRAL STOCKTON 01—2010 WITHOUT PROJECT

INDEX POINT D5						
ANNUAL CHANCE EXCEEDANCE	**UNREGULATED-REGULATED TRANSFORM**		**DISCHARGE-STAGE RATING**		**FRAGILITY CURVE**	
	Unregulated Flow (CFS)	**Regulated Flow (CFS)**	**Regulated Flow (CFS)**	**Regulated Stage (ft)**	**Stage (ft)**	***P* Of Failure**
0.999	0	0	0	3.18	4.10	0.0000
0.50	6,901	3,784	3,784	8.24	7.20	0.0869
0.20	15,360	9,487	9487	10.90	10.00	0.1872
0.10	21,654	9,934	9,934	11.10	13.20	0.2698
0.04	29,659	12,270	12,270	11.97	17.54[*]	0.4023
0.02	35,396	12,751	12,751	12.22	17.55	1.0000
0.01	40,815	15,346	15,346	13.07		
0.005	45,896	15,736	15,736	13.41		
0.002	52,080	19,117	19,117	15.53		

---

[*] Top of levee stage

TABLE 3-26: ENGINEERING INPUTS—CENTRAL STOCKTON 02—2010 WITHOUT PROJECT

INDEX POINT FR1						
ANNUAL CHANCE EXCEEDANCE	UNREGULATED-REGULATED TRANSFORM		DISCHARGE-STAGE RATING		FRAGILITY CURVE	
	Unregulated Flow (CFS)	Regulated Flow (CFS)	Regulated Flow (CFS)	Regulated Stage (ft)	Stage (ft)	*P* Of Failure
0.999	0	0			8.14	0.0000
0.95	0	0	0	3.18	12.96	0.0663
0.50	21,899	1,776	1,776	7.33	15.90[*]	0.2537
0.10	79,122	7,774	7,774	11.75	18.84	0.5039
0.04	124,892	9,142	9,142	12.51	21.77[†]	0.7183
0.02	167,074	10,128	10,128	13.09	21.78	1.0000
0.01	216,499	13,869	13,869	14.65		
0.005	273,861	26,687	26,687	20.12		
0.002	363,117	32,943	32,943	21.98		

---

[*] Effective top of levee stage—elevation of natural upstream bank
[†] Top of levee stage

TABLE 3-27: ENGINEERING INPUTS—CENTRAL STOCKTON 03—2010 WITHOUT PROJECT

INDEX POINT CL2						
ANNUAL CHANCE EXCEEDANCE	UNREGULATED-REGULATED TRANSFORM		DISCHARGE-STAGE RATING		FRAGILITY CURVE	
	Unregulated Flow (CFS)	Regulated Flow (CFS)	Regulated Flow (CFS)	Regulated Stage (ft)	Stage (ft)	*P* Of Failure
0.999	0	0	0	6.60	21.00	0.0000
0.50	6,901	3,848	3,848	19.13	25.50	0.0845
0.20	15,360	9,496	9,496	23.35	27.46	0.1719
0.10	21,654	9,861	9,861	23.58	29.40	0.2527
0.04	29,659	12,282	12,282	24.81	31.43[*]	0.3790
0.02	35,396	12,846	12,846	25.11	31.53	1.0000
0.01	40,815	15,359	15,359	26.29		
0.005	45,896	15,750	15,750	26.46		
0.002	52,080	19,126	19,126	27.98		

---

[*] Top of levee stage

TABLE 3-28: ENGINEERING INPUTS—RD17—WITHOUT PROJECT

INDEX POINT LR2						
ANNUAL CHANCE EXCEEDANCE	**UNREGULATED-REGULATED TRANSFORM**		**DISCHARGE-STAGE RATING**		**FRAGILITY CURVE**	
	Unregulated Flow (CFS)	**Regulated Flow (CFS)**	**Regulated Flow (CFS)**	**Regulated Stage (ft)**	**Stage (ft)**	***P* Of Failure**
0.999	0	0	0	3.18	12.00	0.0000
0.50	21,899	1,771	1,771	7.60	17.00	0.1287
0.10	79,122	7,754	7,754	15.14	21.50	0.3839
0.04	124,892	9,143	9,143	16.47	24.65	0.5587
0.02	167,074	10,130	10,130	17.33	27.80[*]	0.6903
0.01	216,499	13,871	13,871	20.25	28.81	1.0000
0.005	273,861	15,734	15,734	22.96		
0.002	363,117	16,889	16,889	23.78		

---

[*] Top of levee stage

3.3 ANNUALIZED DAMAGES

Equivalent annual damages for the Lower San Joaquin study area are estimated to be approximately $314 million. Damages by consequence area and damage category are shown in Table 3-29 below.

TABLE 3-29: EQUIVALENT ANNUAL DAMAGES BY CONSEQUENCE AREA

CONSEQUENCE AREA	DAMAGE CATEGORY					
	AUTOS	COMMERCIAL	INDUSTRIAL	PUBLIC	RESIDENTIAL	TOTAL
NORTH STOCKTON	14,000	25,000	1,000	8,000	133,000	181,000
CENTRAL STOCKTON	6,000	10,000	19,000	6,000	67,000	108,000
RD17	1,000	1,000	6,000	1,000	16,000	25,000
TOTAL	21,000	36,000	26,000	16,000	217,000	314,000

3.4 WITHOUT-PROJECT PERFORMANCE

In addition to estimating damages, HEC-FDA reports flood risk in terms of project performance. Three statistical measures are provided, in accordance with ER 1105-2-101, to describe performance risk in probabilistic terms. These measures are described below.

ANNUAL EXCEEDANCE PROBABILITY – The chance of having a damaging flood in any given year.

LONG-TERM RISK — The probability of having one or more damaging floods over a period of time.

ASSURANCE — The probability that a target stage will not be exceeded during a specified flood.

A project's performance can be an indicator of its short and long-term risk. However, because probability is only half of the risk equation, poor levee performance does not inherently mean high risk. Without-project performance of the highest risk levee in each impact area is shown below in Table 3-30. Complete performance statistics area provided in Attachment 6.

TABLE 3-30: PROJECT PERFORMANCE BY DAMAGE AREA

DAMAGE AREA	ANNUAL EXCEEDENCE PROBABILITY	LONG-TERM RISK			ASSURANCE BY EVENT					
		10 YEARS	30 YEARS	50 YEARS	0.1	0.04	0.02	0.01	0.004	0.002
NS-02	0.152	0.81	0.99	1.00	0.75	0.72	0.70	0.63	0.47	0.39
NS-03	0.152	0.81	0.99	1.00	0.80	0.77	0.75	0.71	0.62	0.58
NS-04	0.011	0.10	0.28	0.42	0.97	0.93	0.89	0.84	0.77	0.71
CS-01	0.120	0.72	0.98	1.00	0.78	0.76	0.74	0.72	0.68	0.65
CS-02	0.027	0.24	0.56	0.75	0.95	0.91	0.81	0.49	0.10	0.02
CS-03	0.017	0.15	0.39	0.57	0.95	0.92	0.89	0.85	0.80	0.76
RD17	0.021	0.19	0.47	0.66	0.93	0.87	0.79	0.68	0.56	0.52

3.5 FUTURE WITHOUT-PROJECT CONDITION

As discussed in Section 2.4, future sea level rise is expected to result in higher probabilities of flooding at certain index points. Table 3-31 compares expected annual damages and annual exceedance probability for existing and future without-project conditions for index points that are expected to be affected by sea level rise. Index points CL2, CR2, and SL2 are not expected to be impacted by sea level rise and are not included in this table.

TABLE 3-31: EXPECTED IMPACTS OF SEA LEVEL RISE

DAMAGE AREA	INDEX POINT	EXPECTED ANNUAL DAMAGES		ANNUAL EXCEEDANCE PROBABILITY	
		PRESENT YEAR	FUTURE YEAR	PRESENT YEAR	FUTURE YEAR
NS-02	D3	83,245	137,403	0.1519	0.2091
NS-03	D4	47,105	77,489	0.0646	0.0962
	D-BS	33,233	97,846	0.1521	0.189
CS-01	D5	59,363	93,309	0.1197	0.1582
	FR1	10,784	14,999	0.027	0.0415
CS-02	FR1	23,451	34,082	0.027	0.0415
RD17	FL1	12,266	17,680	0.0132	0.0202
	LR1	12,291	13,334	0.0126	0.0141
	LR2	22,766	27,749	0.0211	0.0257
	LR3	18,214	19,304	0.0095	0.0101
	LR4	3,716	3,779	0.0073	0.0075
	LRTB	16,903	17,074	0.0117	0.0075

CHAPTER 4 — ALTERNATIVE EVALUATION

4.1 INITIAL ARRAY OF ALTERNATIVES

An initial array of flood risk management alternative plans was developed, evaluated and compared to identify a plan that reasonably maximizes net benefits. This initial array of flood risk management alternative plans primarily consists of various upstream and downstream dry dam configurations, bypass alignments, setback levees, a ring levee, and channel modifications. Alternatives in the initial array were either screened out or retained based on parametric cost and benefit analysis

Each alternative in the initial array is summarized below. A summary of the alternatives carried forward to the focused array is shown in Table 4-1. Visual representations of each initial alternative can be found in Attachment 7 of this appendix.

4.1.1 NO ACTION ALTERNATIVE

This alternative would have no federal action identified. It would be expected that the future without-project assumptions would be maintained. It is expected that current flood risk management structures would be maintained and existing flood risk would remain.

4.1.2 NORTH STOCKTON ALTERNATIVES

North Stockton-A: Delta Front North and Fourteen Mile Slough. This alternative addresses the delta flooding source. This alternative includes a closure structure across Mosher Slough. This alternative covers 32,400 linear feet (6.136 miles) of levee. This alternative was screened out.

North Stockton-B: Delta Front North and South, and Calaveras River. This alternative addresses the delta and tidal portion of the Calaveras River as the flooding sources. The alternative includes a closure structure across Mosher Slough. The alternative covers a total 50,400 linear feet (9.545 miles) of levee. This alternative was carried forward.

North Stockton-C: Delta Front North. This alternative addresses the delta flooding source. This alternative includes closure structures across Mosher Slough and Fourteen Mile Slough. The alternative covers a total 23,700 linear feet (4.488 miles) of levee. This alternative was screened out.

North Stockton-D: Fourteen Mile Slough, Delta Front South, and Calaveras River. This alternative addresses the delta and tidal portion of the Calaveras River as the flooding sources. The alternative covers a total 42,300 linear feet (8.011 miles) of levee. This alternative was screened out.

North Stockton-E: Full Calaveras River. This alternative addresses the right bank of the Calaveras River as the flooding source. This alternative covers a total 41,900 linear feet (7.936 miles) of levee. This alternative was screened out.

North Stockton-F: Delta Front North and South, and Calaveras River. This alternative addresses the right bank of the Calaveras River and the delta front as flooding sources. This alternative includes closure structures across Mosher Slough and Fourteenmile Slough. This alternative covers a total 69,300 linear feet (13.125 miles) of levee. This alternative was carried forward.

4.1.3 CENTRAL STOCKTON ALTERNATIVES

Central Stockton-A: Calaveras and Diverting Canal. This alternative addresses the Stockton Diverting Canal and Calaveras River as flooding sources. The alternative covers a total 55,800 linear feet (10.568 miles) of levee. This alternative was screened out.

Central Stockton-B: Calaveras River. This alternative addresses the tidal portion of the Calaveras River and the San Joaquin River as sources of flooding and includes the Smith Canal closure structure. The alternative covers a total 19,000 linear feet (3.598 miles) of levee. This alternative was screened out.

Central Stockton-C: San Joaquin River Front. This alternative addresses the San Joaquin River, French Camp Slough, and Duck Creek as sources of flooding. The alternative covers a total 23,100 linear feet (10.189 miles) of levee. This alternative was screened out.

Central Stockton-D: Calaveras River, Diverting Canal, and San Joaquin River. This alternative addresses the San Joaquin River, Stockton Diverting Canal, Calaveras River, French Camp Slough and Duck Creek as flooding sources and includes the Smith Canal closure structure. The alternative covers a total 88,900 linear feet (16.837 miles) of levee. This alternative was carried forward.

Central Stockton-E: Calaveras River and Smith Canal. This alternative addresses the tidal portion of the Calaveras River and Smith Canal area as sources of flooding. The alternative covers a total 46,800 linear feet (8.864 miles) of levee. This alternative was screened out.

Central Stockton-F: Calaveras River and San Joaquin River. This alternative addresses the tidal portion of the Calaveras River, the San Joaquin River, French Camp Slough, and Duck Creek as flooding sources. The Smith Canal closure structure is also included. The alternative covers a total 51,600 linear feet (9.773 miles) of levee. This alternative was carried forward.

Central Stockton-G: Mormon Channel Bypass. This alternative develops a 1,200 cubic foot per second capacity diversion to the Mormon Channel from the Stockton Diverting Canal. The restoration of flows would affect 33,400 linear feet (6.326 miles) of channel. No levees are included. This alternative was screened out.

4.1.4 RECLAMATION DISTRICT 17 ALTERNATIVES

RD17-A: San Joaquin River North. This alternative addresses the San Joaquin River and French Camp Slough as the flooding sources. The alternative covers a total 77,000 linear feet (14.583 miles) of levee. This alternative was screened out.

RD17-B: San Joaquin River Tieback. This alternative addresses the San Joaquin River as the flooding source. The alternative covers a total 21,900 linear feet (4.148 miles) of levee. This alternative was screened out.

RD17-C: San Joaquin River North and Tieback. This alternative addresses the San Joaquin River and French Camp Slough as the flooding sources. The alternative covers a total 98,900 linear feet (18.731 miles) of levee. This alternative was screened out.

RD17-D: San Joaquin River Setback and Tieback. This alternative addresses the San Joaquin River as the flooding source, and includes a setback levee to limit protection of undeveloped floodplain within RD 17. The alternative covers a total 100,300 linear feet (18.996 miles) of levee. This alternative was screened out.

RD17-E: San Joaquin River North with Tieback and Extension. This alternative addresses the San Joaquin River and French Camp Slough as flooding sources. This alternative also extends the tie-back levee to address flanking issues. The alternative covers a total 106,900 linear feet (18.731 miles) of levee. This alternative was carried forward

RD17-F: Weston Ranch Ring Levee. This alternative addresses the San Joaquin River and French Camp Slough as flooding sources for Weston Ranch. The alternative includes new levee to form a ring levee around Weston Ranch, and an extension of RD 404 levees to prevent flanking during lower frequency events. The alternative covers a total 33,370 linear feet (6.3 miles) of levee. This alternative was screened out.

RD17-G: San Joaquin River Setback and Tieback Extension. This alternative addresses the San Joaquin River as the flooding source, and includes a setback levee to limit protection of undeveloped floodplain within RD 17. This alternative extends the tieback levee at the southern-most end of the reclamation district to minimize probability of flanking during extreme high water events. The alternative covers a total 113,500 linear feet (21.5 miles) of levee. This alternative was screened out.

TABLE 4-1: INITIAL ALTERNATIVES RETAINED

INCREMENT	ANNUAL BENEFITS	NET BENEFITS
North Stockton-B	72,000	53,000
North Stockton-F	76,000	54,000
Central Stockton-D	69,000	56,000
Central Stockton-F	56,000	46,000
RD17-E	27,000	12,000

4.2 FOCUSED ARRAY OF ALTERNATIVES

The project delivery team (PDT) used measures retained from the initial array to develop a focused array of alternatives. Each alternative in the focused array was evaluated on its performance relative to planning criteria set forth in USACE guidance, which states that the plan most reasonably maximizing net economic benefits is identified as the National Economic Development (NED) plan. A plan other than the NED Plan may be selected based on additional criteria but would require approval by the Assistant Secretary of the Army for Civil Works (ASA[CW]).

The following alternatives were evaluated as part of the focused array. Visual representations of each focused alternative can be found in Attachment 8 of this appendix.

4.2.1 NO ACTION

This alternative would have no federal action identified. It would be expected that the future without-project assumptions would be maintained. It is expected that current flood risk management structures would be maintained and existing flood risk would remain.

4.2.2 ALTERNATIVE 2a

This alternative combines the following alternatives to arrive at a comprehensive solution: North Stockton-F, Central Stockton-D, and RD 17-E. The alternative would implement levee improvements without implementing either of the Mormon Channel or Paradise Cut bypasses. The estimated extent of levee repairs would be approximately 53.14 miles (280,600 feet). This alternative was removed from consideration.

4.2.3 ALTERNATIVE 2b

This alternative combines the following alternatives to arrive at a comprehensive solution: North Stockton-B, Central Stockton-F, and RD 17-E. The alternative would implement levee improvements without implementing either of the Mormon Channel or Paradise Cut bypasses. The estimated extent of levee repairs would be approximately 42.5 miles (224,400 feet). This alternative was removed from consideration.

4.2.4 ALTERNATIVE 4

This alternative includes levee raises to meet SB 5 height requirements, where required, and also includes additional height increases for projected sea level and climate changes to the planning year 2070. The components of this plan are: North Stockton-B, Central Stockton-F, RD 17-E, and the Mormon Channel Bypass. The alternative would implement levee improvements along with restoration of the Mormon Channel including a diversion control structure at the Stockton Diverting Canal. The estimated extent of levee repairs and would be approximately 42.5 miles (224,400 feet) plus approximately 6.33 miles (33,400 feet) of channel work for the Mormon Channel portion. This alternative was removed from consideration.

4.2.5 ALTERNATIVE 7a

This alternative combines the following alternatives to arrive at a comprehensive solution: North Stockton-B and Central Stockton-F. The alternative would implement levee improvements without implementing either of the Mormon Channel or Paradise Cut bypasses. The alternative would combine the levee improvement measures of cutoff wall, deep soil mixing (seismic), seepage berm, and levee geometry improvements. In addition to the levee improvements, this alternative would address projected sea level change by including raises in levee height where needed. The proposed levee improvements in this alternative are comparable to Alternative 7b, with the exception that the RD17 components are not included. This alternative was carried forward to the final array.

4.2.6 ALTERNATIVE 7b

This alternative combines the following alternatives to arrive at a comprehensive solution: North Stockton-B, Central Stockton-F, and RD 17-E. The alternative would implement levee improvements without implementing either of the Mormon Channel or Paradise Cut bypasses. The alternative would combine the levee improvement measures of cutoff wall, deep soil mixing (seismic), seepage berm, and levee geometry improvements. In addition to the levee improvements, this alternative would address projected sea level change by including raises in levee height where needed. There would also be approximately 2.2 miles of new levee constructed to extend the RD-17 tie-back levee and the secondary levee at the Old River flow split. The new levees would also include a cutoff wall to address potential seepage issues. This alternative was carried forward to the final array.

4.2.7 ALTERNATIVE 8a

This alternative combines the following alternatives to arrive at a comprehensive solution: North Stockton-F and Central Stockton-D. The alternative would implement levee improvements without implementing either of the Mormon Channel or Paradise Cut bypasses. The alternative would combine the levee improvement measures of cutoff wall, deep soil mixing (seismic), seepage berm, and levee geometry improvements. In addition to the levee improvements, this alternative would address projected sea level change by including raises in levee height where needed. The proposed levee improvements in this alternative are comparable to Alternative 8, with the exception that the RD17 components are not included. This alternative was carried forward to the final array.

4.2.8 ALTERNATIVE 8b

This alternative combines the following alternatives to arrive at a comprehensive solution: North Stockton-F, Central Stockton-D, and RD 17-E. The alternative would implement levee improvements without implementing either of the Mormon Channel or Paradise Cut bypasses. The alternative would combine the levee improvement measures of cutoff wall, deep soil mixing (seismic), seepage berm, and levee geometry improvements. In addition to the levee improvements, this alternative would address projected sea level change by including raises in levee height where needed. There would also be approximately 2.2 miles of new levee constructed to extend the RD-17 tie-back levee and the secondary levee at the Old River flow split. The new levees would also include a cutoff wall to address potential seepage issues. This alternative was carried forward to the final array.

4.2.9 ALTERNATIVE 9a

This alternative combines the following alternatives to arrive at a comprehensive solution: North Stockton-B, Central Stockton-F, and the Mormon Channel Bypass. The alternative would implement levee improvements along with restoration of the Mormon Channel including a diversion control structure at the Stockton Diverting Canal. The alternative would combine the levee improvement measures of cutoff wall, deep soil mixing (seismic), seepage berm, and levee geometry improvements. In addition to the levee improvements, this alternative would address projected sea level change by including raises in levee height where needed. The diversion control structure for Mormon Channel at the Stockton Diverting Canal would consist of pipe culverts with gates to control releases to a maximum flow of approximately 1,200 cubic feet per second to Mormon Channel. The proposed levee improvements in this alternative are comparable to Alternative 9b, with the exception that the RD17 components are not included. This alternative was carried forward to the final array.

4.2.10 ALTERNATIVE 9b

This alternative combines the following alternatives to arrive at a comprehensive solution: North Stockton-B, Central Stockton-F, RD 17-E, and the Mormon Channel Bypass. The alternative would implement levee improvements along with restoration of the Mormon Channel including a diversion control structure at the Stockton Diverting Canal. The alternative would combine the levee improvement measures of cutoff wall, deep soil mixing (seismic), seepage berm, and levee geometry improvements. In addition to the levee improvements, this alternative would address projected sea level change by including raises in levee height where needed. There would also be approximately 2.2 miles of new levee constructed to extend the RD-17 tie-back levee and the secondary levee at the Old River flow split. The new levees would also include a cutoff wall to address potential seepage issues. The diversion control structure for Mormon Channel at the Stockton Diverting Canal would consist of pipe culverts with gates to control releases to a maximum flow of approximately 1,200 cubic feet per second to Mormon Channel. This alternative was carried forward to the final array.

4.3 SCREENING OF THE FOCUSED ARRAY

Evaluation of each alternative in the focused array led to the selection of five alternatives to be included in the final array. A key component of the screening process was the consideration of potential sea level rise, which led to the elimination of alternatives 2A, 2B, and 4, none of which include measures that address sea level rise.

4.4 WITH-PROJECT DAMAGES

The residual damages and project benefits for each final alternative are summarized in Table 4-2**.**

TABLE 4-2: FINAL ARRAY OF ALTERNATIVES—RESIDUAL DAMAGES

ALTERNATIVE	RESIDUAL ANNUAL DAMAGES				ANNUAL BENEFITS	ANNUAL DAMAGE REDUCTION
	NORTH STOCKTON	CENTRAL STOCKTON	RD-17	TOTAL		
NO ACTION	181,000	108,000	25,000	314,000	0	-
LS-7a	4,000	21,000	25,000	50,000	264,000	84.1%
LS-8a	2,000	20,000	25,000	47,000	267,000	85.0%
LS-9a	4,000	21,000	25,000	50,000	264,000	84.1%
LS-7b	3,000	18,000	1,000	22,000	292,000	93.0%
LS-8b	1,000	16,000	1,000	18,000	296,000	94.3%
LS-9b	2,000	17,000	1,000	20,000	294,000	93.6%

4.5 WITH-PROJECT PERFORMANCE

Existing and future performance statistics for each of the alternative in the final array are shown in Tables 4-3 through 4-14.

TABLE 4-3: PROJECT PERFORMANCE BY DAMAGE AREA—ALTERNATIVE LS-7A—PRESENT YEAR

DAMAGE AREA	ANNUAL EXCEEDENCE PROBABILITY	LONG-TERM RISK			ASSURANCE BY EVENT					
		10 YEARS	30 YEARS	50 YEARS	0.1	0.04	0.02	0.01	0.004	0.002
NS-02	0.009	0.00	0.01	0.01	1.00	1.00	1.00	0.99	0.95	0.92
NS-03	0.009	0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.99	0.98
NS-04	0.009	0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.99	0.98
CS-01	0.017	0.07	0.20	0.31	1.00	1.00	0.98	0.77	0.27	0.08
CS-02	0.015	0.07	0.20	0.31	1.00	1.00	0.98	0.77	0.27	0.08
CS-03	0.017	0.00	0.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00
RD17	0.021	0.19	0.47	0.66	0.93	0.87	0.79	0.68	0.56	0.52

TABLE 4-4: PROJECT PERFORMANCE BY DAMAGE AREA—ALTERNATIVE LS-7A—FUTURE YEAR

DAMAGE AREA	ANNUAL EXCEEDENCE PROBABILITY	LONG-TERM RISK			ASSURANCE BY EVENT					
		10 YEARS	30 YEARS	50 YEARS	0.1	0.04	0.02	0.01	0.004	0.002
NS-02	0.009	0.02	0.06	0.10	1.00	0.99	0.98	0.93	0.79	0.70
NS-03	0.009	0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.99	0.98
NS-04	0.000	0.00	0.00	0.00	1.00	1.00	1.00	1.00	0.99	0.98
CS-01	0.017	0.08	0.21	0.32	1.00	1.00	0.97	0.74	0.24	0.07
CS-02	0.015	0.08	0.21	0.32	1.00	1.00	0.97	0.74	0.24	0.07
CS-03	0.017	0.00	0.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00
RD17	0.026	0.23	0.54	0.73	0.92	0.84	0.77	0.67	0.55	0.52

TABLE 4-5: PROJECT PERFORMANCE BY DAMAGE AREA—ALTERNATIVE LS-8A—PRESENT YEAR

DAMAGE AREA	ANNUAL EXCEEDENCE PROBABILITY	LONG-TERM RISK			ASSURANCE BY EVENT					
		10 YEARS	30 YEARS	50 YEARS	0.1	0.04	0.02	0.01	0.004	0.002
NS-02	0.000	0.00	0.01	0.01	0.999	0.999	0.999	0.99	0.95	0.92
NS-03	0.000	0.00	0.01	0.02	0.999	0.999	0.997	0.99	0.96	0.93
NS-04	0.000	0.00	0.01	0.02	0.999	0.999	0.997	0.99	0.96	0.93
CS-01	0.007	0.07	0.20	0.31	0.999	0.999	0.98	0.77	0.27	0.08
CS-02	0.007	0.07	0.20	0.31	0.999	0.999	0.98	0.77	0.27	0.08
CS-03	0.000	0.00	0.00	0.00	0.999	0.999	0.999	0.998	0.99	0.97
RD17	0.021	0.19	0.47	0.66	0.93	0.87	0.79	0.68	0.56	0.52

TABLE 4-6: PROJECT PERFORMANCE BY DAMAGE AREA—ALTERNATIVE LS-8A—FUTURE YEAR

DAMAGE AREA	ANNUAL EXCEEDENCE PROBABILITY	LONG-TERM RISK			ASSURANCE BY EVENT					
		10 YEARS	**30 YEARS**	**50 YEARS**	**0.1**	**0.04**	**0.02**	**0.01**	**0.004**	**0.002**
NS-02	0.002	0.02	0.06	0.10	0.996	0.991	0.983	0.93	0.79	0.70
NS-03	0.000	0.00	0.01	0.02	0.999	0.999	0.997	0.99	0.96	0.93
NS-04	0.000	0.00	0.01	0.02	0.999	0.999	0.997	0.99	0.96	0.93
CS-01	0.008	0.08	0.21	0.32	0.999	0.999	0.97	0.74	0.24	0.07
CS-02	0.008	0.08	0.21	0.32	0.999	0.999	0.97	0.74	0.24	0.07
CS-03	0.000	0.00	0.00	0.00	0.999	0.999	0.999	0.998	0.99	0.97
RD17	0.026	0.23	0.54	0.73	0.92	0.84	0.77	0.67	0.55	0.52

TABLE 4-7: PROJECT PERFORMANCE BY DAMAGE AREA—ALTERNATIVE LS-9A—PRESENT YEAR

DAMAGE AREA	ANNUAL EXCEEDENCE PROBABILITY	LONG-TERM RISK			ASSURANCE BY EVENT					
		10 YEARS	30 YEARS	50 YEARS	0.1	0.04	0.02	0.01	0.004	0.002
NS-02	0.005	0.05	0.14	0.23	0.99	0.96	0.93	0.89	0.83	0.80
NS-03	0.005	0.05	0.14	0.23	0.99	0.96	0.93	0.89	0.83	0.80
NS-04	0.005	0.05	0.14	0.23	0.99	0.96	0.93	0.89	0.83	0.80
CS-01	0.015	0.14	0.36	0.52	0.96	0.95	0.94	0.91	0.88	0.85
CS-02	0.011	0.10	0.28	0.42	0.97	0.94	0.92	0.89	0.84	0.80
CS-03	0.015	0.14	0.36	0.52	0.96	0.95	0.94	0.91	0.88	0.85
RD17	0.021	0.19	0.47	0.66	0.93	0.87	0.79	0.68	0.56	0.52

TABLE 4-8: PROJECT PERFORMANCE BY DAMAGE AREA—ALTERNATIVE LS-9A—FUTURE YEAR

DAMAGE AREA	ANNUAL EXCEEDENCE PROBABILITY	LONG-TERM RISK			ASSURANCE BY EVENT					
		10 YEARS	30 YEARS	50 YEARS	0.1	0.04	0.02	0.01	0.004	0.002
NS-02	0.005	0.05	0.14	0.23	0.99	0.96	0.93	0.89	0.83	0.80
NS-03	0.005	0.05	0.14	0.23	0.99	0.96	0.93	0.89	0.83	0.80
NS-04	0.005	0.05	0.14	0.23	0.99	0.96	0.93	0.89	0.83	0.80
CS-01	0.015	0.14	0.36	0.52	0.96	0.95	0.94	0.91	0.88	0.85
CS-02	0.011	0.10	0.28	0.42	0.97	0.94	0.92	0.89	0.84	0.80
CS-03	0.015	0.14	0.36	0.52	0.96	0.95	0.94	0.91	0.88	0.85
RD17	0.026	0.23	0.54	0.73	0.92	0.84	0.77	0.67	0.55	0.52

TABLE 4-9: PROJECT PERFORMANCE BY DAMAGE AREA—ALTERNATIVE LS-7B—PRESENT YEAR

DAMAGE AREA	ANNUAL EXCEEDENCE PROBABILITY	LONG-TERM RISK			ASSURANCE BY EVENT					
		10 YEARS	30 YEARS	50 YEARS	0.1	0.04	0.02	0.01	0.004	0.002
NS-02	0.009	0.10	0.28	0.42	0.97	0.93	0.89	0.84	0.78	0.71
NS-03	0.009	0.10	0.28	0.42	0.97	0.93	0.89	0.84	0.78	0.71
NS-04	0.009	0.10	0.28	0.42	0.97	0.93	0.89	0.84	0.78	0.71
CS-01	0.017	0.16	0.41	0.58	0.95	0.93	0.91	0.88	0.85	0.83
CS-02	0.015	0.15	0.39	0.57	0.95	0.92	0.89	0.85	0.80	0.76
CS-03	0.017	0.16	0.41	0.58	0.95	0.93	0.91	0.88	0.85	0.83
RD17	0.000	0.00	0.00	0.00	0.999	0.999	0.999	0.998	0.99	0.99

TABLE 4-10: PROJECT PERFORMANCE BY DAMAGE AREA—ALTERNATIVE LS-7B—FUTURE YEAR

DAMAGE AREA	ANNUAL EXCEEDENCE PROBABILITY	LONG-TERM RISK			ASSURANCE BY EVENT					
		10 YEARS	30 YEARS	50 YEARS	0.1	0.04	0.02	0.01	0.004	0.002
NS-02	0.009	0.11	0.29	0.44	0.97	0.93	0.89	0.84	0.77	0.71
NS-03	0.009	0.11	0.29	0.44	0.97	0.93	0.89	0.84	0.77	0.71
NS-04	0.009	0.11	0.29	0.44	0.97	0.93	0.89	0.84	0.77	0.71
CS-01	0.017	0.17	0.43	0.60	0.95	0.93	0.91	0.88	0.85	0.83
CS-02	0.015	0.15	0.34	0.57	0.95	0.92	0.89	0.85	0.80	0.76
CS-03	0.017	0.17	0.43	0.60	0.95	0.93	0.91	0.88	0.85	0.83
RD17	0.001	0.01	0.04	0.06	0.999	0.999	0.995	0.955	0.86	0.82

Table 4-11: Project Performance by Damage Area—Alternative LS-8b—Present Year

DAMAGE AREA	ANNUAL EXCEEDENCE PROBABILITY	LONG-TERM RISK			ASSURANCE BY EVENT					
		10 YEARS	30 YEARS	50 YEARS	0.1	0.04	0.02	0.01	0.004	0.002
NS-02	0.000	0.00	0.00	0.00	0.999	0.999	0.999	0.999	0.999	0.999
NS-03	0.000	0.00	0.01	0.02	0.999	0.999	0.997	0.99	0.96	0.93
NS-04	0.000	0.00	0.01	0.02	0.999	0.999	0.997	0.99	0.96	0.93
CS-01	0.007	0.07	0.19	0.30	0.999	0.99	0.93	0.74	0.45	0.35
CS-02	0.007	0.07	0.19	0.30	0.999	0.99	0.93	0.74	0.45	0.35
CS-03	0.000	0.00	0.00	0.00	0.999	0.999	0.999	0.998	0.99	0.97
RD17	0.000	0.00	0.00	0.00	0.999	0.999	0.999	0.999	0.99	0.99

Table 4-12: Project Performance by Damage Area—Alternative LS-8b—Future Year

DAMAGE AREA	ANNUAL EXCEEDENCE PROBABILITY	LONG-TERM RISK			ASSURANCE BY EVENT					
		10 YEARS	30 YEARS	50 YEARS	0.1	0.04	0.02	0.01	0.004	0.002
NS-02	0.001	0.01	0.03	0.05	0.996	0.991	0.987	0.982	0.977	0.974
NS-03	0.000	0.00	0.01	0.02	0.999	0.999	0.997	0.99	0.96	0.93
NS-04	0.000	0.00	0.01	0.02	0.999	0.999	0.997	0.99	0.96	0.93
CS-01	0.012	0.11	0.30	0.45	0.993	0.95	0.83	0.59	0.32	0.23
CS-02	0.012	0.11	0.30	0.45	0.993	0.95	0.83	0.59	0.32	0.23
CS-03	0.000	0.00	0.00	0.00	0.999	0.999	0.999	0.998	0.99	0.97
RD17	0.001	0.01	0.04	0.06	0.999	0.999	0.995	0.955	0.86	0.82

TABLE 4-13: PROJECT PERFORMANCE BY DAMAGE AREA—ALTERNATIVE LS-9B—PRESENT YEAR

DAMAGE AREA	ANNUAL EXCEEDENCE PROBABILITY	LONG-TERM RISK			ASSURANCE BY EVENT					
		10 YEARS	**30 YEARS**	**50 YEARS**	**0.1**	**0.04**	**0.02**	**0.01**	**0.004**	**0.002**
NS-02	0.005	0.05	0.15	0.24	0.99	0.95	0.92	0.88	0.82	0.78
NS-03	0.005	0.05	0.15	0.24	0.99	0.95	0.92	0.88	0.82	0.78
NS-04	0.005	0.05	0.15	0.24	0.99	0.95	0.92	0.88	0.82	0.78
CS-01	0.015	0.14	0.36	0.52	0.96	0.95	0.93	0.91	0.87	0.85
CS-02	0.007	0.07	0.19	0.30	0.999	0.99	0.93	0.74	0.45	0.35
CS-03	0.015	0.14	0.36	0.52	0.96	0.95	0.93	0.91	0.87	0.85
RD17	0.000	0.00	0.00	0.00	0.999	0.999	0.999	0.998	0.99	0.99

TABLE 4-14: PROJECT PERFORMANCE BY DAMAGE AREA—ALTERNATIVE LS-9B—FUTURE YEAR

DAMAGE AREA	ANNUAL EXCEEDENCE PROBABILITY	LONG-TERM RISK			ASSURANCE BY EVENT					
		10 YEARS	**30 YEARS**	**50 YEARS**	**0.1**	**0.04**	**0.02**	**0.01**	**0.004**	**0.002**
NS-02	0.005	0.06	0.17	0.27	0.99	0.95	0.92	0.88	0.82	0.77
NS-03	0.005	0.06	0.17	0.27	0.99	0.95	0.92	0.88	0.82	0.77
NS-04	0.005	0.06	0.17	0.27	0.99	0.95	0.92	0.88	0.82	0.77
CS-01	0.015	0.14	0.37	0.53	0.96	0.94	0.93	0.90	0.87	0.85
CS-02	0.012	0.11	0.30	0.45	0.993	0.95	0.83	0.59	0.32	0.23
CS-03	0.015	0.14	0.37	0.53	0.96	0.94	0.93	0.90	0.87	0.85
RD17	0.001	0.01	0.04	0.06	0.999	0.999	0.995	0.955	0.86	0.82

4.6 PROJECT COSTS

Project costs were estimated by USACE, Sacramento District's cost engineering section. Total first cost and construction duration for each alternative are shown in Tables 4-15 through 4-20 below. These estimates do not include interest during construction.

TABLE 4-15: FIRST COST ESTIMATE—ALTERNATIVE 7A

FIX	START YEAR	END YEAR	TOTAL FIRST COST
NORTH STOCKTON	2018	2028	$616,800
CENTRAL STOCKTON	2017	2020	$210,500
RD17	2017	2028	$0

TABLE 4-16: FIRST COST ESTIMATE—ALTERNATIVE 8A

FIX	START YEAR	END YEAR	TOTAL FIRST COST
NORTH STOCKTON	2018	2028	$669,400
CENTRAL STOCKTON	2017	2020	$291,500
RD17	2017	2028	$0

TABLE 4-17: FIRST COST ESTIMATE—ALTERNATIVE 9A

FIX	START YEAR	END YEAR	TOTAL FIRST COST
NORTH STOCKTON	2018	2028	$607,200
CENTRAL STOCKTON	2017	2020	$248,300
RD17	2017	2028	$0

TABLE 4-18: FIRST COST ESTIMATE—ALTERNATIVE 7B

FIX	START YEAR	END YEAR	TOTAL FIRST COST
NORTH STOCKTON	2018	2028	$599,700
CENTRAL STOCKTON	2017	2020	$204,000
RD17	2024	2030	$410,100

TABLE 4-19: FIRST COST ESTIMATE—ALTERNATIVE 8B

FIX	START YEAR	END YEAR	TOTAL FIRST COST
NORTH STOCKTON	2018	2028	$644,000
CENTRAL STOCKTON	2017	2020	$280,000
RD17	2024	2030	$410,000

TABLE 4-20: FIRST COST ESTIMATE—ALTERNATIVE 9B

FIX	START YEAR	END YEAR	TOTAL FIRST COST
NORTH STOCKTON	2018	2028	$594,000
CENTRAL STOCKTON	2017	2020	$242,000
RD17	2024	2030	$406,000

4.6.1 INTEREST AND BENEFITS DURING CONSTRUCTION

As delivered, the total project costs did not included interest during construction or benefits during construction.

Interest during construction (IDC) accrues each year between the start of construction and the base year. Total IDC is annualized over the period of analysis and added to the annual project cost.

Benefits during construction (BDC) are benefits that accrue annually between the year that one or more elements of the project begin to realize benefits and the base year. Total BDC is annualized over the period of analysis and added to the annual project benefits.

For this study, both IDC and BDC were calculated using the FY2014 discount rate of 3.5% and a 50 year period of analysis. Complete IDC and BDC calculations can be found in Attachment 9.

4.7 NET BENEFITS AND BENEFIT-TO-COST RATIO

Once benefit and cost calculations are complete, each alternative can be evaluated based on its net benefits (total return on investment) and benefit-to-cost ratio (return on each dollar invested). These metrics may provide the basis for decision-makers when selecting a plan. The net benefits and benefit-to-cost ratios for each final alternative are reported in Table 4-21.

TABLE 4-21: FINAL ARRAY OF ALTERNATIVES—ECONOMIC SUMMARY

ALTERNATIVE	RESIDUAL DAMAGES	ANNUAL BENEFITS[*]	ANNUAL COST[†]	NET BENEFITS	BENEFIT TO COST RATIO
NO ACTION	314,000	0	0	0	0
LS-7a	50,000	299,000	45,000	254,000	6.64
LS-8a	47,000	302,000	52,000	250,000	5.81
LS-9a	50,000	299,000	47,000	252,000	6.38
LS-7b	22,000	355,000	66,000	289,000	5.38
LS-8b	18,000	359,000	73,000	286,000	4.92
LS-9b	20,000	356,000	68,000	288,000	5.24

---

[*] Includes benefits during construction

[†] Includes interest during construction

ATTACHMENT 1: DESCRIPTION OF FLOOD SOURCES

Sacramento and San Joaquin Delta

The Sacramento and San Joaquin Delta covers more than 1,000 square miles of Central California. The delta is located at the confluence of the Sacramento and San Joaquin Rivers at the head of Suisun Bay, the most easterly extending arm of the San Francisco Bay system. In general, the Delta extends from about Sacramento on the north, to Stockton on the south, and near Pittsburg on the west. This region, which is very flat, has been reclaimed from a natural tidal area by hundreds of miles of levees along natural and manmade waterways that divide it into about 100 tracts locally know as "islands".

Before the islands were reclaimed, much of the Delta was covered by water from the daily tide cycle. During times of high runoff from the Sacramento and San Joaquin Basins, much of the Delta would be flooded. Reclamation of the many of the Delta islands has subjected the peat soils to oxidation. As a result, the interior of most islands have subsided well below sea level. Elevations within the islands now range from just above mean sea level to 10 feet below mean sea level.

Maximum stages within the Delta result from runoff from storms of different origins which do not have the same annual exceedance frequency at all locations, and from tides of varying magnitudes which seldom reach their maximum stages concurrently with the peak flows. In some years the annual maximum stage at all locations occurs during the same storm event. However, in other years, the peak stages in the northern part of the Delta occur during a different time period than those in the southern part of the Delta and vice versa. The differences are caused by the geographical distribution of the contributing drainage basin, antecedent conditions such as snowpack and soil moisture, and the fluctuation of the storm tracks over California. If the flood runoff is from the Sacramento River basin, the stages will be higher in the northern part of the Delta. If the main flood runoff is from the San Joaquin River, then the stages will be higher in the southern part of the Delta.

Several sloughs of the Delta including Five Mile Slough, Fourteen Mile Slough, and Ten Mile present significant flood risk to the study area. These sloughs have relatively small tributary areas and stages within the sloughs are primarily influenced by the combined tide and runoff from the Sacramento and San Joaquin Rivers.

San Joaquin River

The San Joaquin River is the principle stream in the southern half of the Central Valley of California. The San Joaquin is a perennial stream sustained through the summer by melting snow and releases from reservoirs. Its main headwater tributaries, the south and middle forks, rise in glacial lakes in the southern Sierra Nevada. They join at about elevation 3600 feet NAVD88 to form the main stem, which flows west-southwesterly to the valley floor, thence northwesterly down the main trough of the valley to the study area and its terminus at Suisun Bay. Upstream from the study area, the river is joined by several major tributaries flowing from the east and by a number of minor low elevation tributaries that flow from the east and west and have little effect on flood flows and stages. The major tributaries flowing from the east are the Stanislaus, Tuolumne, Merced, Chowchilla, and Fresno Rivers. Less significant eastside tributaries comprise French Camp Slough (terminus of Duck and LittleJohns Creeks systems). The principal Westside tributaries are Panoche, Los Banos, San Luis, and Orestimba Creeks. Fresno Slough, a distributary of the Kings river that cuts through the valley-floor barrier ridge separating the Tulare Lake Basin from the San Joaquin River Basin proper, could contribute runoff to the San Joaquin River during extreme flood events.

Calaveras River

The Calaveras River is a tributary of the San Joaquin River. Elevations in the Calaveras River drainage vary from about 6,000 feet in the highest headwater areas to about 30 feet in the lower part of the study area. In the study area, the Calaveras River is distributary in nature, the stream divides into the north and south branches at Bellota, where a diversion of flow structure has been provided. The northern branch Calaveras River, flows westerly across the valley floor to join the San Joaquin River just west of Stockton. Very little flow enters this branch except during the summer when diversions are made for irrigation and ground-water replenishment. The southern branch, Mormon Slough, carries most of the flow. Its course extends in a general southwesterly direction from Bellota to the Stockton Diverting Canal flow diversion structure. The structure diverts all flood flows to the diverting canal which discharges into the Calaveras River. The Mormon Slough reach below the diverting dam is referred to locally as Mormon Channel. The source of flow in Mormon Channel is the local tributary area downstream of the diversion structure.

French Camp Slough

French Camp Slough is a tributary to the San Joaquin River south of the City of Stockton. The slough receives waters from Duck Creek and Littlejohn Creek. This slough, with or without upstream reservoirs has no effect on major flood flows in the San Joaquin River (USACE, 1955).

Duck Creek

Duck Creek is a small tributary of the French Camp Slough, south of the City of Stockton, lying between the Calaveras River-Mormon Slough system and Littlejohn Creek. It has a total drainage area of 54 square miles. Reduction of flood flow in the stream is accomplished by the Farmington Reservoir Project, which prevents overflow of Littlejohn Creek floodwater into Duck Creek, and the Duck Creek Diversion which diverts floodwater from upper Duck Creek into the improved channel of Littlejohn Creek. Approximately half of the Duck Creek drainage area lies above the Duck Creek Diversion Dam. The upstream area, about 28 square miles in extent, lies below 500 feet in elevation and is a typical foothill area, with an overall streambed slope of about 20 feet per mile. Downstream of the diversion structure the gently sloping flat valley floor is a poorly defined tributary drainage area. This creek, with or without upstream reservoirs has no effect on major flood flows in the San Joaquin River.

ATTACHMENT 2: DESCRIPTION OF RELATED FEDERAL FLOOD RISK MANAGEMENT PROJECTS

New Hogan Lake

New Hogan Lake was authorized by the Flood Control Act of 1944 (Public Law 534, December 22 1044, 78th Congress, 2nd Session). The project is located on the Calaveras River about 28 miles northeast of Stockton, Ca and comprises a rockfill dam with an impervious earth core and a maximum height of about 200 feet. The project also includes four dikes, with a maximum height of 18 feet, and a gated spillway to create a reservoir with a gross storage capacity of 325,900 acre-feet for flood control, irrigation and other water conservation purposes. Construction was initiated in May 1960, dam closure was made in November 1963, and the project was completed for operational use in June 1964.

Stockton and Mormon Channels (Diverting Canal)

Improvement of Stockton and Mormon Channels was authorized by the River and Harbor Act of June 13, 1902 (H. Doc. 152, 55th Congress, 3d Session, and Annual Report for 1899, p. 3188), to provide for diversion of the waters of Mormon Slough before reaching Mormon and Stockton Channels, for the purpose of preventing deposits of material in the navigable portions of Mormon and Stockton Channels and to divert flood flows past the city of Stockton, California. The results were obtained by construction of (1) a dam across Mormon Slough; (2) a diverting canal 150 feet wide, extending 4.63 miles to the north branch of the Calaveras River; (3) enlargement of the Calaveras River to cross-sectional area of 1,550 square feet, thence to its mouth at San Joaquin River, 5 miles; and (4) a levee along the left bank of the diverting canal and Calaveras River, using material excavated for the channel enlargement.

Construction of new work was initiated in November 1908; the initial construction phase was completed in September 1910. No further new work was accomplished until fiscal year 1922; the project was completed in fiscal year 1923. Most of the silt formerly deposited in Stockton and Mormon Channels is diverted by this canal, obviating serious inconveniences to navigation in the harbor area.

Federal maintenance of these channels for navigation purposes has been discontinued due to completion of levee and channel improvements constructed under provisions included in the Mormon Slough, Calaveras River, project authorized by the Flood Control Act of 1962 (Public Law 874, October 23, 1962, 87th Congress, 2d Session). No Federal maintenance costs have been incurred since Fiscal Year 1969. The project capacity was increased by the Mormon Slough project which was completed in 1971. The Mormon Slough project is described below.

Mormon Slough Project

The Mormon Slough project was authorized by the Flood Control Act of 1962 (Public Law 874, October 23, 1962, 87th Congress, 2nd Session). The project provides for the improvement of the Calaveras River system between the town of Bellota and the city of Stockton, California, and consists of minor channel enlargement of Mormon Slough between Bellota and Jack Tone Road; substantial channel enlargement of lower Mormon Slough and the Diverting Canal; new levees along the north bank of the Diverting Canal, along both banks of lower Mormon Slough, and along the south bank of Potters Creek between Jack Tone Road and Mormon Slough; and bank protection on lower Calaveras River levee. The project is an element of the comprehensive development of the Calaveras River basin, contains the flood flows which originate in the area downstream from New Hogan Reservoir and contains the flood control releases for efficient operation of that reservoir.

Preconstruction planning was initiated in January 1964. Construction was initiated in October 1967. Work was substantially completed in February 1970; remaining miscellaneous minor work was completed in December 1971. Project design flows are described in Table x.

Farmington Dam and Reservoir

Farmington Dam was authorized by the Flood Control Act of 1944 (Public Law, 534, December 22, 1944, 78th Congress, 2nd Session). The project is located on Littlejohn Creek about 2.5 miles upstream from Farmington and about 18 miles east of Stockton, California and consists of an earthfill dam, maximum height 58 feet, and an ungated saddle spillway, creating a reservoir gross storage capacity of 52,000 acre feet (USACE,1974).

Also included in the Farmington project were appurtenant facilities for diverting Duck Creek floodwaters to Littlejohn Creek. However, several of the appurtenant features were later updated by the Little Johns Creek and Calaveras River Stream Group Project and the Duck Creek Project. All facilities are for the exclusive purpose of flood management.

The Duck Creek diversion is located about 0.5 miles east of Farmington California and approximately 3.5 miles downstream from Farmington Dam. The diversion works consist of a low compacted earth dike across Duck Creek with on 72" gated and one 60" ungated outlet discharging into Duck Creek, and an ungated concrete spillway 73 feet long discharging into the diversion channel. According to exhibit B of the operations and maintenance manual, the 72" gate is to remain fully open unless closure is authorized or directed by the District Engineer, Sacramento District, Corps of Engineers (USACE, 1952).

The Duck Creek Diversion Unit also includes dike "B" built across the North Branch of Duck Creek approximately 4 miles downstream from the diversion works; and dike "C" built across the North Branch of Duck Creek approximately 9 miles downstream from the diversion works and just upstream from Jack Tone Road.

Construction was initiated in July 1949; the main dam and spillway were completed in June 1951; the Duck Creek channel improvements were completed in November 1951; and the downstream improvements along Littlejohn Creek were completed in May 1955. Enlargement of the Duck Creek channel downstream of the diversion structure as part of the later Duck Creek Project was authorized under Public Law 685, 84th Congress, 2nd Session. The Duck Creek project is described below.

Duck Creek Project

The Duck Creek Project is a small tributary of the San Joaquin River south of the City of Stockton, San Joaquin County, lying between the Calaveras River-Mormon Slough system and Littlejohn Creek. The Duck Creek Channel extends from the Duck Creek Diversion (Unit of the Farmington Project) located about 0.5 miles northeast of Farmington California and meanders downstream a distance of about 20 miles to French Camp Slough. Authority to improve the Duck Creek channel was approved by the Chief of Engineers under the small flood control project program authorized by Section 205 of the 1948 Flood Control Act as amended by Public Law 685, 84th Congress, 2nd Session. The project works consist of channel improvements along approximately 20 miles of the Duck Creek channel from 1/2 mile upstream of Escalon-Bellota Road to French Camp Slough. The project includes a short reach of levee on the lower end of Duck Creek along the left and right banks. The design flows are 700 cfs from the Diversion Dam to Mariposa Road and 900cfs below the diversion dam. Construction of the project was initiated May 1965 and completed by January 1967.

Lower San Joaquin River and Tributaries Project

Improvement of lower reaches of the San Joaquin River and Tributaries was authorized by the Flood Control Act of 1944 (Public Law 534, December 22, 1944, 78th Congress, 2nd Session), as modified by Public Law 327, 84th Congress, 1st Session). The project provided for improvement by the Federal Government of the existing channel and levee system on the San Joaquin River from the delta upstream to the mouth of Merced river, and on the lower reaches of the Stanislaus and Tuolumne Rivers, by raising and strengthening of existing levees, construction of new levees, revetment of river banks where required, and removal of accumulated snags in the main river channel. The project also provided for protection of flood plain areas about the mouth of Merced River through local interests construction of levee and channel improvements. The Upper Delta is defined roughly as that portion lying within the influence of flood flows while the lower Delta is that portion influenced mainly by tides. The line of demarcation is considered to be the downstream limits of the San Joaquin Flood Control Project and passes across the Delta from the confluence of the Stockton Deep water ship Channel and the San Joaquin River at the Port of Stockton, to Williams Bridge on Middle River, and to the junction of Paradise Cut and Salmon Slough with Grant Line Canal near Tracy.

The local interest plan of improvement was coordinated with that of the Federal Government to insure the effectiveness of the Federal portion of the projects. In addition to bearing the cost of improvements as required along the San Joaquin River upstream of the mouth of Merced River, Local interests were required for the Federal improvement downstream from Merced River, to furnish flowage rights to overflow certain lands along the San Joaquin River, to furnish all lands, easements, and rights-of-way for construction of improvement of levees; to accomplish all necessary utility alterations and relocations; to hold and save the United States free from damages due to the construction works and their subsequent maintenance and operation; and to maintain all levees and channel improvements after completion in accordance with regulations prescribed by the Secretary of the Army.

Federal construction was initiated in June 1956 and was completed in November 1968 except for the left bank levee along the San Joaquin River, Tuolumne to Merced River reach, which at that time was in the “inactive” category. This work was restored to “active” status on 25 June 1969 as required assurances of local cooperation for the reach were furnished after a change in land ownership. Contract for construction of this reach was initiated in November 1971 and completed in September 1972. The State of California has completed construction of the non-federal portion of the project above the mouth of the Merced River, comprising about 193 miles of new levees, including appurtenant features and about 80 miles of surfacing of existing levees.

Friant Dam

Friant Dam was authorized by the River and Harbor Act (Public Law No. 392) of August 26, 1937 (50 Stat. 850), and the River and Harbor Act of October 17, 1940 (ch 895, 54 Stat. 1198, 1199) extended the authorization to include irrigation distribution systems. The project is located about 25 miles northeast of Fresno and an equal distance east of Madera. It is a concrete gravity structure, 319 feet high and 3,488 feet long at the crest. The spillway is 332 feet wide and is located near the center of the dam. It has three 100 by 18-foot drum gates and a discharge capacity of 83,000 cfs at gross pool elevation.

Initial construction was started in October of 1939 and was completed in November 1942. Work deferred during the war, including spillway gates, outlet valves, Friant-Kern Canal stilling basin, etc., was again started in March of 1946 and the project was completed for operation in 1949.

Big Dry Creek Dam

Big Dry Creek Dam was authorized by the Flood Control Act of 1941 (Public Law 288, August 18, 1941, 77th Congress, 1st Session). The project is located about 10 miles northeast of Fresno, California, and about 4 miles northeast of Clovis, California and comprises and earthfill dam across the channel of Big Dry Creek, with a maximum height of 40 feet, creating a reservoir with a maximum capacity of 16,250 acre-feet, all for flood control, together with appurtenant diversion facilities both upstream and downstream from the dam. Construction of the project was initiated in April 1947 and completed in February 1948. Construction of remedial work consisting of erosion control structures to control side-hill erosion was initiated in October 1952 and completed in March 1955.

Comanche Dam

Federal participation in the construction of Comanche Dam was authorized by the Flood Control Act of 1960 (Public Law 86-645, 14 July 1960, 86th Congress, 2d Session). Comanche Dam and Reservoir is a multiple-purpose dam and reservoir on the Mokelumne River about 20 miles northeast of Stockton. The dam and reservoir was constructed by the East Bay Municipal Utility District which owns and operates the project facilities. Federal interest in the project is in the flood protection afforded by the dam and reservoir commensurate with the flood control benefits to be derived. The project comprises a rock fill dam with impervious earth core, maximum height 171 feet, together with six dikes totaling 19,250 feet in length and a gated spillway, creating a reservoir gross storage capacity of 431,500 acre-feet for flood control and water supply.

In consideration of the Federal contribution toward the first cost of Comanche Reservoir, the East Bay Municipal Utility District provides a flood-control reservation of 200,000 acre-feet, under an agreement with the Department of the Army providing for operation of the reservoir in such manner as will produce the flood-control benefits upon which the monetary contribution is predicated, and will operate the flood-control reservation in accordance with the rules and regulations prescribed by the Secretary of the Army.

The cost allocation for the project was approved by the President on 9 March 1962. Contract for Federal payment for flood control benefits to be attained was consummated 19 March 1962 with the East Bay Municipal Utility District and approved by the Secretary of the Army 19 April 1962. Contract for construction of the main dam and appurtenances was awarded in March 1962; dam closure was completed 7 November 1963. The project was operationally completed in April 1964.

Los Banos Dam

Los Banos Dam was authorized by the Central Valley Project, California Act of 1960 (Public Law 488, June 3, 1960, 86th Congress, 2nd Session) and was constructed by the US Bureau of Reclamation, with funds contributed in part by the Federal Government in the interest of flood control, and are operated by the State of California. The project is located on Los Banos Creek, a west side tributary to San Joaquin River, approximately seven miles southwest of the small city of Los Banos in Merced County, California and comprises of a earthfill dam, with a maximum height of 167 feet, creating a reservoir with a maximum capacity of 34,600 acre-feet, most of which is for flood protection, with a provision of a pool for recreation and other purposes. There is also an uncontrolled concrete chute spillway located in the left abutment of the dam with a discharge capacity of 8,600 cfs.Outlet works, including an intake structure, conduit, emergency gate, and control gates are located in the left abutment of the dam and discharge the water into a stilling basin which, in turn, empties into the existing channel of Los Banos Creek downstream from the structure. Construction of the project began in May 1964 and completed by November 1965.

New Exchequer Dam

New Exchequer Dam was authorized by the Flood Control Act of 1960 (Public Law 645, July 14th, 1960, 86th Congress, 2nd Session). The project is located in the southern half of the Central Valley in Mariposa County, California. It is on the Merced River about 60 miles above its confluence with the San Joaquin River. New Exchequer Dam and Reservoir were constructed for the purposes of irrigation, power, recreation, and flood control providing. The reservoir includes a maximum of 400,000 acre-feet of flood control space. New Exchequer Reservoir has a capacity of 1,024,600 acre-feet. The dam is a rockfill dam, concrete faced with a height of 490 feet and is located immediately downstream from the old concrete Exchequer Dam, which is incorporated into the upstream toe of the embankment. A dike of similar gravel fill construction is located about ¾ of a mile northwest of New Exchequer Dam. A spillway, located approximately one mile northwest of the right abutment of New Exchequer Dam consists of a gated spillway and an ungated emergency spillway, each with a concrete ogee crest. The total combined discharge capacity of the gated and emergency spillways is 375,000 cfs. The outlet works consists of a single conduit under the right abutment of both the old and new portions of the dam. Construction of the project was initiated in June 1964 and completed in December 1967.

Don Pedro Dam

Don Pedro Dam was authorized by the Flood Control Act of 1944 (Public Law 534, December 22nd, 1944, 78th Congress, 2nd Session). The project is located on the Toulumne River about 35 miles east of Modesto. The dam is a combination rock and earthfill dam with a maximum height of 585 feet and a total capacity of 2,030,000 acre-feet which is primarily to store irrigation water and has additional benefits including power generation, flood control, and recreation. A spillway located on the abutment ridge west of the dam, consists of both a gated spillway and an ungated emergency spillway, each with a long concrete ogee section. The total combined discharge capacity of the spillway is 472,500 cfs. The outlet works is located in a concrete plug centered approximately on the axis of the dam. Three separate parallel outlets are provided, each controlled by two high-pressure slide gates in tandem. The combined capacity of the three outlets is 7,370 cfs. Construction of the project was initiated in August 1967 and completed in March 1971.

Buchanan Dam

Buchanan Lake was authorized by the Flood Control Act of 1962 (Public Law 874, 23 October 1962, 87th Congress, 2d Session). The project provides for construction of a dam on Chowchilla River, about 16 miles northeast of the city of Chowchilla, California, to create a reservoir with gross storage capacity of about 150,000 acre-feet for flood control, irrigation, recreation, and other purposes. The project plan provides for approximately 20 miles of levee and channel improvements along Ash and Berenda Sloughs, distributaries of Chowchilla River. Construction of the project was initiated in June 1972 and completed in June 1978.

Hidden Dam and Lake

Hidden Dam and Lake was authorized by the Flood Control Act of 1962 (Public Law 874, 23 October 1962, 87th Congress, 2d Session). The project provides for construction of a dam on Fresno River, about 15 miles northeast of Madera, California, to create a reservoir with gross storage capacity of about 90,000 acre-feet for flood control, irrigation, recreation, and other purposes. The project plan as authorized also provides for approximately 13.3 miles of levee and channel improvements on Fresno River downstream from the dam site. Construction of the project was initiated in June 1972 and completed in June 1978.

New Melones Dam

New Melones Lake was authorized by the Flood Control Act of 1944 (Public Law 534. December 22, 1944. 78th Congress, 2d Session), as modified by the Flood Control Act of 1962 (Public Law 874, October 23, 1962, 87th Congress, 2d Session). The project is located on Stanislaus River, about 35 miles northeast of Modesto, California. The project plan provides for construction of a 625 foot high earth and rockfill dam to create a reservoir with a gross storage capacity of 2,400,000 acre-feet for flood control, irrigation, power, recreation, fish and wildlife and water quality control. The plan of improvement also includes construction of a 300,000 KW capacity hydroelectric power plant immediately below the dam. Construction of the project was initiated in 1966 and completed in October 1978.

ATTACHMENT 3: 2011 INVENTORY DEVELOPMENT

Data Cleaning

Tax assessor data containing geospatially referenced land parcel information was reviewed in preparation for the structure inventory described in task three below. This data included the address, geospatial location, square footage and land use for each parcel located in the Lower San Joaquin Dam break floodplain maps. Problematic data such as duplicate entries and missing observations were identified and corrected or deleted in order to facilitate unbiased sampling and structure valuation work.

Create Samples and Inventory Maps

Stratified random samples containing properties to be included in the structure inventory were generated using. Samples were stratified according to land use type. Land use type data taken from the tax assessor dataset. Sample sizes were chosen based on the number of working days allotted for the structure inventory. Once all the properties included in the structure inventory had been selected a driving route for the inventory was created using Google Fusion Tables.

Performed Structure Characteristics Survey

Four Economists (in two vehicles) surveyed 833 separate parcels based upon observations from the nearest accessible public road or access point. Parcels were located using addresses and geospatial references on Google Maps as needed and seven characteristics were assessed: bad address, first floor elevation, stories, construction class, construction quality, condition, and Marshall & Swift Use (MS Use) category. A parcel is marked as a bad address if no structure is present or the parcel cannot be located. First floor elevation is the elevation in half-foot increments from the bottom of the front doorway to ground level. Stories are the number of stories in the surveyed structures. Construction class, quality, condition and MS Use follow guidance from the *Marshall Valuation Service* and in all cases were limited to exterior surveys of the structure. Construction class is the type of framing, walls, floors and roof structures, and fireproofing. Class is represented by B, C, D or S. Construction quality is judged by materials, workmanship, and complexity and is represented by Low Cost, Fair, Average, Good, Very Good, and Excellent. Condition is the level of accumulated depreciation apparent to the structure exterior, which is also used as a proxy for interior depreciation and is represented by Dilapidated, Poor, Fair, Average, Good, Very Good, and Excellent. The MS Use category is the apparent structure function or use based on indicators such as signage, other structure uses in the vicinity, building type, etc. It is represented by distinct uses that captured the generic function of all the structures surveyed. See *Marshall Valuation Service* for further details.

Performed Structure Square Footage Survey

The majority of commercial, industrial, and public parcels in the assessor's database did not have square footage. A number of these parcels did in fact have structures. However, structure values for these structures were estimated directly and therefore, no adjustment the tax assessor square footage data was necessary. Most of the residential structures had square footage; however, the square footage needed to be tested for accuracy. To accomplish this aerial surveys were performed using GIS and Google Earth Pro. Structures were randomly sampled from the surveyed parcels shown to have structures present in Step 3. Since aerial resolution in GIS was judged to be insufficient for accurate square footage estimates, it was used to verify the location of parcels only. Google Earth Pro has superior image resolution and was used to make the square footage measurements by calculating the area of a polygon that traces the roof line of the structure. Structure square footage estimates taken from tax assessor data and aerial surveys were relatively similar. Therefore no adjustment was made to tax assessor square footage estimates.

Applied Characteristics to Non-Surveyed Parcels

Survey results showed substantial errors in the assessor's data on broad use category (Residential, Commercial, Industrial, Agricultural, and Public) and whether a structure is present on the parcel (i.e., zero vs. nonzero square footage. The following steps were taken to address these errors and to apply the survey characteristics to the non-sampled data.

SEPARATED SURVEY DATA BY ZERO/NONZERO SQUARE FOOTAGE - This was done to create separate distributions for these two types of parcels under the assumption that parcels listed with positive square footage and parcels listed with zero square footage are systematically different on average. For instance, during the surveys we noticed that some recent housing developments contained finished or nearly-finished structures that were assigned zero square footage by the assessor. To account for this and other potential systematic differences a separate distribution of characteristics was made for non-surveyed parcels the assessor listed with zero and nonzero square footage (i.e., without and with a structure on the parcel).

ADJUSTED BROAD USE CATEGORY – The surveyed broad use category was compared to the assessor broad use category. The assessor broad use category was adjusted based on survey results. For instance, among 190 surveyed parcels the assessor data labeled commercial (and with zero square footage), only 82% were demonstrated to be commercial properties during the survey. The remaining 18% were industrial, public, or residential. Therefore, 18% of the non-surveyed parcels labeled commercial by the assessor were randomly adjusted to be industrial, public, or residential. This broad use adjustment was made to all the non-surveyed parcels.

ADJUSTED STRUCTURE COUNT - As explained in step I, the surveyed parcels were separated into nonzero and zero square footage. Alternatively, these can be thought of as parcels with and without square footage. Most parcels we surveyed that the assessor labeled with square footage (nonzero square feet) had a structure present. Parcels the assessor labeled without square footage (zero square feet) sometimes had a structure and sometimes did not. The share that did have a structure versus the share that did not were calculated and these two percentages were used to randomly reduce the number of non-sampled parcels that the assessor incorrectly labeled without a structure (i.e. zero square footage). Likewise, a small number of non-sampled parcels with square footage in the assessor's records were removed from the population, based on percentage of sampled parcels the assessor incorrectly labeled as having a structure (i.e. positive square footage).

Adjusted Characteristics – Characteristics of the surveyed structures were applied to the non-survey structures. @Risk was used to assign a number of stories to each non-sampled structure. The @Risk number of stories simulations were based strata specific (broad category) sampled structure number of stories probability distributions. Each non-sampled structure was the average first floor elevation of the strata to which it belongs. However, a triangular first floor elevation distribution was entered into HEC-FDA, based on the survey results. As a result the (average) first floor elevation assigned to each non-sampled structure in the database will vary (based on this triangular distribution) in each HEC-FDA simulation.

Structure Value – Non-sampled structures were each assigned a structure value. Non-sampled structures with a square footage entry (based on tax assessor records and task 5.I. above) were assigned a structure value equal to the product of the square footage entry and the within strata (broad category) average per square foot structure value. Non-sampled structures without square footage entries were assigned the within strata average structure value. Again, a triangular distribution, based on the sampled distribution, was entered into HEC-FDA. Thus each structure's value will vary in each HEC-FDA simulation, based on this triangular distribution.

Valued Structures and Contents

Once the non-surveyed parcels were assigned the characteristics from the survey results, per square foot depreciated replacement costs for each structure were determined. This per square foot depreciated structure replacement cost was then applied to each structure's recorded square footage to obtain its depreciated structure replacement cost. Structure content were calculated using the following ratios: Residential structure contents were valued at 50% of the structure value; Industrial, commercial, agricultural and public structure contents were valued using the methodology described in *Analysis of Nonresidential Content-to-Structure Ratios and Depth-Damage Functions for Flood Damage Reduction Studies*. Using this method structure contents are a ratio of structure value that varies by structure use category.

Prepared Data for HEC-FDA

The content-to-structure ratios and content depth damage curves were taken from *Analysis of Nonresidential Content-to-Structure Ratios and Depth-Damage Functions for Flood Damage Reduction Studies* and set up in a spreadsheet consistent with guidance from the *HEC-FDA User Manual* dated November 2008. To account for risk and uncertainty, error values were included in the HEC-FDA import spreadsheet file.

> **CONTENT-TO-STRUCTURE RATIO ERROR** — TAKEN from Analysis of Nonresidential Content-to-Structure Ratios and Depth-Damage Functions for Flood Damage Reduction Studies
>
> **STRUCTURE VALUATION ERROR** – triangular distribution based on the distribution of sampled structure values.
>
> **FIRST-FLOOR ELEVATION ERROR** – triangular distribution based on the distribution of sampled structure values.

Key Assumptions

- Since interior housing characteristics could not be observed, external and observable characteristics were only used to assess the surveyed structures and assign structure valuations.
- First floor elevation, stories, construction class, construction quality, condition, and Marshall & Swift Use (MS Use) category completely and accurately define the characteristics of the surveyed structures necessary to estimate depreciated value per square foot.
- Observations were unbiased in a manner that would not lead to upward or downward depreciated structure valuations on average.
- Roof line profile measured from aerial imagery approximates actual structure square footage but is slightly upwardly biased due to roof overhangs, contiguous porch area, etc. Thus tax assessor records with square footage entries within ~25% of aerial square footage estimates are approximately equivalent.
- Parcels the assessor listed with a structure are systematically different from parcels the assessor listed without a structure. This assumption appears correct because the distribution of characteristics between the two parcel types are noticeably different for most broad use categories.
- The surveyed structures were representative of the non-surveyed structures across all characteristics evaluated and the sample sizes were sufficient to extrapolate surveyed characteristics to the non-surveyed parcels.
- Structure value is not correlated with depth of flooding.
- Content value varies proportionally with structure value and, on average, is equal to a fixed percentage of structure value

The three error terms—content-to-structure error, structure valuation error, first-floor elevation error—adequately address the risk and uncertainty inherent in this model.

ATTACHMENT 4: DEPTH-PERCENT DAMAGE CURVES

DEPTH-PERCENT DAMAGE FOR STRUCTURES BY OCCUPANCY TYPE

OCCUPANCY TYPE	INUNDATION DEPTH IN FEET																			
	-1	-0.5	0	0.5	1	1.5	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Automobiles	0%	0%	0%	2.8%	21.8%	31.2%	40.5%	56.9%	71.1%	83.2%	91.9%	96.1%	99.2%	100%	100%	100%	100%	100%	100%	100%
Commercial Auto Sales 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Commercial Auto Sales 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Commercial Fast Food Rest 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Commercial Fast Food Rest 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Commercial FoodRetail 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Commercial FoodRetail 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Commercial Grocery Store 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Commercial Grocery Store 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Commercial Medical 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Commercial Medical 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Commercial Office 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Commercial Office 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Commercial Restaurants 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Commercial Restaurants 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Commercial Retail 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Commercial Retail 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Commercial ServiceAuto 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Commercial ServiceAuto 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Commercial Shopping Center 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Commercial Shopping Center 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Farm Buildings Including Residence	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Full Service Auto Dealership 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Full Service Auto Dealership 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Furniture Store 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Furniture Store 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Hospital 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Hospital 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Hotel 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Hotel 2-story	2.6%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Industrial Heavy Manufacture 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Industrial Heavy Manufacture 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Industrial Light 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Industrial Light 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Industrial Warehouse 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Industrial Warehouse 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Mobile Home Single/Double	6.4%	7.3%	9.9%	43.4%	44.7%	45.0%	45.7%	45.9%	50.0%	65.6%	65.6%	66.0%	66.0%	66.0%	66.0%	66.0%	66.0%	66.0%	66.0%	66.0%
MultiFamily Residential 1-story	2.5%	8.0%	13.4%	18.4%	23.3%	27.7%	32.1%	40.1%	47.1%	53.2%	58.6%	63.2%	67.2%	70.5%	73.2%	75.4%	77.2%	78.5%	79.5%	80.2%
MultiFamily Residential 2-story	3.0%	6.2%	9.3%	12.3%	15.2%	18.1%	20.9%	26.3%	31.4%	36.2%	40.7%	44.9%	48.8%	52.4%	55.7%	58.7%	61.4%	63.8%	65.9%	67.7%
Public and Private Schools 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Public and Private Schools 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Public Church 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Public Church 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Public Government Building 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Public Government Building 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Public Recreation/Assembly 1-story	0%	3.5%	7.0%	11.7%	16.3%	20.5%	24.7%	27.7%	29.6%	30.9%	39.8%	42.8%	43.3%	44.8%	45.8%	47.0%	47.3%	48.2%	48.8%	49.9%
Public Recreation/Assembly 2-story	0%	1.3%	2.5%	3.8%	5.0%	7.6%	10.1%	15.3%	17.1%	18.9%	21.5%	22.8%	22.8%	24.1%	26.1%	40.4%	43.3%	46.2%	46.2%	49.1%
Single Family Residential 1-story	2.5%	8.0%	13.4%	18.4%	23.3%	27.7%	32.1%	40.1%	47.1%	53.2%	58.6%	63.2%	67.2%	70.5%	73.2%	75.4%	77.2%	78.5%	79.5%	80.2%
Single Family Res 1-story w/bsmt	19.4%	22.5%	25.5%	28.8%	32.0%	35.4%	38.7%	45.5%	52.2%	58.6%	64.5%	69.8%	74.2%	77.7%	80.1%	81.1%	81.1%	81.1%	81.1%	81.1%
Single Family Residential 2-story	3.0%	6.2%	9.3%	12.3%	15.2%	18.1%	20.9%	26.3%	31.4%	36.2%	40.7%	44.9%	48.8%	52.4%	55.7%	58.7%	61.4%	63.8%	65.9%	67.7%
Single Family Res 1-story w/bsmt	13.9%	15.9%	17.9%	20.1%	22.3%	24.7%	27.0%	31.9%	36.9%	41.9%	46.9%	51.8%	56.4%	60.8%	64.8%	68.4%	71.4%	73.7%	75.4%	76.4%
Single Family Residential Split Level	6.4%	6.8%	7.2%	8.3%	9.4%	11.2%	12.9%	17.4%	22.8%	28.9%	35.5%	42.3%	49.2%	56.1%	62.6%	68.6%	73.9%	78.4%	81.7%	83.8%
Single Family Res 1-story w/bsmt	14.2%	16.4%	18.5%	20.9%	23.2%	25.7%	28.2%	33.4%	38.6%	43.8%	48.8%	53.5%	57.8%	61.6%	64.8%	67.2%	68.8%	69.3%	69.3%	69.3%

DEPTH-PERCENT DAMAGE FOR CONTENTS BY OCCUPANCY TYPE

	INUNDATION DEPTH IN FEET																			
OCCUPANCY TYPE	**-1**	**-0.5**	**0**	**0.5**	**1**	**1.5**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
Automobiles	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%
Commercial Auto Sales 1-story	0%	0%	0%	18.1%	34.9%	59.2%	78.4%	90.4%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Commercial Auto Sales 2-story	0%	0%	0%	15.5%	29.3%	40.7%	49.8%	49.8%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Commercial Fast Food Rest 1-story	0%	0%	0%	12.0%	23.3%	38.6%	59.4%	90.2%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Commercial Fast Food Rest 2-story	0%	0%	0%	10.1%	19.6%	26.5%	37.7%	49.7%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Commercial FoodRetail 1-story	0%	0%	0%	15.8%	29.3%	43.1%	72.2%	96.2%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Commercial FoodRetail 2-story	0%	0%	0%	13.3%	24.6%	29.7%	45.8%	49.8%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Commercial Grocery Store 1-story	0%	0%	0%	17.6%	32.0%	47.6%	69.8%	88.6%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Commercial Grocery Store 2-story	0%	0%	0%	14.8%	26.9%	32.8%	44.4%	48.8%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Commercial Medical 1-story	0%	0%	0%	16.8%	33.5%	51.3%	72.8%	88.7%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Commercial Medical 2-story	0%	0%	0%	14.1%	28.1%	35.3%	46.3%	48.9%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Commercial Office 1-story	0%	0%	0%	18.1%	34.9%	59.2%	78.4%	90.4%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Commercial Office 2-story	0%	0%	0%	15.5%	29.3%	40.7%	49.8%	49.8%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Commercial Restaurants 1-story	0%	0%	0%	15.0%	29.6%	52.6%	77.3%	96.1%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Commercial Restaurants 2-story	0%	0%	0%	12.6%	24.8%	36.2%	49.1%	49.8%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Commercial Retail 1-story	0%	0%	0%	69.3%	80.4%	86.8%	95.0%	96.5%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Commercial Retail 2-story	0%	0%	0%	14.0%	19.1%	25.1%	31.5%	35.7%	45.1%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Commercial ServiceAuto 1-story	9.1%	9.1%	9.9%	17.7%	23.2%	37.5%	42.8%	67.4%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Commercial ServiceAuto 2-story	7.6%	7.6%	8.3%	14.8%	19.5%	25.8%	27.2%	37.1%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Commercial Shopping Center 1-story	0%	0%	0%	20.5%	32.8%	47.6%	58.5%	71.9%	97.2%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Commercial Shopping Center 2-story	0%	0%	0%	17.2%	27.5%	32.7%	37.2%	39.6%	48.6%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Farm Buildings Including Residence	0%	0%	0%	12.9%	30.1%	42.8%	56.0%	75.6%	99.2%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Full Service Auto Dealership 1-story	5.3%	5.3%	5.8%	16.2%	25.3%	41.2%	52.1%	72.0%	96.2%	99.0%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Full Service Auto Dealership 2-story	4.4%	4.4%	4.8%	13.6%	21.3%	28.3%	33.1%	39.6%	48.1%	49.5%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Furniture Store 1-story	0%	0%	0%	69.3%	80.4%	86.8%	95.0%	96.5%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Furniture Store 2-story	0%	0%	0%	35.8%	41.5%	44.8%	49.1%	49.8%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Hospital 1-story	0%	0%	0%	16.8%	33.5%	51.3%	72.8%	88.7%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Hospital 2-story	0%	0%	0%	14.1%	28.1%	35.3%	46.3%	48.9%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Hotel 1-story	0%	0%	0%	12.0%	23.3%	38.6%	59.4%	90.2%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Hotel 2-story	0%	0%	0%	12.6%	24.8%	36.2%	49.1%	49.8%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Industrial Heavy Manufacture 1-story	0%	0%	0%	5.8%	16.1%	28.9%	41.0%	56.4%	85.4%	92.5%	97.1%	98.1%	98.1%	99.1%	100%	100%	100%	100%	100%	100%
Industrial Heavy Manufacture 2-story	0%	0%	0%	4.9%	13.6%	19.9%	26.0%	31.1%	42.7%	46.2%	48.6%	49.1%	49.1%	49.5%	50.0%	59.6%	72.3%	96.3%	100%	100%
Industrial Light 1-story	0%	0%	0%	19.1%	35.2%	48.9%	64.2%	74.8%	91.8%	96.3%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Industrial Light 2-story	0%	0%	0%	16.0%	29.6%	33.6%	40.8%	41.2%	45.9%	48.1%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Industrial Warehouse 1-story	0%	0%	0%	11.3%	23.4%	36.5%	54.9%	69.0%	84.2%	95.7%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Industrial Warehouse 2-story	0%	0%	0%	9.5%	19.6%	25.1%	34.8%	38.0%	42.1%	47.8%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Mobile Home Single/Double	0%	0%	0%	28.1%	38.3%	44.8%	56.4%	68.6%	79.9%	89.6%	89.7%	89.7%	89.7%	89.7%	89.7%	89.7%	89.7%	89.7%	89.7%	89.7%
MultiFamily Residential 1-story	2.4%	5.3%	8.1%	10.7%	13.3%	15.6%	17.9%	22.0%	25.7%	28.8%	31.5%	33.8%	35.7%	37.2%	38.4%	39.2%	39.7%	40.0%	40.0%	40.0%
MultiFamily Residential 2-story	1.0%	3.0%	5.0%	6.9%	8.7%	10.5%	12.2%	15.5%	18.5%	21.3%	23.9%	26.3%	28.4%	30.3%	32.0%	33.4%	34.7%	35.6%	36.4%	36.9%
Public and Private Schools 1-story	0%	0%	0%	12.6%	21.9%	33.4%	47.3%	66.7%	76.1%	87.8%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Public and Private Schools 2-story	0%	0%	0%	10.6%	18.4%	23.0%	30.1%	36.8%	38.0%	43.9%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Public Church 1-story	0%	0%	0%	22.7%	32.9%	45.8%	74.8%	85.5%	98.8%	98.8%	98.8%	98.8%	98.8%	98.8%	98.8%	98.8%	99.3%	100%	100%	100%
Public Church 2-story	0%	0%	0%	19.1%	27.6%	31.5%	47.1%	47.1%	49.4%	49.4%	49.4%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Public Government Building 1-story	0%	0%	0%	18.1%	34.9%	59.2%	78.4%	90.4%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Public Government Building 2-story	0%	0%	0%	15.7%	30.1%	42.1%	49.9%	49.9%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	71.2%	96.2%	100%	100%
Public Recreation/Assembly 1-story	0%	0%	0%	24.5%	37.8%	57.3%	74.6%	94.7%	98.0%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
Public Recreation/Assembly 2-story	0%	0%	0%	20.6%	31.7%	39.4%	47.1%	49.0%	49.0%	50.0%	50.0%	50.0%	50.0%	50.0%	50.0%	59.6%	72.3%	96.3%	100%	100%
Single Family Residential 1-story	2.4%	5.3%	8.1%	10.7%	13.3%	15.6%	17.9%	22.0%	25.7%	28.8%	31.5%	33.8%	35.7%	37.2%	38.4%	39.2%	39.7%	40.0%	40.0%	40.0%
Single Family Res 1-story w/bsmt	13.2%	14.6%	16.0%	17.5%	18.9%	20.4%	21.8%	24.7%	27.4%	30.0%	32.4%	34.5%	36.3%	37.7%	38.6%	39.1%	39.1%	39.1%	39.1%	39.1%
Single Family Residential 2-story	1.0%	3.0%	5.0%	6.9%	8.7%	10.5%	12.2%	15.5%	18.5%	21.3%	23.9%	26.3%	28.4%	30.3%	32.0%	33.4%	34.7%	35.6%	36.4%	36.9%
Single Family Res 1-story w/bsmt	10.1%	11.0%	11.9%	12.9%	13.8%	14.8%	15.7%	17.7%	19.8%	22.0%	24.3%	26.7%	29.1%	31.7%	34.4%	37.2%	40.0%	43.0%	46.1%	49.3%
Single Family Residential Split Level	2.2%	2.6%	2.9%	3.8%	4.7%	6.1%	7.5%	11.1%	15.3%	20.1%	25.2%	30.5%	35.7%	40.9%	45.8%	50.2%	54.1%	57.2%	59.4%	60.5%
Single Family Res 1-story w/bsmt	9.4%	10.5%	11.6%	12.7%	13.8%	15.0%	16.1%	18.2%	20.2%	22.1%	23.6%	24.9%	25.8%	26.3%	26.3%	26.3%	26.3%	26.3%	26.3%	26.3%

ATTACHMENT 5: WITHOUT-PROJECT ENGINEERING INPUTS

INDEX POINT LR1

Annual Chance Exceedance	WITHOUT PROJECT											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
0	0	0	0	0.00	0	0					0.00	0
0	0	0	0	3.18	*12.42*	*0.0000*					*12.42*	*0.0000*
95	0	0	0	0.00	17.00	0.0876	0	0	0	4.84	17.00	0.0876
50	21,899	1,773	1,773	7.50	19.80	0.2557	0	0	0	0.00	19.80	0.2557
20	0	0	0	0.00	22.40	0.4408	21,899	1,717	1,717	8.71	22.40	0.4408
10	79,122	7,757	7,757	14.21	**25.00**	**0.6114**	0	0	0	0.00	**25.00**	**0.6114**
4	124,892	9,142	9,142	15.44	25.01	1.0000	79,122	7,677	7,677	14.66	25.01	1.0000
2	167,074	10,129	10,129	16.23	0	0	124,892	9,031	9,031	15.76	0	0
1	216,499	13,871	13,871	18.93	0	0	167,074	10,012	10,012	16.48	0	0
0.5	273,861	15,724	15,724	22.58	0	0	216,499	13,767	13,767	18.93	0	0
0.2	363,117	16,625	16,625	23.66	0	0	273,861	15,535	15,535	22.59	0	0

INDEX POINT LR2

Annual Chance Exceedance	WITHOUT PROJECT											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
				0.00	0	0				0.00	0.00	0
	0	0	0	3.18	*12.00*	*0.0000*	0	0	0	4.84	*12.00*	*0.0000*
95		0	0	0.00	17.00	0.1287					17.00	0.1287
50	21899	1,771	1,771	7.60	21.50	0.3839	21,899	1,716	1,716	8.77	21.50	0.3839
20		0	0	0.00	24.65	0.5587					24.65	0.5587
10	79122	7,754	7,754	15.14	**27.80**	**0.6903**	79,122	7,669	7,669	15.70	**27.80**	**0.6903**
4	124892	9,143	9,143	16.47	28.81	1.0000	124,892	9,032	9,032	16.94	28.81	1.0000
2	167074	10,130	10,130	17.33			167,074	10,013	10,013	17.76		
1	216499	13,871	13,871	20.25			216,499	13,767	13,767	20.55		
0.5	273861	15,734	15,734	22.96			273,861	15,556	15,556	23.00		
0.2	363117	16,889	16,889	23.78			363,117	16,749	16,749	23.78		

INDEX POINT LR3

Annual Chance Exceedance	WITHOUT PROJECT											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
				0.00	0	0				0.00	0	0
	0	0	**0**	3.18	*18.53*	*0.0000*	0	0	0	4.84	*18.53*	*0.0000*
95		0		**0.00**	24.00	0.1472			0		24.00	0.1472
50	21,899	6,391	6,391	8.42	26.90	0.4782	21,899	6,394	6394	9.30	26.90	0.4782
20		0	**0**	**0.00**	28.95	0.8014			0		28.95	0.8014
10	79,122	25,165	25,165	18.21	**31.00**	**0.9999**	79,122	25,000	25000	18.45	**31.00**	**0.9999**
4	124,892	28,844	28,844	19.66	31.01	1.0000	124,892	28,707	28707	19.88	31.01	1.0000
2	167,074	31,599	31,599	20.64			167,074	31,449	31449	20.83		
1	216,499	42,793	42,793	23.93			216,499	42,596	42596	24.06		
0.5	273,861	51,601	51,601	25.56			273,861	51,431	51431	25.61		
0.2	363,117	57,151	57,151	26.78			363,117	57,058	57058	26.79		

INDEX POINT LR4

Annual Chance Exceedance	WITHOUT PROJECT											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
				0.00	0	0			0	0.00	0	0
	0	0	0	3.18	*18.60*	*0.0000*	0	0	0	4.84	*18.60*	*0.0000*
95		0		0.00	23.75	0.0538		0	0	0.00	23.75	0.0538
50	21,899	6,390	6,390	10.27	27.50	0.1144	21,899	6,392	6,392	10.82	27.50	0.1144
20		0	0	0.00	31.25	0.1719	0	0	0	0.00	31.25	0.1719
10	79,122	25,167	25,167	21.62	**33.90**	**0.2289**	79,122	25,002	25,002	21.73	**33.90**	**0.2289**
4	124,892	28,849	28,849	23.17	33.91	1.0000	124,892	28,712	28,712	23.27	33.91	1.0000
2	167,074	31,612	31,612	24.21			167,074	31,459	31,459	24.30		
1	216,499	42,800	42,800	27.78			216,499	42,602	42,602	27.83		
0.5	273,861	52,486	52,486	30.03			273,861	52,328	52,328	30.04		
0.2	363,117	63,467	63,467	31.33			363,117	63,389	63,389	31.33		

INDEX POINT LRTB

Annual Chance Exceedance	WITHOUT PROJECT											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
				0.00	0	0				0.00	0	0
	0	0	**0**	3.18	*18.60*	*0.0000*	0	0	0	4.84	*18.60*	*0.0000*
95		0		**0.00**	23.75	0.0029		0	0	0.00	23.75	0.0029
50	21,899	6,390	6,390	10.27	27.50	0.0131	21,899	6,392	6,392	10.82	27.50	0.0131
20		0	**0**	**0.00**	**27.78**	**0.0169**	0	0	0	0.00	**27.78**	**0.0169**
10	79,122	25,167	25,167	21.62	27.79	1.0000	79,122	25,002	25,002	21.73	27.79	1.0000
4	124,892	28,849	28,849	23.17	0.00	0.0000	124,892	28,712	28,712	23.27	0.00	0.0000
2	167,074	31,612	31,612	24.21	0	0	167,074	31,459	31,459	24.30	0	0
1	216,499	42,800	42,800	27.78			216,499	42,602	42,602	27.83		
0.5	273,861	52,486	52,486	30.03			273,861	52,328	52,328	30.04		
0.2	363,117	63,467	63,467	31.33			363,117	63,389	63,389	31.33		

INDEX POINT FL1

Annual Chance Exceedance	WITHOUT PROJECT											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
0	0	0	0	0.00	0	0	0	0			0	0
0	0	0	0	0.00	*9.36*	*0.0000*	0	0	0	4.84	*9.36*	*0.0000*
95	0	0	0	3.18	13.00	0.0610	0	0	0		13.00	0.0610
50	21,899	1,776	1,776	7.33	15.90	0.1282	21,899	1,720	1,720	8.62	15.90	0.1282
20	0	0	0	0.00	18.65	0.1917	0	0	0	0.00	**18.65**	**0.1917**
10	79,122	7,774	7,774	11.75	**21.40**	**0.2418**	79,122	7,690	7,690	12.83	21.40	0.2418
4	124,892	9,142	9,142	12.51	21.41	1.0000	124,892	9,031	9,031	13.51	21.41	1.0000
2	167,074	10,128	10,128	13.09	0	0	167,074	10,012	10,012	14.04	0	0
1	216,499	13,869	13,869	14.65	0	0	216,499	13,766	13,766	15.43	0	0
0.5	273,861	26,687	26,687	20.12	0	0	273,861	25,913	25,913	20.36	0	0
0.2	363,117	32,943	32,943	21.98	0	0	363,117	32,389	32,389	22.05	0	0

INDEX POINT FR1

Annual Chance Exceedance	WITHOUT PROJECT											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
	0	0	0	0.00	0	0	0	0	0	0.00	0	0
	0	0	0	0.00	*8.14*	*0.0000*	0	0	0	4.84	*8.14*	*0.0000*
95	0	0	0	3.18	12.96	0.0663	0	0	0		12.96	0.0663
50	21,899	1,776	1,776	7.33	***15.90***	***0.2537***	21,899	1,720	1,720	8.62	***15.90***	***0.2537***
20	0	0	0	0.00	18.84	0.5039	0	0	0	0.00	18.84	0.5039
10	79,122	7,774	7,774	11.75	**21.77**	**0.7183**	79,122	7,690	7,690	12.83	**21.77**	**0.7183**
4	124,892	9,142	9,142	12.51	21.78	1.0000	124,892	9,031	9,031	13.51	21.78	1.0000
2	167,074	10,128	10,128	13.09	0	0	167,074	10,012	10,012	14.04	0	0
1	216,499	13,869	13,869	14.65	0	0	216,499	13,766	13,766	15.43	0	0
0.5	273,861	26,687	26,687	20.12	0	0	273,861	25,913	25,913	20.36	0	0
0.2	363,117	32,943	32,943	21.98	0	0	363,117	32,389	32,389	22.05	0	0

INDEX POINT D3

Annual Chance Exceedance	WITHOUT PROJECT											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
	0	0	0	0.00	0	0	0	0	0	0.00	0	0
	0	0	0	0.00	*2.00*	*0.0000*	0	0	0	4.84	*2.00*	*0.0000*
95	0	0	0	3.18	6.00	0.0928	0	0	0	0.00	*6.00*	*0.0928*
50	21,899	2,424	2,424	7.70	8.50	0.2098	21,899	2,257	2,257	8.43	8.50	0.2098
20	0	0	0	0.00	11.00	0.3419	0	0	0	0.00	11.00	0.3419
10	79,122	9,864	9,864	9.30	**13.20**	**0.4593**	79,122	9,774	9,774	10.93	**13.20**	**0.4593**
4	124,892	11,158	11,158	9.70	13.21	1.0000	124,892	11,046	11,046	11.32	13.21	1.0000
2	167,074	12,298	12,298	9.90	0	0	167,074	12,175	12,175	11.54	0	0
1	216,499	15,920	15,920	10.10	0	0	216,499	15,792	15,792	11.76	0	0
0.5	273,861	28,712	28,712	12.12	0	0	273,861	27,834	27,834	13.60	0	0
0.2	363,117	33,013	33,013	13.01	0	0	363,117	31,429	31,429	14.40	0	0

INDEX POINT D-BS

Annual Chance Exceedance	WITHOUT PROJECT											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
0	0	0	0	0.00	0	0	0	0	0	0.00	0	0
0	0	0	0	3.18	*-3.50*	*0.0000*	0	0	0	4.84	*-3.50*	*0.0000*
95	0	0			6.00	0.0743	0	0	0		*6.00*	*0.0743*
50	21,899	2,424	2,424	7.70	10.00	0.2006	21,899	2,257	2,257	8.43	10.00	0.2006
20	0	0	0	0.00	14.00	0.5153	0	0	0	0.00	14.00	0.5153
10	79,122	9,864	9,864	9.29	**18.00**	**0.8532**	79,122	9,774	9,774	10.93	**18.00**	**0.8532**
4	124,892	11,158	11,158	9.70	18.01	1.0000	124,892	11,046	11,046	11.32	18.01	1.0000
2	167,074	12,298	12,298	9.90	0	0	167,074	12,175	12,175	11.54	0	0
1	216,499	15,920	15,920	10.10	0	0	216,499	15,792	15,792	11.76	0	0
0.5	273,861	28,712	28,712	12.12	0	0	273,861	27,834	27,834	13.60	0	0
0.2	363,117	33,013	33,013	13.01	0	0	363,117	31,429	31,429	14.40	0	0

INDEX POINT D4

Annual Chance Exceedance	WITHOUT PROJECT											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
0	0	0	0	0.00	0	0	0	0	0	0.00	0	0
0	0	0	0	3.18	*5.37*	*0.0000*	0	0	0	4.84	*5.37*	*0.0000*
95	0	0	0	0.00	11.89	0.1181	0	0	0	0.00	11.89	0.1181
50	6,901	3,792	3,792	8.35	14.20	0.2809	6,901	3,778	3,778	9.79	14.20	0.2809
20	15,360	9,487	9487	11.29	16.51	0.5062	15,360	9,486	9,486	12.36	16.51	0.5062
10	21,654	9,933	9,933	11.51	**18.82**	**0.8686**	**21,654**	9,933	**9,933**	**12.55**	**18.82**	**0.8686**
4	29,659	12,270	12,270	12.51	18.83	1.0000	29,659	12,270	12,270	13.44	18.83	1.0000
2	35,396	12,752	12,752	12.71			35,396	12,742	12,742	13.69		
1	40,815	15,346	15,346	13.77			40,815	15,346	15,346	14.59		
0.5	45,896	15,736	15,736	14.11			45,896	15,719	15,719	14.96		
0.2	52,080	19,117	19,117	16.30			52,080	19,118	19,118	17.03		

INDEX POINT D5

Annual Chance Exceedance	WITHOUT PROJECT											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
0	0	0	0	0.00	0	0	0	0	0	0.00	0	0
0	0	0	0	3.18	*4.10*	*0.0000*	0	0	0	4.84	*4.10*	*0.0000*
95	0	0	0	0.00	7.20	0.0869	0	0	0	0.00	7.20	0.0869
50	6,901	3,784	3,784	8.24	10.00	0.1872	6,901	3,775	3,775	9.71	10.00	0.1872
20	15,360	9,487	9487	10.90	13.20	0.2698	15,360	9,486	9,486	12.04	13.20	0.2698
10	21,654	9,934	9,934	11.10	**17.54**	**0.4023**	21,654	9,944	9,944	12.22	**17.54**	**0.4023**
4	29,659	12,270	12,270	11.97	17.55	1.0000	29,659	12,269	12,269	12.98	17.55	1.0000
2	35,396	12,751	12,751	12.22	0		35,396	12,740	12,740	13.22	0	
1	40,815	15,346	15,346	13.07	0		40,815	15,355	15,355	13.98	0	
0.5	45,896	15,736	15,736	13.41	0		45,896	15,692	15,692	14.35	0	
0.2	52,080	19,117	19,117	15.53	0		52,080	19,118	19,118	16.34	0	

INDEX POINT SL1

Annual Chance Exceedance	Without Project											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
			0	0.00	0	0	Sea level rise does not affect this index point.					
	0	0	0	8.64	*25.00*	*0.0000*						
95			0	0.00	30.20	0.0666						
50	6,901	3,696	3,696	21.72	33.19	0.1739						
20	15,360	9,351	9351	26.90	36.17	0.3073						
10	21,654	9,653	9,653	27.16	**39.16**	**0.4424**						
4	29,659	11,963	11,963	28.64	39.26	1.0000						
2	35,396	12,502	12,502	29.01								
1	40,815	14,917	14,917	30.15								
0.5	45,896	15,285	15,285	30.32								
0.2	52,080	18,529	18,529	31.80								

INDEX POINT SL2

Annual Chance Exceedance	Without Project											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
	0	0	0	0.00	0	0	Sea level rise does not affect this index point.					
	0	0	0	23.00	*34.30*	*0.0000*						
95	0	0	0	0.00	37.20	0.0514						
50	6,901	3,740	3,740	31.39	38.80	0.1009						
20	15,360	9,318	9,318	36.61	40.40	0.1533						
10	21,654	9,652	9,652	36.79	**44.56**	**0.3745**						
4	29,659	11,920	11,920	38.12	44.57	1.0000						
2	35,396	12,713	12,713	38.51	0	0						
1	40,815	14,813	14,813	39.64	0	0						
0.5	45,896	15,204	15,204	39.83	0	0						
0.2	52,080	18,436	18,436	42.23	0	0						

INDEX POINT CR2

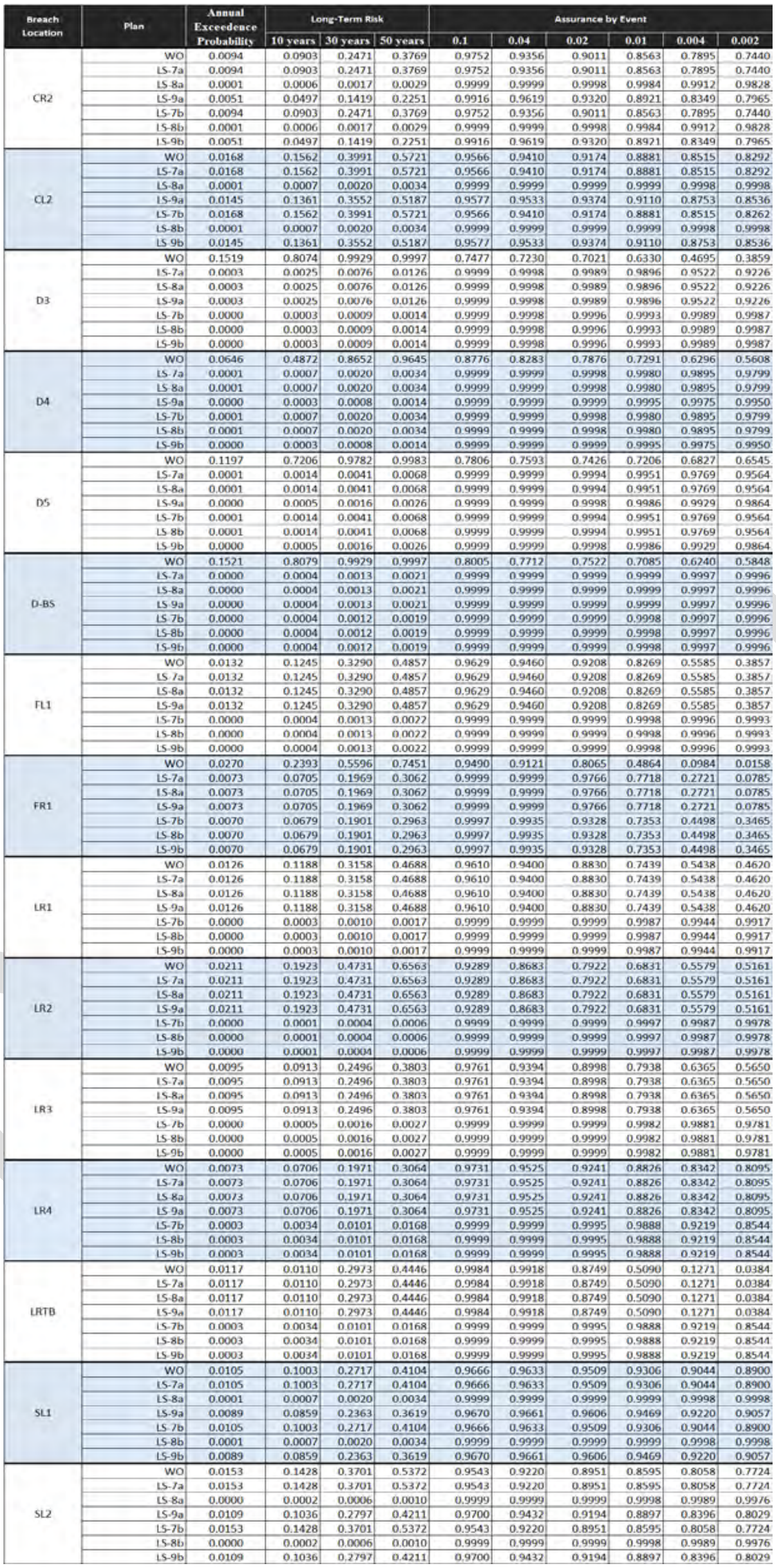
Annual Chance Exceedance	WITHOUT PROJECT											
	2010						2070					
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve	
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure
0	0	0	0	0.00	0	0	Sea level rise does not affect this index point.					
0	0	0	0	6.60	*23.80*	*0.0000*						
95	0	0	0	0.00	25.30	0.0892						
50	6,901	3,848	3,848	19.13	26.90	0.1783						
20	15,360	9,496	9,496	23.35	28.20	0.3036						
10	21,654	9,861	9,861	23.58	**29.66**	**0.4846**						
4	29,659	12,282	12,282	24.81	29.76	1.0000						
2	35,396	12,846	12,846	25.11	0	0						
1	40,815	15,359	15,359	26.29	0	0						
0.5	45,896	15,750	15,750	26.46	0	0						
0.2	52,080	19,126	19,126	27.98	0	0						

INDEX POINT CL2

Annual Chance Exceedance	WITHOUT PROJECT															
	2010						2070									
	Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve		Unregulated-Regulated Transform		Discharge-Stage Rating		Fragility Curve					
	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure	Unregulated Flow	Regulated Flow	Regulated Discharge	Regulated Stage	Stage	*P* of Failure				
0	0	0	0	0.00	0	0	Sea level rise does not affect this index point.									
0	0	0	0	6.60	*21.00*	*0.0000*										
95	0	0	0	0.00	25.50	0.0845										
50	6,901	3,848	3,848	19.13	27.46	0.1719										
20	15,360	9,496	9,496	23.35	29.40	0.2527										
10	21,654	9,861	9,861	23.58	**31.43**	**0.3790**										
4	29,659	12,282	12,282	24.81	31.53	1.0000										
2	35,396	12,846	12,846	25.11												
1	40,815	15,359	15,359	26.29												
0.5	45,896	15,750	15,750	26.46												
0.2	52,080	19,126	19,126	27.98												

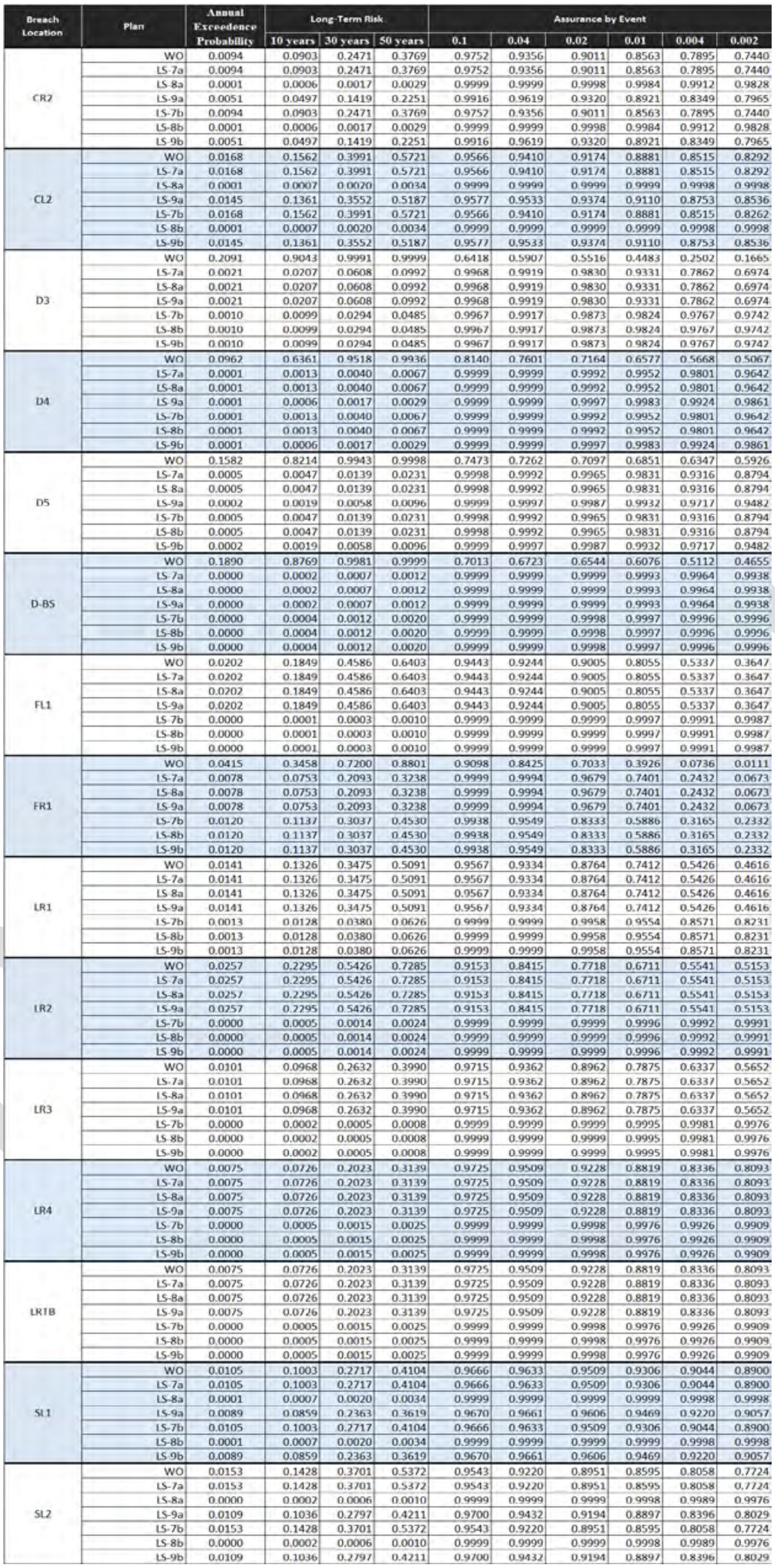
ATTACHMENT 6: PROJECT PERFORMANCE STATISTICS

PROJECT PERFORMANCE—EXISTING CONDITION



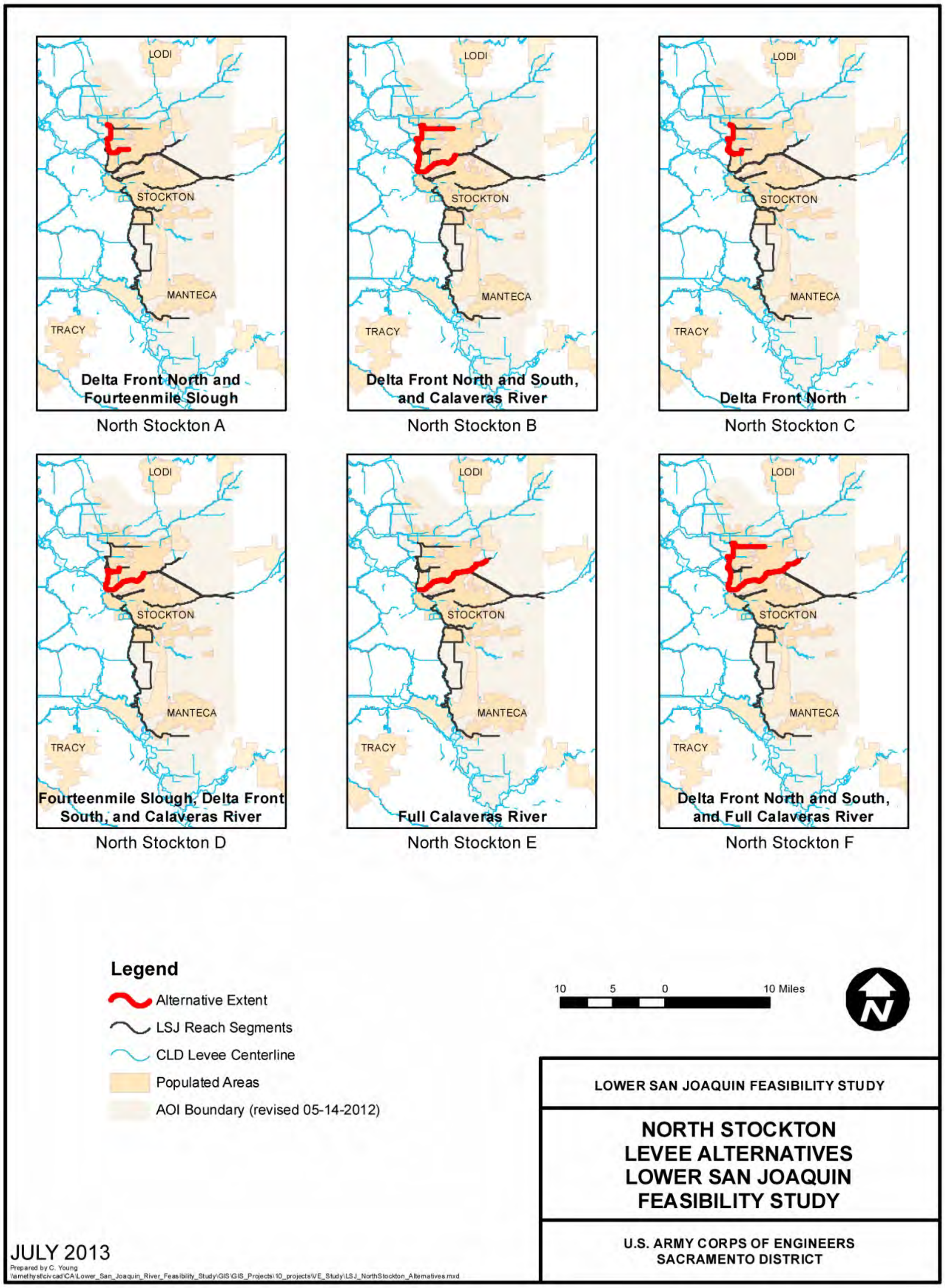
Breach Location	Plan	Annual Exceedence Probability	Long-Term Risk			Assurance by Event					
			10 years	30 years	50 years	0.1	0.04	0.02	0.01	0.004	0.002
CR2	WO	0.0094	0.0903	0.2471	0.3769	0.9752	0.9356	0.9011	0.8563	0.7895	0.7440
	LS-7a	0.0094	0.0903	0.2471	0.3769	0.9752	0.9356	0.9011	0.8563	0.7895	0.7440
	LS-8a	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9998	0.9984	0.9912	0.9828
	LS-9a	0.0051	0.0497	0.1419	0.2251	0.9916	0.9619	0.9320	0.8921	0.8349	0.7965
	LS-7b	0.0094	0.0903	0.2471	0.3769	0.9752	0.9356	0.9011	0.8563	0.7895	0.7440
	LS-8b	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9998	0.9984	0.9912	0.9828
	LS-9b	0.0051	0.0497	0.1419	0.2251	0.9916	0.9619	0.9320	0.8921	0.8349	0.7965
CL2	WO	0.0168	0.1562	0.3991	0.5721	0.9566	0.9410	0.9174	0.8881	0.8515	0.8292
	LS-7a	0.0168	0.1562	0.3991	0.5721	0.9566	0.9410	0.9174	0.8881	0.8515	0.8292
	LS-8a	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
	LS-9a	0.0145	0.1361	0.3552	0.5187	0.9577	0.9533	0.9374	0.9110	0.8753	0.8536
	LS-7b	0.0168	0.1562	0.3991	0.5721	0.9566	0.9410	0.9174	0.8881	0.8515	0.8262
	LS-8b	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
	LS-9b	0.0145	0.1361	0.3552	0.5187	0.9577	0.9533	0.9374	0.9110	0.8753	0.8536
D3	WO	0.1519	0.8074	0.9929	0.9997	0.7477	0.7230	0.7021	0.6330	0.4695	0.3859
	LS-7a	0.0003	0.0025	0.0076	0.0126	0.9999	0.9998	0.9989	0.9896	0.9522	0.9226
	LS-8a	0.0003	0.0025	0.0076	0.0126	0.9999	0.9998	0.9989	0.9896	0.9522	0.9226
	LS-9a	0.0003	0.0025	0.0076	0.0126	0.9999	0.9998	0.9989	0.9896	0.9522	0.9226
	LS-7b	0.0000	0.0003	0.0009	0.0014	0.9999	0.9998	0.9996	0.9993	0.9989	0.9987
	LS-8b	0.0000	0.0003	0.0009	0.0014	0.9999	0.9998	0.9996	0.9993	0.9989	0.9987
	LS-9b	0.0000	0.0003	0.0009	0.0014	0.9999	0.9998	0.9996	0.9993	0.9989	0.9987
D4	WO	0.0646	0.4872	0.8652	0.9645	0.8776	0.8283	0.7876	0.7291	0.6296	0.5608
	LS-7a	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9998	0.9980	0.9895	0.9799
	LS-8a	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9998	0.9980	0.9895	0.9799
	LS-9a	0.0000	0.0003	0.0008	0.0014	0.9999	0.9999	0.9999	0.9995	0.9975	0.9950
	LS-7b	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9998	0.9980	0.9895	0.9799
	LS-8b	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9998	0.9980	0.9895	0.9799
	LS-9b	0.0000	0.0003	0.0008	0.0014	0.9999	0.9999	0.9999	0.9995	0.9975	0.9950
D5	WO	0.1197	0.7206	0.9782	0.9983	0.7806	0.7593	0.7426	0.7206	0.6827	0.6545
	LS-7a	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9994	0.9951	0.9769	0.9564
	LS-8a	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9994	0.9951	0.9769	0.9564
	LS-9a	0.0000	0.0005	0.0016	0.0026	0.9999	0.9999	0.9998	0.9986	0.9929	0.9864
	LS-7b	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9994	0.9951	0.9769	0.9564
	LS-8b	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9994	0.9951	0.9769	0.9564
	LS-9b	0.0000	0.0005	0.0016	0.0026	0.9999	0.9999	0.9998	0.9986	0.9929	0.9864
D-BS	WO	0.1521	0.8079	0.9929	0.9997	0.8005	0.7712	0.7522	0.7085	0.6240	0.5848
	LS-7a	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
	LS-8a	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
	LS-9a	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
	LS-7b	0.0000	0.0004	0.0012	0.0019	0.9999	0.9999	0.9999	0.9998	0.9997	0.9996
	LS-8b	0.0000	0.0004	0.0012	0.0019	0.9999	0.9999	0.9999	0.9998	0.9997	0.9996
	LS-9b	0.0000	0.0004	0.0012	0.0019	0.9999	0.9999	0.9999	0.9998	0.9997	0.9996
FL1	WO	0.0132	0.1245	0.3290	0.4857	0.9629	0.9460	0.9208	0.8269	0.5585	0.3857
	LS-7a	0.0132	0.1245	0.3290	0.4857	0.9629	0.9460	0.9208	0.8269	0.5585	0.3857
	LS-8a	0.0132	0.1245	0.3290	0.4857	0.9629	0.9460	0.9208	0.8269	0.5585	0.3857
	LS-9a	0.0132	0.1245	0.3290	0.4857	0.9629	0.9460	0.9208	0.8269	0.5585	0.3857
	LS-7b	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
	LS-8b	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
	LS-9b	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
FR1	WO	0.0270	0.2393	0.5596	0.7451	0.9490	0.9121	0.8065	0.4864	0.0984	0.0158
	LS-7a	0.0073	0.0705	0.1969	0.3062	0.9999	0.9999	0.9766	0.7718	0.2721	0.0785
	LS-8a	0.0073	0.0705	0.1969	0.3062	0.9999	0.9999	0.9766	0.7718	0.2721	0.0785
	LS-9a	0.0073	0.0705	0.1969	0.3062	0.9999	0.9999	0.9766	0.7718	0.2721	0.0785
	LS-7b	0.0070	0.0679	0.1901	0.2963	0.9997	0.9935	0.9328	0.7353	0.4498	0.3465
	LS-8b	0.0070	0.0679	0.1901	0.2963	0.9997	0.9935	0.9328	0.7353	0.4498	0.3465
	LS-9b	0.0070	0.0679	0.1901	0.2963	0.9997	0.9935	0.9328	0.7353	0.4498	0.3465
LR1	WO	0.0126	0.1188	0.3158	0.4688	0.9610	0.9400	0.8830	0.7439	0.5438	0.4620
	LS-7a	0.0126	0.1188	0.3158	0.4688	0.9610	0.9400	0.8830	0.7439	0.5438	0.4620
	LS-8a	0.0126	0.1188	0.3158	0.4688	0.9610	0.9400	0.8830	0.7439	0.5438	0.4620
	LS-9a	0.0126	0.1188	0.3158	0.4688	0.9610	0.9400	0.8830	0.7439	0.5438	0.4620
	LS-7b	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9987	0.9944	0.9917
	LS-8b	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9987	0.9944	0.9917
	LS-9b	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9987	0.9944	0.9917
LR2	WO	0.0211	0.1923	0.4731	0.6563	0.9289	0.8683	0.7922	0.6831	0.5579	0.5161
	LS-7a	0.0211	0.1923	0.4731	0.6563	0.9289	0.8683	0.7922	0.6831	0.5579	0.5161
	LS-8a	0.0211	0.1923	0.4731	0.6563	0.9289	0.8683	0.7922	0.6831	0.5579	0.5161
	LS-9a	0.0211	0.1923	0.4731	0.6563	0.9289	0.8683	0.7922	0.6831	0.5579	0.5161
	LS-7b	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9997	0.9987	0.9978
	LS-8b	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9997	0.9987	0.9978
	LS-9b	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9997	0.9987	0.9978
LR3	WO	0.0095	0.0913	0.2496	0.3803	0.9761	0.9394	0.8998	0.7938	0.6365	0.5650
	LS-7a	0.0095	0.0913	0.2496	0.3803	0.9761	0.9394	0.8998	0.7938	0.6365	0.5650
	LS-8a	0.0095	0.0913	0.2496	0.3803	0.9761	0.9394	0.8998	0.7938	0.6365	0.5650
	LS-9a	0.0095	0.0913	0.2496	0.3803	0.9761	0.9394	0.8998	0.7938	0.6365	0.5650
	LS-7b	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9982	0.9881	0.9781
	LS-8b	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9982	0.9881	0.9781
	LS-9b	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9982	0.9881	0.9781
LR4	WO	0.0073	0.0706	0.1971	0.3064	0.9731	0.9525	0.9241	0.8826	0.8342	0.8095
	LS-7a	0.0073	0.0706	0.1971	0.3064	0.9731	0.9525	0.9241	0.8826	0.8342	0.8095
	LS-8a	0.0073	0.0706	0.1971	0.3064	0.9731	0.9525	0.9241	0.8826	0.8342	0.8095
	LS-9a	0.0073	0.0706	0.1971	0.3064	0.9731	0.9525	0.9241	0.8826	0.8342	0.8095
	LS-7b	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9995	0.9888	0.9219	0.8544
	LS-8b	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9995	0.9888	0.9219	0.8544
	LS-9b	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9995	0.9888	0.9219	0.8544
LRTB	WO	0.0117	0.0110	0.2973	0.4446	0.9984	0.9918	0.8749	0.5090	0.1271	0.0384
	LS-7a	0.0117	0.0110	0.2973	0.4446	0.9984	0.9918	0.8749	0.5090	0.1271	0.0384
	LS-8a	0.0117	0.0110	0.2973	0.4446	0.9984	0.9918	0.8749	0.5090	0.1271	0.0384
	LS-9a	0.0117	0.0110	0.2973	0.4446	0.9984	0.9918	0.8749	0.5090	0.1271	0.0384
	LS-7b	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9995	0.9888	0.9219	0.8544
	LS-8b	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9995	0.9888	0.9219	0.8544
	LS-9b	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9995	0.9888	0.9219	0.8544
SL1	WO	0.0105	0.1003	0.2717	0.4104	0.9666	0.9633	0.9509	0.9306	0.9044	0.8900
	LS-7a	0.0105	0.1003	0.2717	0.4104	0.9666	0.9633	0.9509	0.9306	0.9044	0.8900
	LS-8a	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
	LS-9a	0.0089	0.0859	0.2363	0.3619	0.9670	0.9661	0.9606	0.9469	0.9220	0.9057
	LS-7b	0.0105	0.1003	0.2717	0.4104	0.9666	0.9633	0.9509	0.9306	0.9044	0.8900
	LS-8b	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
	LS-9b	0.0089	0.0859	0.2363	0.3619	0.9670	0.9661	0.9606	0.9469	0.9220	0.9057
SL2	WO	0.0153	0.1428	0.3701	0.5372	0.9543	0.9220	0.8951	0.8595	0.8058	0.7724
	LS-7a	0.0153	0.1428	0.3701	0.5372	0.9543	0.9220	0.8951	0.8595	0.8058	0.7724
	LS-8a	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9998	0.9989	0.9976
	LS-9a	0.0109	0.1036	0.2797	0.4211	0.9700	0.9432	0.9194	0.8897	0.8396	0.8029
	LS-7b	0.0153	0.1428	0.3701	0.5372	0.9543	0.9220	0.8951	0.8595	0.8058	0.7724
	LS-8b	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9998	0.9989	0.9976
	LS-9b	0.0109	0.1036	0.2797	0.4211	0.9700	0.9432	0.9194	0.8897	0.8396	0.8029

Project Performance—Future Condition



Breach Location	Plan	Annual Exceedence Probability	Long-Term Risk			Assurance by Event					
			10 years	30 years	50 years	0.1	0.04	0.02	0.01	0.004	0.002
CR2	WO	0.0094	0.0903	0.2471	0.3769	0.9752	0.9356	0.9011	0.8563	0.7895	0.7440
	LS-7a	0.0094	0.0903	0.2471	0.3769	0.9752	0.9356	0.9011	0.8563	0.7895	0.7440
	LS-8a	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9998	0.9984	0.9912	0.9828
	LS-9a	0.0051	0.0497	0.1419	0.2251	0.9916	0.9619	0.9320	0.8921	0.8349	0.7965
	LS-7b	0.0094	0.0903	0.2471	0.3769	0.9752	0.9356	0.9011	0.8563	0.7895	0.7440
	LS-8b	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9998	0.9984	0.9912	0.9828
	LS-9b	0.0051	0.0497	0.1419	0.2251	0.9916	0.9619	0.9320	0.8921	0.8349	0.7965
CL2	WO	0.0168	0.1562	0.3991	0.5721	0.9566	0.9410	0.9174	0.8881	0.8515	0.8292
	LS-7a	0.0168	0.1562	0.3991	0.5721	0.9566	0.9410	0.9174	0.8881	0.8515	0.8292
	LS-8a	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
	LS-9a	0.0145	0.1361	0.3552	0.5187	0.9577	0.9533	0.9374	0.9110	0.8753	0.8536
	LS-7b	0.0168	0.1562	0.3991	0.5721	0.9566	0.9410	0.9174	0.8881	0.8515	0.8262
	LS-8b	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
	LS-9b	0.0145	0.1361	0.3552	0.5187	0.9577	0.9533	0.9374	0.9110	0.8753	0.8536
D3	WO	0.2091	0.9043	0.9991	0.9999	0.6418	0.5907	0.5516	0.4483	0.2502	0.1665
	LS-7a	0.0021	0.0207	0.0608	0.0992	0.9968	0.9919	0.9830	0.9331	0.7862	0.6974
	LS-8a	0.0021	0.0207	0.0608	0.0992	0.9968	0.9919	0.9830	0.9331	0.7862	0.6974
	LS-9a	0.0021	0.0207	0.0608	0.0992	0.9968	0.9919	0.9830	0.9331	0.7862	0.6974
	LS-7b	0.0010	0.0099	0.0294	0.0485	0.9967	0.9917	0.9873	0.9824	0.9767	0.9742
	LS-8b	0.0010	0.0099	0.0294	0.0485	0.9967	0.9917	0.9873	0.9824	0.9767	0.9742
	LS-9b	0.0010	0.0099	0.0294	0.0485	0.9967	0.9917	0.9873	0.9824	0.9767	0.9742
D4	WO	0.0962	0.6361	0.9518	0.9936	0.8140	0.7601	0.7164	0.6577	0.5668	0.5067
	LS-7a	0.0001	0.0013	0.0040	0.0067	0.9999	0.9999	0.9992	0.9952	0.9801	0.9642
	LS-8a	0.0001	0.0013	0.0040	0.0067	0.9999	0.9999	0.9992	0.9952	0.9801	0.9642
	LS-9a	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9997	0.9983	0.9924	0.9861
	LS-7b	0.0001	0.0013	0.0040	0.0067	0.9999	0.9999	0.9992	0.9952	0.9801	0.9642
	LS-8b	0.0001	0.0013	0.0040	0.0067	0.9999	0.9999	0.9992	0.9952	0.9801	0.9642
	LS-9b	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9997	0.9983	0.9924	0.9861
D5	WO	0.1582	0.8214	0.9943	0.9998	0.7473	0.7262	0.7097	0.6851	0.6347	0.5926
	LS-7a	0.0005	0.0047	0.0139	0.0231	0.9998	0.9992	0.9965	0.9831	0.9316	0.8794
	LS-8a	0.0005	0.0047	0.0139	0.0231	0.9998	0.9992	0.9965	0.9831	0.9316	0.8794
	LS-9a	0.0002	0.0019	0.0058	0.0096	0.9999	0.9997	0.9987	0.9932	0.9717	0.9482
	LS-7b	0.0005	0.0047	0.0139	0.0231	0.9998	0.9992	0.9965	0.9831	0.9316	0.8794
	LS-8b	0.0005	0.0047	0.0139	0.0231	0.9998	0.9992	0.9965	0.9831	0.9316	0.8794
	LS-9b	0.0002	0.0019	0.0058	0.0096	0.9999	0.9997	0.9987	0.9932	0.9717	0.9482
D-BS	WO	0.1890	0.8769	0.9981	0.9999	0.7013	0.6723	0.6544	0.6076	0.5112	0.4655
	LS-7a	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9993	0.9964	0.9938
	LS-8a	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9993	0.9964	0.9938
	LS-9a	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9993	0.9964	0.9938
	LS-7b	0.0000	0.0004	0.0012	0.0020	0.9999	0.9999	0.9998	0.9997	0.9996	0.9996
	LS-8b	0.0000	0.0004	0.0012	0.0020	0.9999	0.9999	0.9998	0.9997	0.9996	0.9996
	LS-9b	0.0000	0.0004	0.0012	0.0020	0.9999	0.9999	0.9998	0.9997	0.9996	0.9996
FL1	WO	0.0202	0.1849	0.4586	0.6403	0.9443	0.9244	0.9005	0.8055	0.5337	0.3647
	LS-7a	0.0202	0.1849	0.4586	0.6403	0.9443	0.9244	0.9005	0.8055	0.5337	0.3647
	LS-8a	0.0202	0.1849	0.4586	0.6403	0.9443	0.9244	0.9005	0.8055	0.5337	0.3647
	LS-9a	0.0202	0.1849	0.4586	0.6403	0.9443	0.9244	0.9005	0.8055	0.5337	0.3647
	LS-7b	0.0000	0.0001	0.0003	0.0010	0.9999	0.9999	0.9999	0.9997	0.9991	0.9987
	LS-8b	0.0000	0.0001	0.0003	0.0010	0.9999	0.9999	0.9999	0.9997	0.9991	0.9987
	LS-9b	0.0000	0.0001	0.0003	0.0010	0.9999	0.9999	0.9999	0.9997	0.9991	0.9987
FR1	WO	0.0415	0.3458	0.7200	0.8801	0.9098	0.8425	0.7033	0.3926	0.0736	0.0111
	LS-7a	0.0078	0.0753	0.2093	0.3238	0.9999	0.9994	0.9679	0.7401	0.2432	0.0673
	LS-8a	0.0078	0.0753	0.2093	0.3238	0.9999	0.9994	0.9679	0.7401	0.2432	0.0673
	LS-9a	0.0078	0.0753	0.2093	0.3238	0.9999	0.9994	0.9679	0.7401	0.2432	0.0673
	LS-7b	0.0120	0.1137	0.3037	0.4530	0.9938	0.9549	0.8333	0.5886	0.3165	0.2332
	LS-8b	0.0120	0.1137	0.3037	0.4530	0.9938	0.9549	0.8333	0.5886	0.3165	0.2332
	LS-9b	0.0120	0.1137	0.3037	0.4530	0.9938	0.9549	0.8333	0.5886	0.3165	0.2332
LR1	WO	0.0141	0.1326	0.3475	0.5091	0.9567	0.9334	0.8764	0.7412	0.5426	0.4616
	LS-7a	0.0141	0.1326	0.3475	0.5091	0.9567	0.9334	0.8764	0.7412	0.5426	0.4616
	LS-8a	0.0141	0.1326	0.3475	0.5091	0.9567	0.9334	0.8764	0.7412	0.5426	0.4616
	LS-9a	0.0141	0.1326	0.3475	0.5091	0.9567	0.9334	0.8764	0.7412	0.5426	0.4616
	LS-7b	0.0013	0.0128	0.0380	0.0626	0.9999	0.9999	0.9958	0.9554	0.8571	0.8231
	LS-8b	0.0013	0.0128	0.0380	0.0626	0.9999	0.9999	0.9958	0.9554	0.8571	0.8231
	LS-9b	0.0013	0.0128	0.0380	0.0626	0.9999	0.9999	0.9958	0.9554	0.8571	0.8231
LR2	WO	0.0257	0.2295	0.5426	0.7285	0.9153	0.8415	0.7718	0.6711	0.5541	0.5153
	LS-7a	0.0257	0.2295	0.5426	0.7285	0.9153	0.8415	0.7718	0.6711	0.5541	0.5153
	LS-8a	0.0257	0.2295	0.5426	0.7285	0.9153	0.8415	0.7718	0.6711	0.5541	0.5153
	LS-9a	0.0257	0.2295	0.5426	0.7285	0.9153	0.8415	0.7718	0.6711	0.5541	0.5153
	LS-7b	0.0000	0.0005	0.0014	0.0024	0.9999	0.9999	0.9999	0.9996	0.9992	0.9991
	LS-8b	0.0000	0.0005	0.0014	0.0024	0.9999	0.9999	0.9999	0.9996	0.9992	0.9991
	LS-9b	0.0000	0.0005	0.0014	0.0024	0.9999	0.9999	0.9999	0.9996	0.9992	0.9991
LR3	WO	0.0101	0.0968	0.2632	0.3990	0.9715	0.9362	0.8962	0.7875	0.6337	0.5652
	LS-7a	0.0101	0.0968	0.2632	0.3990	0.9715	0.9362	0.8962	0.7875	0.6337	0.5652
	LS-8a	0.0101	0.0968	0.2632	0.3990	0.9715	0.9362	0.8962	0.7875	0.6337	0.5652
	LS-9a	0.0101	0.0968	0.2632	0.3990	0.9715	0.9362	0.8962	0.7875	0.6337	0.5652
	LS-7b	0.0000	0.0002	0.0005	0.0008	0.9999	0.9999	0.9999	0.9995	0.9981	0.9976
	LS-8b	0.0000	0.0002	0.0005	0.0008	0.9999	0.9999	0.9999	0.9995	0.9981	0.9976
	LS-9b	0.0000	0.0002	0.0005	0.0008	0.9999	0.9999	0.9999	0.9995	0.9981	0.9976
LR4	WO	0.0075	0.0726	0.2023	0.3139	0.9725	0.9509	0.9228	0.8819	0.8336	0.8093
	LS-7a	0.0075	0.0726	0.2023	0.3139	0.9725	0.9509	0.9228	0.8819	0.8336	0.8093
	LS-8a	0.0075	0.0726	0.2023	0.3139	0.9725	0.9509	0.9228	0.8819	0.8336	0.8093
	LS-9a	0.0075	0.0726	0.2023	0.3139	0.9725	0.9509	0.9228	0.8819	0.8336	0.8093
	LS-7b	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9998	0.9976	0.9926	0.9909
	LS-8b	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9998	0.9976	0.9926	0.9909
	LS-9b	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9998	0.9976	0.9926	0.9909
LRTB	WO	0.0075	0.0726	0.2023	0.3139	0.9725	0.9509	0.9228	0.8819	0.8336	0.8093
	LS-7a	0.0075	0.0726	0.2023	0.3139	0.9725	0.9509	0.9228	0.8819	0.8336	0.8093
	LS-8a	0.0075	0.0726	0.2023	0.3139	0.9725	0.9509	0.9228	0.8819	0.8336	0.8093
	LS-9a	0.0075	0.0726	0.2023	0.3139	0.9725	0.9509	0.9228	0.8819	0.8336	0.8093
	LS-7b	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9998	0.9976	0.9926	0.9909
	LS-8b	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9998	0.9976	0.9926	0.9909
	LS-9b	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9998	0.9976	0.9926	0.9909
SL1	WO	0.0105	0.1003	0.2717	0.4104	0.9666	0.9633	0.9509	0.9306	0.9044	0.8900
	LS-7a	0.0105	0.1003	0.2717	0.4104	0.9666	0.9633	0.9509	0.9306	0.9044	0.8900
	LS-8a	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
	LS-9a	0.0089	0.0859	0.2363	0.3619	0.9670	0.9661	0.9606	0.9469	0.9220	0.9057
	LS-7b	0.0105	0.1003	0.2717	0.4104	0.9666	0.9633	0.9509	0.9306	0.9044	0.8900
	LS-8b	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
	LS-9b	0.0089	0.0859	0.2363	0.3619	0.9670	0.9661	0.9606	0.9469	0.9220	0.9057
SL2	WO	0.0153	0.1428	0.3701	0.5372	0.9543	0.9220	0.8951	0.8595	0.8058	0.7724
	LS-7a	0.0153	0.1428	0.3701	0.5372	0.9543	0.9220	0.8951	0.8595	0.8058	0.7724
	LS-8a	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9998	0.9989	0.9976
	LS-9a	0.0109	0.1036	0.2797	0.4211	0.9700	0.9432	0.9194	0.8897	0.8396	0.8029
	LS-7b	0.0153	0.1428	0.3701	0.5372	0.9543	0.9220	0.8951	0.8595	0.8058	0.7724
	LS-8b	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9998	0.9989	0.9976
	LS-9b	0.0109	0.1036	0.2797	0.4211	0.9700	0.9432	0.9194	0.8897	0.8396	0.8029

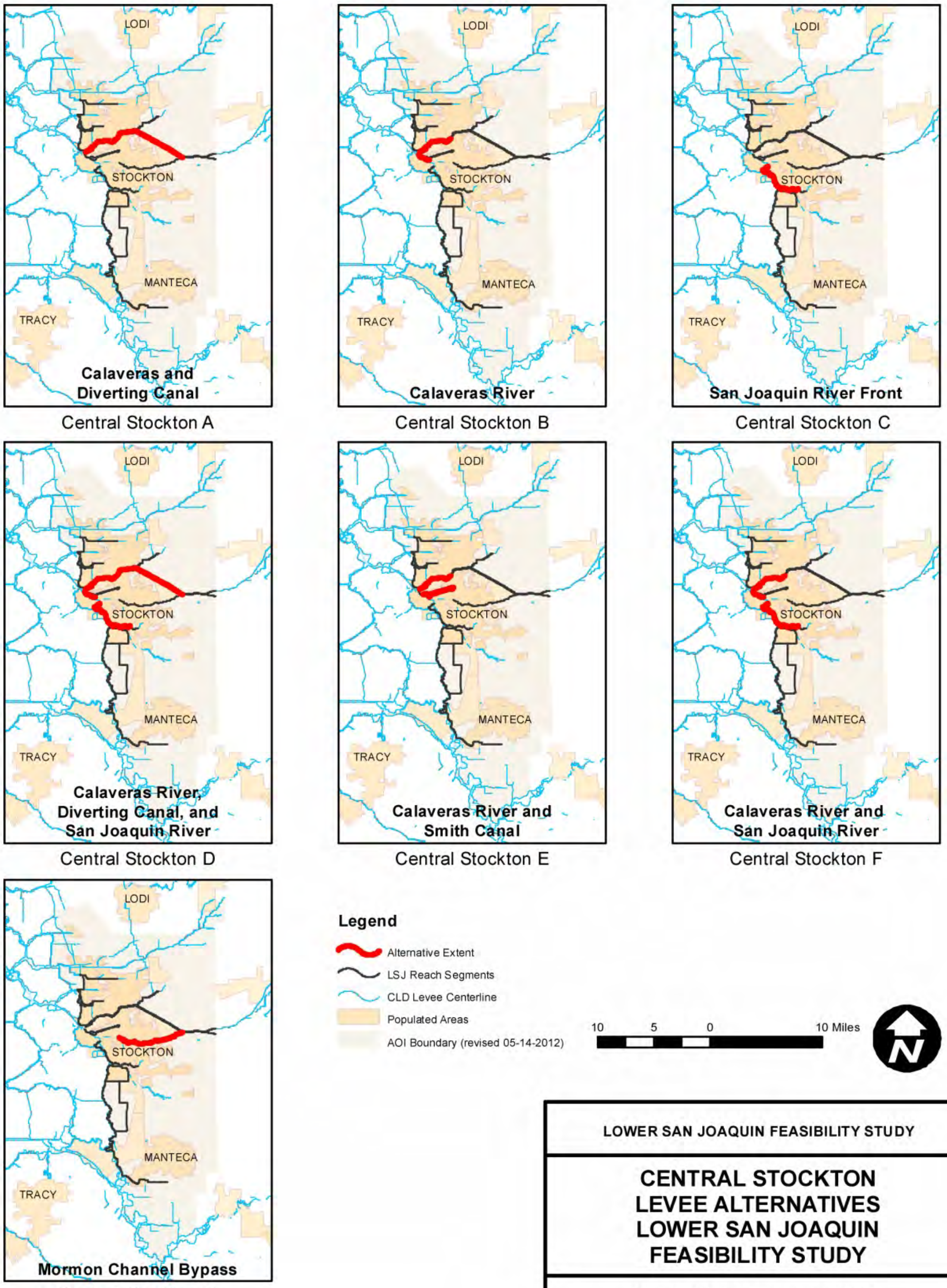
ATTACHMENT 7: INITIAL ARRAY OF ALTERNATIVES MAPS



LOWER SAN JOAQUIN FEASIBILITY STUDY

NORTH STOCKTON
LEVEE ALTERNATIVES
LOWER SAN JOAQUIN
FEASIBILITY STUDY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



Central Stockton A

Central Stockton B

Central Stockton C

Central Stockton D

Central Stockton E

Central Stockton F

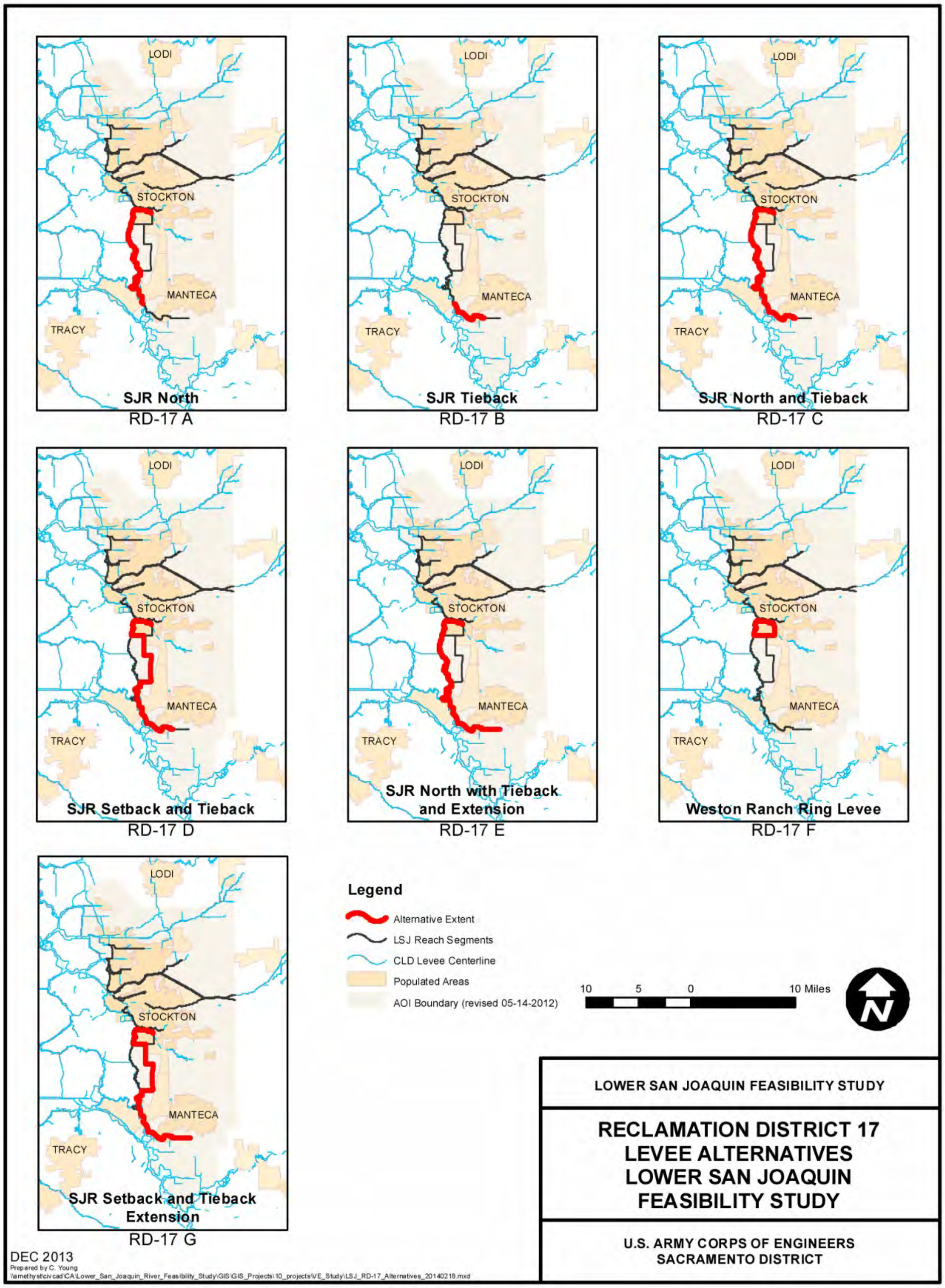
Central Stockton G

LOWER SAN JOAQUIN FEASIBILITY STUDY

**CENTRAL STOCKTON
LEVEE ALTERNATIVES
LOWER SAN JOAQUIN
FEASIBILITY STUDY**

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JULY 2013
Prepared by C. Young
\\amethyst\civcad\CA\Lower_San_Joaquin_River_Feasibility_Study\GIS\GIS_Projects\10_projects\VE_Study\LSJ_CentralStockton_Alternatives.mxd



LOWER SAN JOAQUIN FEASIBILITY STUDY

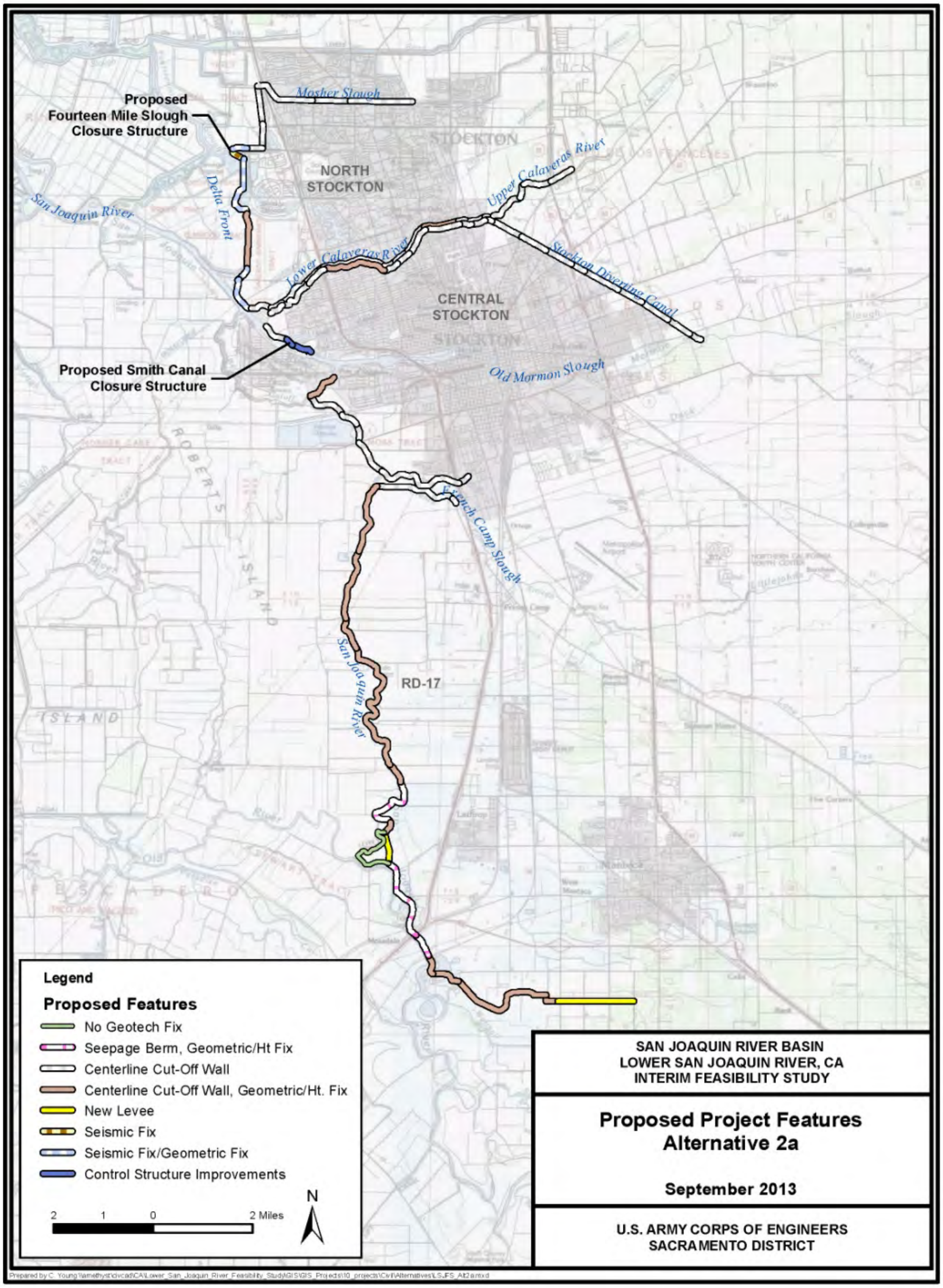
RECLAMATION DISTRICT 17
LEVEE ALTERNATIVES
LOWER SAN JOAQUIN
FEASIBILITY STUDY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

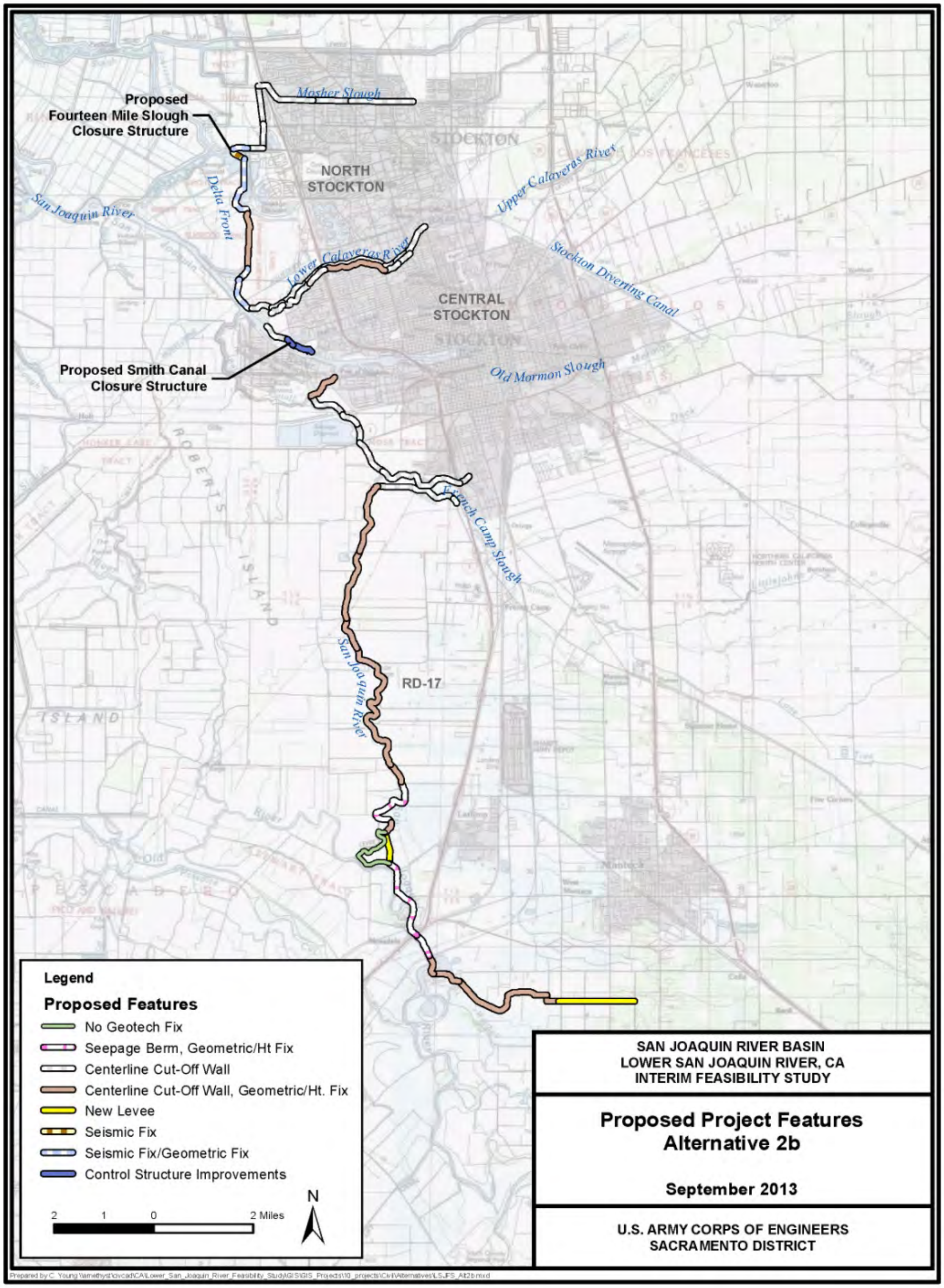
DEC 2013
Prepared by C. Young
\\amethyst\civcad\CA\Lower_San_Joaquin_River_Feasibility_Study\GIS\GIS_Projects\10_projects\VE_Study\LSJ_RD-17_Alternatives_20140218.mxd

ATTACHMENT 8: FOCUSED ARRAY OF ALTERNATIVES MAPS

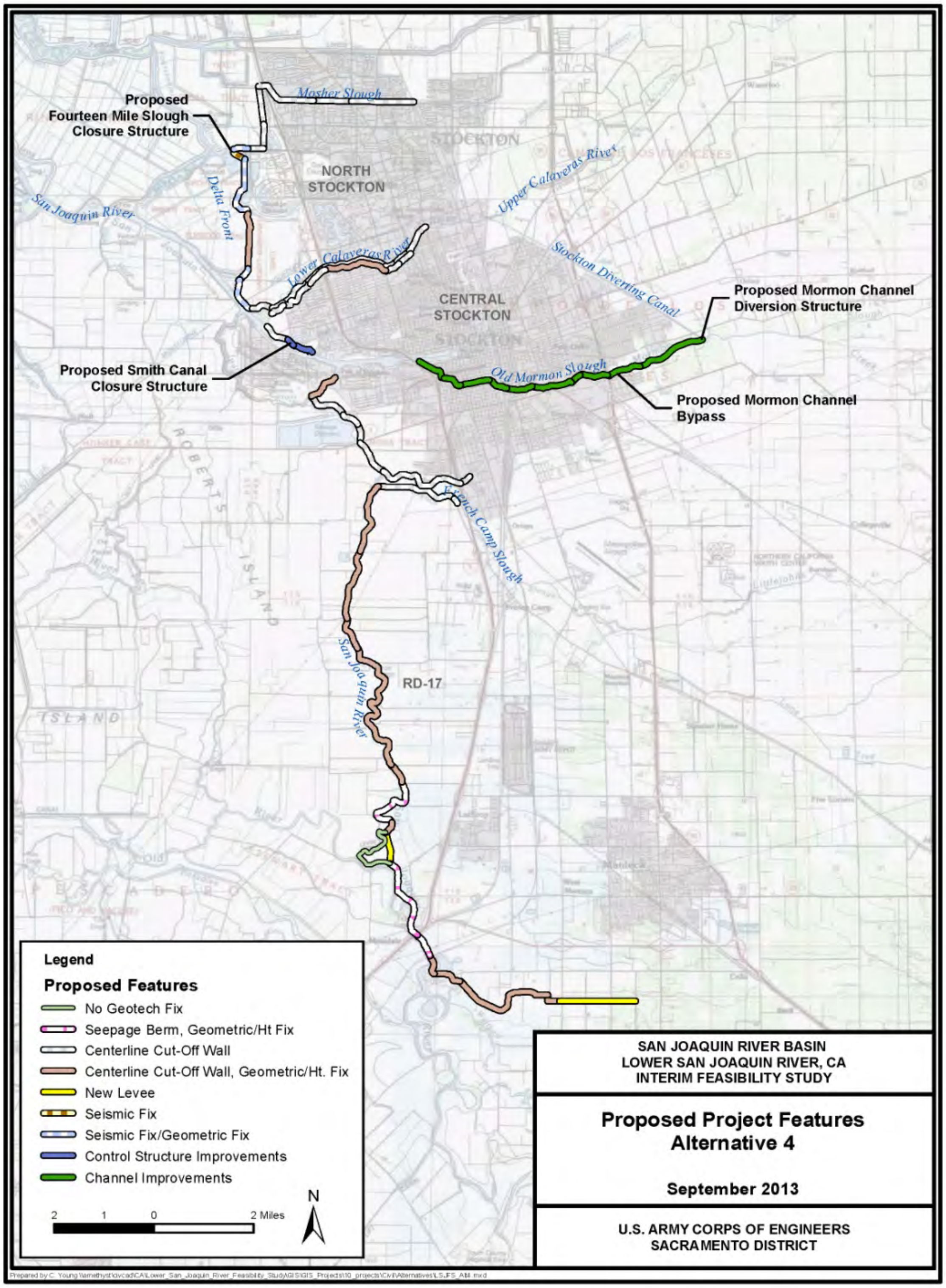
ALTERNATIVE 2A



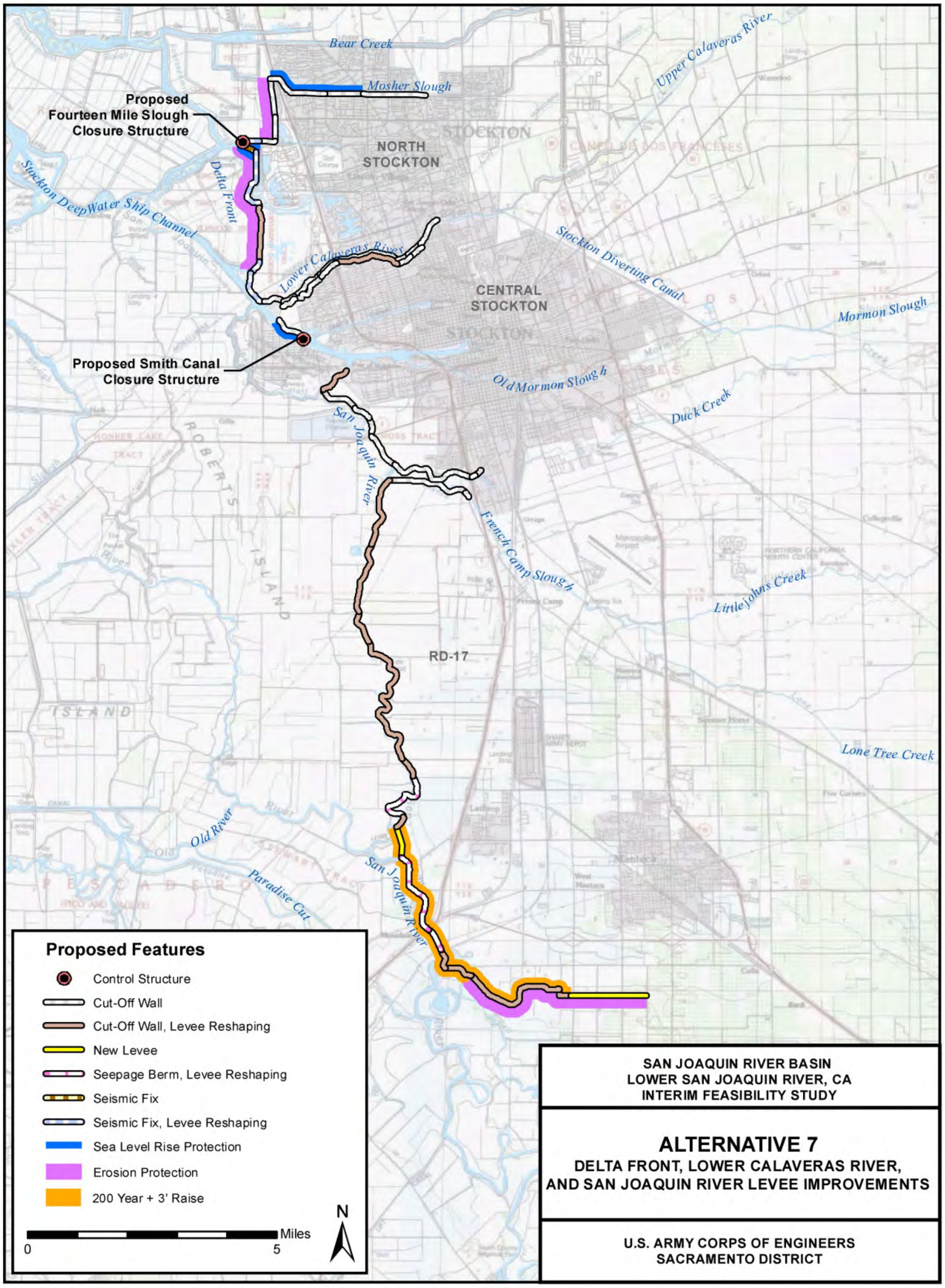
ALTERNATIVE 2B



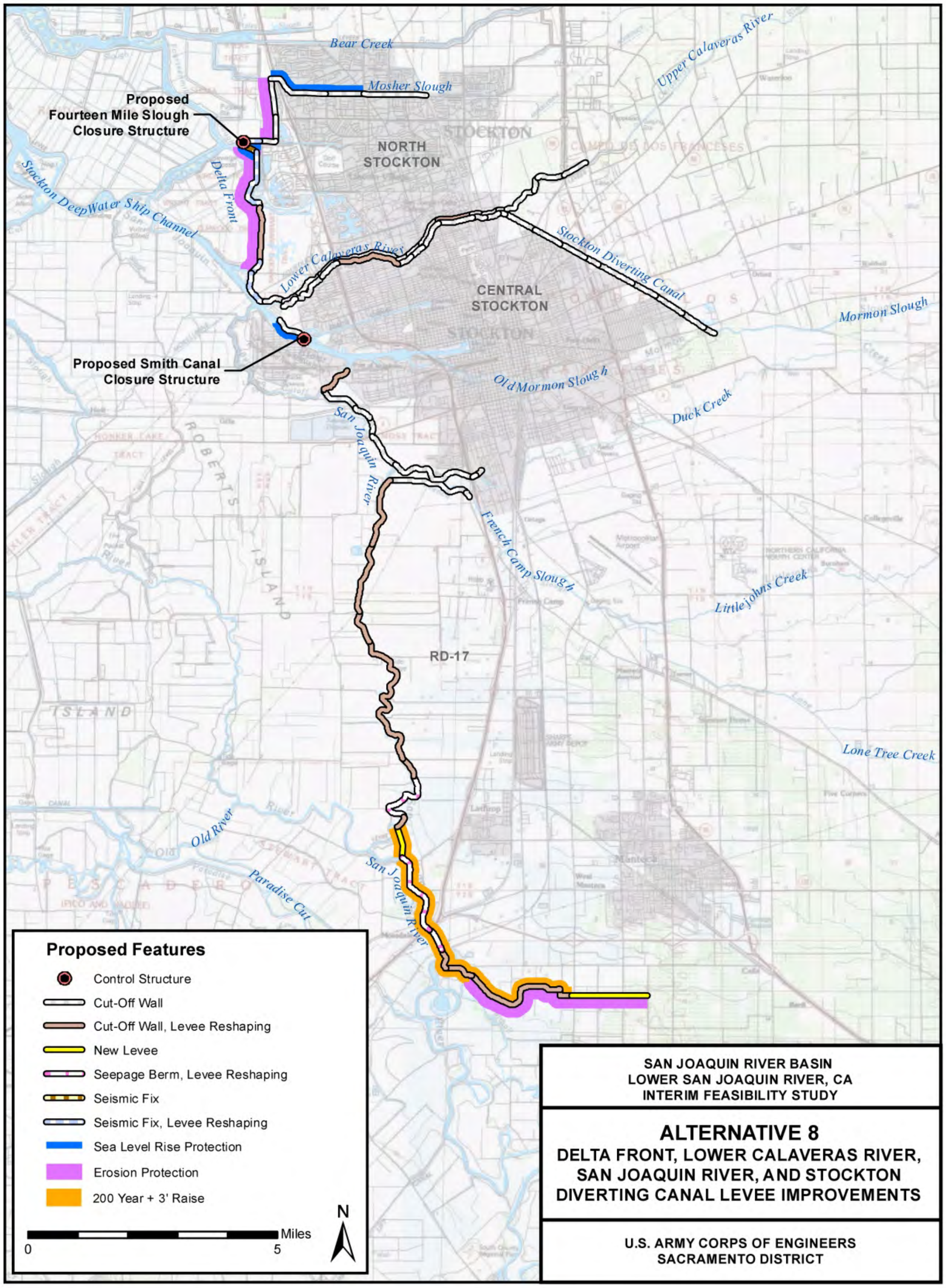
ALTERNATIVE 4



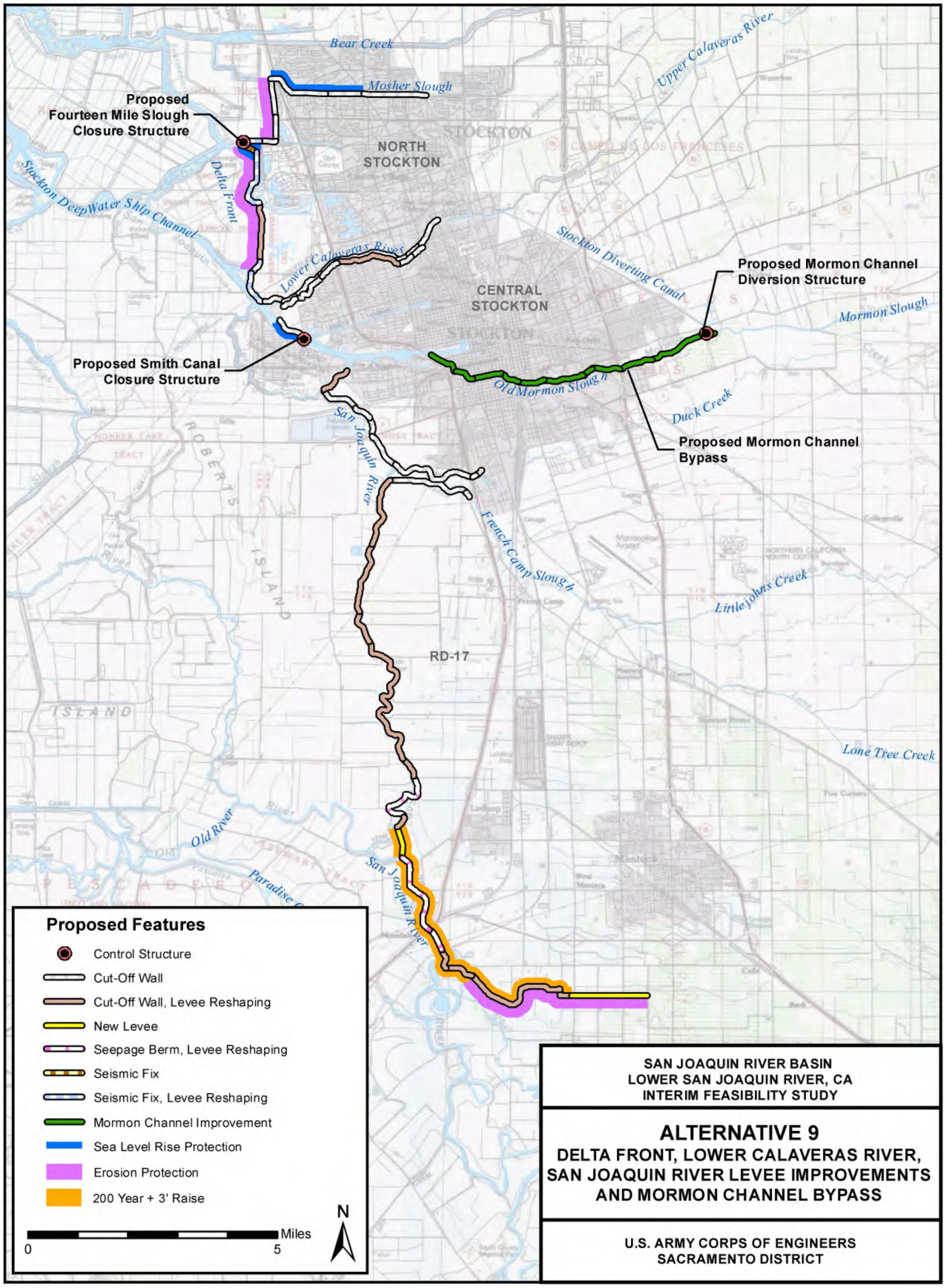
ALTERNATIVE 7



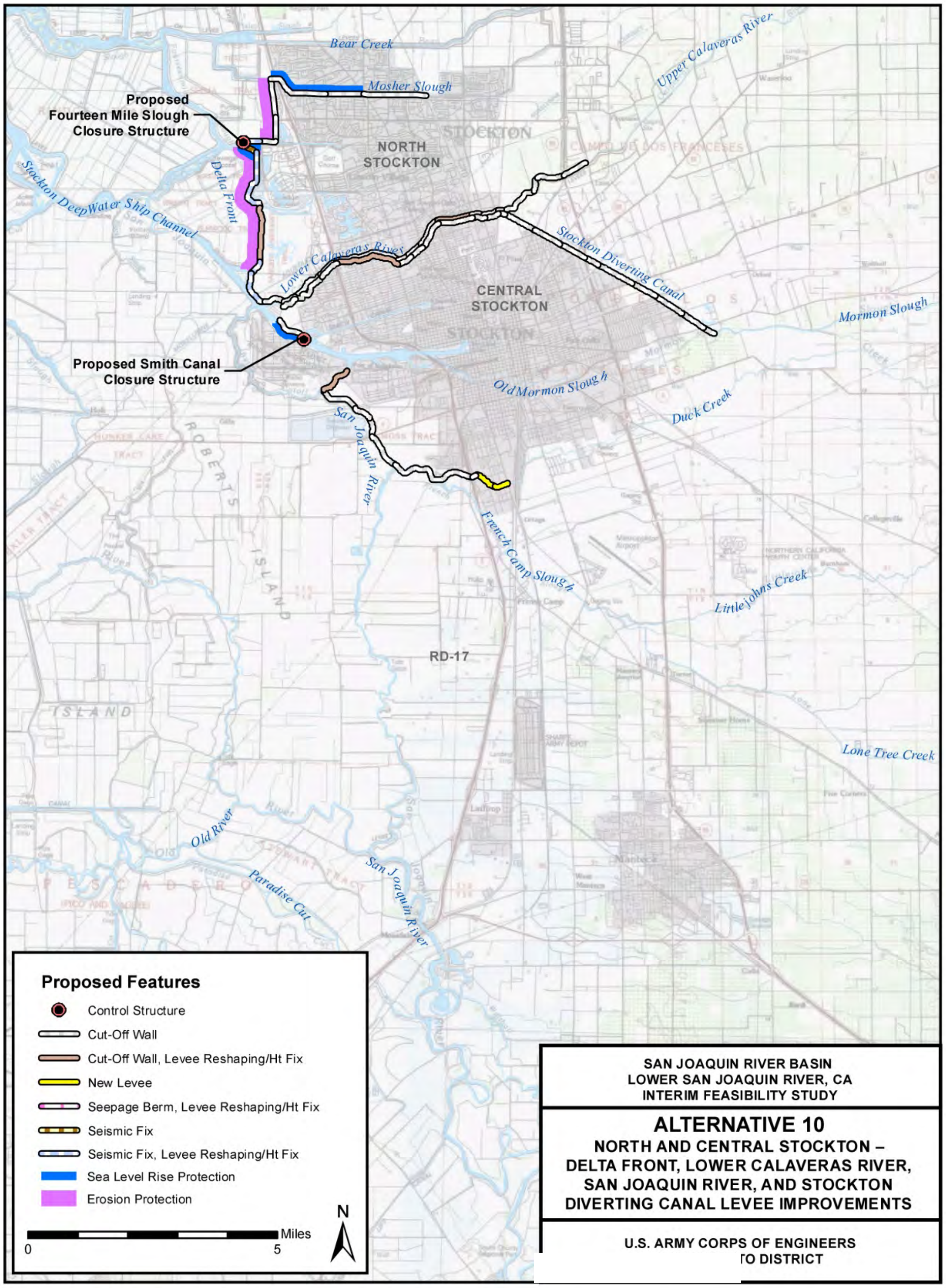
ALTERNATIVE 8



ALTERNATIVE 9



ALTERNATIVE 10



ATTACHMENT 9: IDC AND BDC CALCULATIONS

ALTERNATIVE LS-7A

INTEREST RATE	**3.500%**
PERIOD	**50**
CONSTRUCTION YEAR	**2017**
BASE YEAR	**2028**
CRF	0.042634

ALTERNATIVE	LS-7a
TOTAL IDC	$222,331,136
IDC ANNUAL EQUIVALENT	$9,478,801
TOTAL BDC	$812,553,213
BDC ANNUAL EQUIVALENT	$34,642,158

FIX	START YEAR	END YEAR	TOTAL FIRST COST
North Stockton	2018	2028	$627,088,544
Central Stockton	2017	2020	$214,053,622
RD 17	2018	2028	$0

PERIOD	YEAR	PRESENT WORTH FACTOR	COSTS PRIOR TO BASE	BENEFITS PRIOR TO BASE	COSTS PERIOD OF ANALYSIS	BENEFITS PERIOD OF ANALYSIS	TOTAL COSTS	TOTAL BENEFITS	PRESENT VALUE OF COSTS	PRESENT VALUE OF BENEFITS
-11	2017	1.459970	$71,351,207	$0	$0	$0	$71,351,207	$0	$104,170,602	$0
-10	2018	1.410599	$134,060,062	$0	$0	$0	$134,060,062	$0	$189,104,957	$0
-9	2019	1.362897	$134,060,062	$0	$0	$0	$134,060,062	$0	$182,710,103	$0
-8	2020	1.316809	$62,708,854	$86,732,516	$0	$0	$62,708,854	$86,732,516	$82,575,586	$114,210,161
-7	2021	1.272279	$62,708,854	$86,732,516	$0	$0	$62,708,854	$86,732,516	$79,783,175	$110,347,982
-6	2022	1.229255	$62,708,854	$86,732,516	$0	$0	$62,708,854	$86,732,516	$77,085,193	$106,616,407
-5	2023	1.187686	$62,708,854	$86,732,516	$0	$0	$62,708,854	$86,732,516	$74,478,448	$103,011,022
-4	2024	1.147523	$62,708,854	$86,732,516	$0	$0	$62,708,854	$86,732,516	$71,959,853	$99,527,557
-3	2025	1.108718	$62,708,854	$86,732,516	$0	$0	$62,708,854	$86,732,516	$69,526,428	$96,161,891
-2	2026	1.071225	$62,708,854	$86,732,516	$0	$0	$62,708,854	$86,732,516	$67,175,293	$92,910,039
-1	2027	1.035000	$62,708,854	$86,732,516	$0	$0	$62,708,854	$86,732,516	$64,903,664	$89,768,154
0	2028	1.000000	$0	$0	$0	$0	$0	$0	$0	$0

ALTERNATIVE LS-8

INTEREST RATE	3.500%
PERIOD	50
CONSTRUCTION YEAR	2017
BASE YEAR	2028
CRF	0.042634

ALTERNATIVE	LS-8a
TOTAL IDC	$263,231,321
IDC ANNUAL EQUIVALENT	$11,222,528
TOTAL BDC	$827,137,984
BDC ANNUAL EQUIVALENT	$35,263,961

FIX	START YEAR	END YEAR	TOTAL FIRST COST
North Stockton	2018	2028	$669,445,471
Central Stockton	2017	2020	$291,463,223
RD 17	2018	2028	$0

PERIOD	YEAR	PRESENT WORTH FACTOR	COSTS PRIOR TO BASE	BENEFITS PRIOR TO BASE	COSTS PERIOD OF ANALYSIS	BENEFITS PERIOD OF ANALYSIS	TOTAL COSTS	TOTAL BENEFITS	PRESENT VALUE OF COSTS	PRESENT VALUE OF BENEFITS
-11	2017	1.459970	$97,154,408	$0	$0	$0	$97,154,408	$0	$141,842,493	$0
-10	2018	1.410599	$164,098,955	$0	$0	$0	$164,098,955	$0	$231,477,782	$0
-9	2019	1.362897	$164,098,955	$0	$0	$0	$164,098,955	$0	$223,650,031	$0
-8	2020	1.316809	$66,944,547	$88,289,305	$0	$0	$66,944,547	$88,289,305	$88,153,185	$116,260,155
-7	2021	1.272279	$66,944,547	$88,289,305	$0	$0	$66,944,547	$88,289,305	$85,172,159	$112,328,652
-6	2022	1.229255	$66,944,547	$88,289,305	$0	$0	$66,944,547	$88,289,305	$82,291,941	$108,530,098
-5	2023	1.187686	$66,944,547	$88,289,305	$0	$0	$66,944,547	$88,289,305	$79,509,122	$104,859,998
-4	2024	1.147523	$66,944,547	$88,289,305	$0	$0	$66,944,547	$88,289,305	$76,820,408	$101,314,008
-3	2025	1.108718	$66,944,547	$88,289,305	$0	$0	$66,944,547	$88,289,305	$74,222,616	$97,887,931
-2	2026	1.071225	$66,944,547	$88,289,305	$0	$0	$66,944,547	$88,289,305	$71,712,672	$94,577,711
-1	2027	1.035000	$66,944,547	$88,289,305	$0	$0	$66,944,547	$88,289,305	$69,287,606	$91,379,431
0	2028	1.000000	$0	$0	$0	$0	$0	$0	$0	$0

ALTERNATIVE LS-9A

INTEREST RATE	**3.500%**
PERIOD	**50**
CONSTRUCTION YEAR	**2017**
BASE YEAR	**2028**
CRF	0.042634

ALTERNATIVE	LS-9a
TOTAL IDC	$235,893,759
IDC ANNUAL EQUIVALENT	$10,057,026
TOTAL BDC	$819,119,713
BDC ANNUAL EQUIVALENT	$34,922,112

FIX	**START YEAR**	**END YEAR**	**TOTAL FIRST COST**
North Stockton	2018	2028	$617,026,815
Central Stockton	2017	2020	$252,282,092
RD 17	2018	2028	$0

PERIOD	YEAR	PRESENT WORTH FACTOR	COSTS PRIOR TO BASE	BENEFITS PRIOR TO BASE	COSTS PERIOD OF ANALYSIS	BENEFITS PERIOD OF ANALYSIS	TOTAL COSTS	TOTAL BENEFITS	PRESENT VALUE OF COSTS	PRESENT VALUE OF BENEFITS
-11	2017	1.459970	$84,094,031	$0	$0	$0	$84,094,031	$0	$122,774,738	$0
-10	2018	1.410599	$145,796,712	$0	$0	$0	$145,796,712	$0	$205,660,661	$0
-9	2019	1.362897	$145,796,712	$0	$0	$0	$145,796,712	$0	$198,705,953	$0
-8	2020	1.316809	$61,702,682	$87,433,429	$0	$0	$61,702,682	$87,433,429	$81,250,649	$115,133,129
-7	2021	1.272279	$61,702,682	$87,433,429	$0	$0	$61,702,682	$87,433,429	$78,503,042	$111,239,739
-6	2022	1.229255	$61,702,682	$87,433,429	$0	$0	$61,702,682	$87,433,429	$75,848,350	$107,478,008
-5	2023	1.187686	$61,702,682	$87,433,429	$0	$0	$61,702,682	$87,433,429	$73,283,430	$103,843,486
-4	2024	1.147523	$61,702,682	$87,433,429	$0	$0	$61,702,682	$87,433,429	$70,805,246	$100,331,871
-3	2025	1.108718	$61,702,682	$87,433,429	$0	$0	$61,702,682	$87,433,429	$68,410,866	$96,939,006
-2	2026	1.071225	$61,702,682	$87,433,429	$0	$0	$61,702,682	$87,433,429	$66,097,455	$93,660,875
-1	2027	1.035000	$61,702,682	$87,433,429	$0	$0	$61,702,682	$87,433,429	$63,862,275	$90,493,599
0	2028	1.000000	$0	$0	$0	$0	$0	$0	$0	$0

ALTERNATIVE 7B

INTEREST RATE	**3.500%**
PERIOD	**50**
CONSTRUCTION YEAR	**2017**
BASE YEAR	**2030**
CRF	0.042634

ALTERNATIVE	LS-7b
TOTAL IDC	$337,967,947
IDC ANNUAL EQUIVALENT	$14,408,827
TOTAL BDC	$1,470,087,291
BDC ANNUAL EQUIVALENT	$62,675,275

FIX	**START YEAR**	**END YEAR**	**TOTAL FIRST COST**
North Stockton	2018	2028	$599,662,745
Central Stockton	2017	2020	$204,029,427
RD 17	2024	2030	$410,052,683

PERIOD	YEAR	PRESENT WORTH FACTOR	COSTS PRIOR TO BASE	BENEFITS PRIOR TO BASE	COSTS PERIOD OF ANALYSIS	BENEFITS PERIOD OF ANALYSIS	TOTAL COSTS	TOTAL BENEFITS	PRESENT VALUE OF COSTS	PRESENT VALUE OF BENEFITS
-13	2017	1.563956	$68,009,809	$0	$0	$0	$68,009,809	$0	$106,364,353	$0
-12	2018	1.511069	$127,976,084	$0	$0	$0	$127,976,084	$0	$193,380,649	$0
-11	2019	1.459970	$127,976,084	$0	$0	$0	$127,976,084	$0	$186,841,207	$0
-10	2020	1.410599	$59,966,275	$90,119,406	$0	$0	$59,966,275	$90,119,406	$84,588,353	$127,122,322
-9	2021	1.362897	$59,966,275	$90,119,406	$0	$0	$59,966,275	$90,119,406	$81,727,877	$122,823,500
-8	2022	1.316809	$59,966,275	$90,119,406	$0	$0	$59,966,275	$90,119,406	$78,964,132	$118,670,048
-7	2023	1.272279	$59,966,275	$90,119,406	$0	$0	$59,966,275	$90,119,406	$76,293,848	$114,657,051
-6	2024	1.229255	$128,308,388	$90,119,406	$0	$0	$128,308,388	$90,119,406	$157,723,770	$110,779,760
-5	2025	1.187686	$128,308,388	$90,119,406	$0	$0	$128,308,388	$90,119,406	$152,390,116	$107,033,584
-4	2026	1.147523	$128,308,388	$90,119,406	$0	$0	$128,308,388	$90,119,406	$147,236,827	$103,414,091
-3	2027	1.108718	$128,308,388	$90,119,406	$0	$0	$128,308,388	$90,119,406	$142,257,804	$99,916,996
-2	2028	1.071225	$68,342,114	$268,570,517	$0	$0	$68,342,114	$268,570,517	$73,209,781	$287,699,452
-1	2029	1.035000	$68,342,114	$268,570,517	$0	$0	$68,342,114	$268,570,517	$70,734,088	$277,970,485
0	2030	1.000000	$0	$0	$0	$0	$0	$0	$0	$0

ALTERNATIVE 8B

INTEREST RATE	**3.500%**	ALTERNATIVE	LS-8b				
PERIOD	**50**	TOTAL IDC	$390,037,243	**FIX**	**START YEAR**	**END YEAR**	**TOTAL FIRST COST**
CONSTRUCTION YEAR	**2017**	IDC ANNUAL EQUIVALENT	$16,628,735	North Stockton	2018	2028	$643,644,882
BASE YEAR	**2030**	TOTAL BDC	$1,492,356,211	Central Stockton	2017	2020	$279,993,296
CRF	0.042634	BDC ANNUAL EQUIVALENT	$63,624,681	RD 17	2024	2030	$409,873,204

PERIOD	YEAR	PRESENT WORTH FACTOR	COSTS PRIOR TO BASE	BENEFITS PRIOR TO BASE	COSTS PERIOD OF ANALYSIS	BENEFITS PERIOD OF ANALYSIS	TOTAL COSTS	TOTAL BENEFITS	PRESENT VALUE OF COSTS	PRESENT VALUE OF BENEFITS
-13	2017	1.563956	$93,331,099	$0	$0	$0	$93,331,099	$0	$145,965,737	$0
-12	2018	1.511069	$157,695,587	$0	$0	$0	$157,695,587	$0	$238,288,859	$0
-11	2019	1.459970	$157,695,587	$0	$0	$0	$157,695,587	$0	$230,230,781	$0
-10	2020	1.410599	$64,364,488	$91,676,195	$0	$0	$64,364,488	$91,676,195	$90,792,467	$129,318,327
-9	2021	1.362897	$64,364,488	$91,676,195	$0	$0	$64,364,488	$91,676,195	$87,722,191	$124,945,244
-8	2022	1.316809	$64,364,488	$91,676,195	$0	$0	$64,364,488	$91,676,195	$84,755,740	$120,720,042
-7	2023	1.272279	$64,364,488	$91,676,195	$0	$0	$64,364,488	$91,676,195	$81,889,604	$116,637,722
-6	2024	1.229255	$132,676,689	$91,676,195	$0	$0	$132,676,689	$91,676,195	$163,093,527	$112,693,451
-5	2025	1.187686	$132,676,689	$91,676,195	$0	$0	$132,676,689	$91,676,195	$157,578,287	$108,882,561
-4	2026	1.147523	$132,676,689	$91,676,195	$0	$0	$132,676,689	$91,676,195	$152,249,552	$105,200,542
-3	2027	1.108718	$132,676,689	$91,676,195	$0	$0	$132,676,689	$91,676,195	$147,101,017	$101,643,036
-2	2028	1.071225	$68,312,201	$271,725,616	$0	$0	$68,312,201	$271,725,616	$73,177,737	$291,079,273
-1	2029	1.035000	$68,312,201	$271,725,616	$0	$0	$68,312,201	$271,725,616	$70,703,128	$281,236,013
0	2030	1.000000	$0	$0	$0	$0	$0	$0	$0	$0

ALTERNATIVE 9B

INTEREST RATE	**3.500%**
PERIOD	**50**
CONSTRUCTION YEAR	**2017**
BASE YEAR	**2030**
CRF	0.042634

ALTERNATIVE	LS-9b
TOTAL IDC	$355,176,353
IDC ANNUAL EQUIVALENT	$15,142,485
TOTAL BDC	$1,478,835,155
BDC ANNUAL EQUIVALENT	$63,048,228

FIX	**START YEAR**	**END YEAR**	**TOTAL FIRST COST**
North Stockton	2018	2028	$593,738,462
Central Stockton	2017	2020	$242,171,508
RD 17	2024	2030	$406,001,626

PERIOD	YEAR	PRESENT WORTH FACTOR	COSTS PRIOR TO BASE	BENEFITS PRIOR TO BASE	COSTS PERIOD OF ANALYSIS	BENEFITS PERIOD OF ANALYSIS	TOTAL COSTS	TOTAL BENEFITS	PRESENT VALUE OF COSTS	PRESENT VALUE OF BENEFITS
-13	2017	1.563956	$80,723,836	$0	$0	$0	$80,723,836	$0	$126,248,532	$0
-12	2018	1.511069	$140,097,682	$0	$0	$0	$140,097,682	$0	$211,697,216	$0
-11	2019	1.459970	$140,097,682	$0	$0	$0	$140,097,682	$0	$204,538,373	$0
-10	2020	1.410599	$59,373,846	$90,820,319	$0	$0	$59,373,846	$90,820,319	$83,752,674	$128,111,029
-9	2021	1.362897	$59,373,846	$90,820,319	$0	$0	$59,373,846	$90,820,319	$80,920,458	$123,778,772
-8	2022	1.316809	$59,373,846	$90,820,319	$0	$0	$59,373,846	$90,820,319	$78,184,017	$119,593,017
-7	2023	1.272279	$59,373,846	$90,820,319	$0	$0	$59,373,846	$90,820,319	$75,540,113	$115,548,809
-6	2024	1.229255	$127,040,784	$90,820,319	$0	$0	$127,040,784	$90,820,319	$156,165,560	$111,641,361
-5	2025	1.187686	$127,040,784	$90,820,319	$0	$0	$127,040,784	$90,820,319	$150,884,599	$107,866,049
-4	2026	1.147523	$127,040,784	$90,820,319	$0	$0	$127,040,784	$90,820,319	$145,782,221	$104,218,405
-3	2027	1.108718	$127,040,784	$90,820,319	$0	$0	$127,040,784	$90,820,319	$140,852,388	$100,694,111
-2	2028	1.071225	$67,666,938	$269,384,136	$0	$0	$67,666,938	$269,384,136	$72,486,515	$288,571,021
-1	2029	1.035000	$67,666,938	$269,384,136	$0	$0	$67,666,938	$269,384,136	$70,035,280	$278,812,581
0	2030	1.000000	$0	$0	$0	$0	$0	$0	$0	$0

US Army Corps
of Engineers®

Sacramento District
Planning Division

Lower San Joaquin River Feasibility Report

San Joaquin County, California

OTHER SOCIAL EFFECTS
REGIONAL ECONOMIC DEVELOPMENT

This page intentionally left blank

INTRODUCTION

In the past, planning studies at the Corps of Engineers have focused primarily on the National Economic Development (NED) account to formulate and evaluate water resource infrastructure projects. In recent years, however, there has been a renewed emphasis on considering the Other Social Effects (OSE), Regional Economic Development (RED), and Environmental Quality (EQ) accounts when making investment decisions, as can be seen in the publication of Engineering Circular (EC) 1105-2-409, "Planning in a Collaborative Environment." EC 1105-2-409 encourages the use of all four accounts in order to develop water resource solutions that are more holistic and acceptable, and which take into account both national and local stakeholder interests.

The following sections describe the OSE and RED assessments developed for the Lower San Joaquin River Feasibility Study (LSJRFS).

Contents

Tables

Figures

PART I — OTHER SOCIAL EFFECTS

The objective of the Other Social Effects (OSE) assessment is to provide a portrait of the social landscape of the Lower San Joaquin Feasibility Study area and offer a glimpse into the potential vulnerability of the people who live there. Table 1 below summarizes the elements commonly included in the OSE account and the metrics used to evaluate them.

TABLE 1: ELEMENTS OF OSE ANALYSIS

SOCIAL ELEMENT	METRICS
Social connectedness	Gender, race, ethnicity, age, rural versus urban communities, rental versus owner-occupied dwellings, and occupation
Community social capital	Education, family structure, rural vs. urban communities, and population growth
Community resilience	Income, political power, neighborhood prestige, employment loss, residential property characteristics, infrastructure and lifelines, family structure, and medical services

This assessment compares the other social effects associated with the without-project and with-project conditions. The 1% annual chance exceedance (ACE) floodplain serves as the baseline to assess effects.

CURRENT SOCIAL LANDSCAPE

Describing the social landscape of the area provides an understanding of who lives in the study area, who has a stake in the problem or issue, and why it is important to them. A demographic profile of the area is performed using social statistics, and the information is presented in a meaningful way through the use of comparisons and rankings. It is important to note that the profile itself is not an OSE analysis but rather a data collection step that provides a basic level of understanding about the social conditions in the area; the data provides input into a more in-depth analysis that targets areas of special concern or relevance to the water resources issue at hand. The basic social statistics of the study area are summarized in Table 2 below. These statistics, along with the social elements listed in Table 1, are indicators used to portray basic information about the social life and the processes of the study area.

TABLE 2: BASIC SOCIAL CHARACTERISTICS OF THE STUDY AREA

SOCIAL STATISTIC	STOCKTON			CALIFORNIA		
	2000	2010	% Δ	2000	2010	% Δ
Population						
Total	243,771	291,707	19.7%	33,871,648	37,253,956	10%
Age						
Median	29.8	30.8	3.4%	33.3	35.2	5.70%
% >65	10.20%	10.00%	-2.0%	10.60%	11.40%	7.50%
% <18	32.40%	29.90%	-7.7%	27.30%	25.00%	-8.40%
Race & Ethnicity						
Asian	19.90%	21.50%	8.0%	10.90%	12.80%	17.40%
Black	11.20%	12.20%	8.9%	6.70%	5.80%	-13.40%
Hispanic	32.50%	40.30%	24.0%	32.40%	37.60%	16%
White	32.20%	22.90%	-28.9%	46.70%	40.10%	-14.10%
Other	4.20%	3.10%	-26.2%	4.30%	3.70%	86%
Education						
% HS Graduates	68.2%	73.70%	8.1%	81%	80.80%	-0.20%
% College Graduates	15.4%	17.50%	13.6%	30.50%	30.20%	-0.90%
Income and Poverty						
% Unemployed	7.3%	10.50%	43.8%	4.30%	7.10%	65.00%
Median Household Income	35,453	$47,246	33.3%	61,400	61,632	0.00%
% Below Poverty	38.4%	23.30%	-39.3%	15.30%	14.40%	-5.90%
Housing						
% Own	51.60%	51.90%	0.6%	56%	55.90%	0%
% Rent	48.40%	48.10%	-0.6%	44%	44.10%	0%
Quality of Life						
Avg. Household Size	3.04	3.17	4.3%	2.98	3.45	16%
Language Other than English Spoken at Home	41.5%	45.1%	8.7%	43.50%	43.20%	-0.70%
Mean Travel Time to Work (minutes)	27.2	26.4	-2.9%	27.1	27	-0.40%

Source: US Census Bureau

SOCIAL EFFECTS ASSESSMENT

A social effects assessment considers the social vulnerability and resiliency of a population. Social vulnerability refers to the sensitivity of a population to natural hazards, whereas social resiliency refers to the population’s ability to respond to and recover from the impacts of a natural hazard. The characteristics that are recognized as having an influence on social vulnerability and resiliency generally include age, gender, race, and socioeconomic status as well as population segments with special needs or those without the normal social safety nets typically necessary to recover from a disaster. The quality of human settlements (e.g., housing type and construction, infrastructure, and lifelines) and the built environment also play an important role in assessing social vulnerability and resiliency, especially as these characteristics influence potential economic losses, injuries, and fatalities from natural hazards. The two tables below provide a discussion of factors that may influence social vulnerability and resiliency and also provides a qualitative assessment of the Lower San Joaquin River Feasibility study area based on indicator statistics from the 2010 U.S. Census. The discussion column is from the article, *Social Vulnerability to Environmental Hazards*, which was published in the June 2003 edition of Social Science Quarterly.

INDICATOR	DISCUSSION	ASSESSMENT
Income, political power, and prestige	This measure focuses on the ability to absorb losses and enhance resilience to hazard impacts. Wealth enables communities to absorb and recover from losses more quickly due to insurance, social safety nets, and entitlement programs.	The median household income of the area is 30% less than the median for the state of California; however, the city's proximity to the state's Capital of Sacramento may provide significant access to of political resources.
Gender	Women can have a more difficult time during recovery than men, often due to sector-specific employment, lower wages, and family care responsibilities.	Women make up 46.0% of the work force while men make up 54.0%; the median income for women in the area is $42,824, which is 89% of the median income for men.
Race and Ethnicity	Race and ethnicity may impose language and cultural barriers that affect access to post-disaster funding	The area is highly diverse in terms of race and ethnicity. Over 40% of the residents speak a language other than English at home; this may contribute to the vulnerability and possibly the resiliency of the community.
Age	Extremes on the age spectrum inhibit the movement out of harm's way. Parents lose time and money caring for children when daycare facilities are affected; the elderly may have mobility constraints or mobility concerns increasing the burden of care and lack of resilience.	Those age 65 and over make up a slightly lower percentage of the community's population as compared to the percentage for the same age category for the state as a whole; the percentage of residents younger than 18 (29.9%) is slightly higher than the state statistic (25%).
Employment Loss	The potential loss of employment following a disaster exacerbates the number of unemployed workers in a community, contributing to a slower recovery from the disaster.	The latest Census indicates that the current unemployment rate in the area may be significantly higher than the state's. A flood event which causes additional unemployment may exacerbate the current unemployment rate.
Rural/Urban	Rural residents may be more vulnerable due to lower incomes, and may be more dependent on locally-based resource extraction economies (farming and fishing). High-density areas (urban) complicate evacuation from harm's way.	The area is highly urbanized and close to many resources.
Residential Property	The value, quality, and density of residential construction affect potential losses and recovery. For example, expensive homes are costly to replace, while mobile homes are easily destroyed and less resilient to hazards.	The area is comprised of a full spectrum of homes – from average quality to excellent. Medium density neighborhoods are typical, with higher density neighborhoods in the downtown area.
Infrastructure and Lifelines	Loss of sewers, bridges, water, communications, and transportation infrastructure may place an insurmountable financial burden on the smaller communities that lack the financial resources to rebuild.	Many of the neighborhoods within the study area are well-established and would most likely have access to the many resources available within the city itself as well as within the greater Sacramento area to the north.

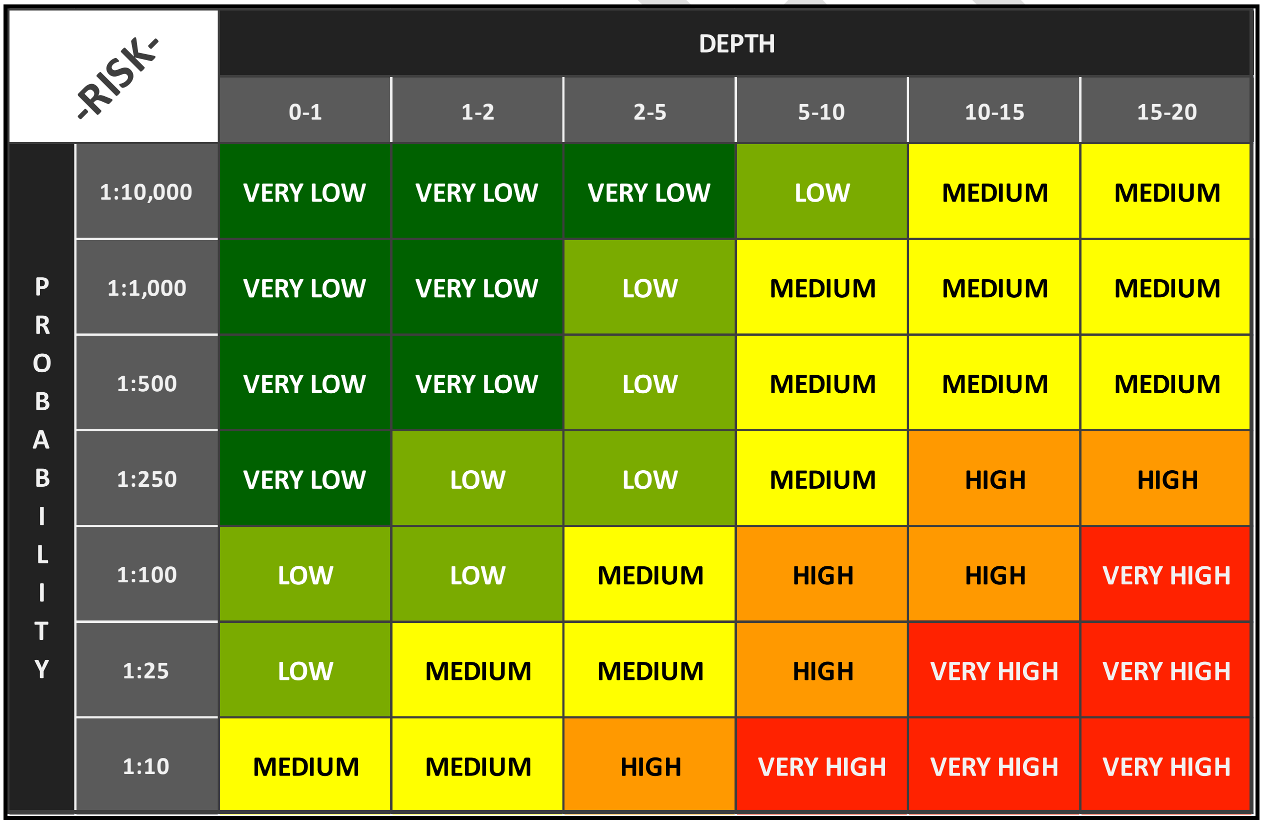
INDICATOR	DISCUSSION	ASSESSMENT
Renters	People that rent typically do so because they are either transient or do not have the financial resources for home ownership. They often lack access to information about financial aid during recovery. In the most extreme cases, renters lack sufficient shelter options when lodging becomes uninhabitable or too costly to afford.	The number of rentals in the area is significant (about 48%), and is higher than the state average of about 44%. The high rental population may contribute to communication cohesion issues; research indicates that renters do not have the same level of community pride as owners do, which may lead to more challenges in redeveloping a community after a flood event.
Occupation	Some occupations, especially those of resource extraction, may be severely impacted by a hazard event. Self-employed fishermen suffer when their means of production is lost and may not have the requisite capital to resume work in a timely fashion and thus will seek alternative employment. Migrant workers engaged in agriculture and low skilled service jobs (e.g., housekeeping, childcare, and gardening) may similarly suffer, as disposable income fades and the need for services decline. Immigration status also affects occupational recovery.	The number of people that live in the area and work in resource extraction occupations is fairly low; the 2010 Census indicates that around 4,329 people (or 3.2% of the total work force) work in the farming, fishing, and forestry occupations.
Family Structure	Families with large numbers of dependents or single-parent households often have limited finances to outsource care for dependents, and thus must juggle work responsibilities and care for family members. All affect the resilience to recover from hazards.	The literature indicates that families having greater than four persons have more financial difficulty than smaller families. Accordingly, community planners need to be aware of issues that may arise.
Education	Education is strongly linked to socioeconomic status, with higher educational attainment resulting in greater lifetime earnings. Lower education constrains the ability to understand warning information and access to recovery information.	Nearly 74% of the population has graduated from high school and 17.5% hold a bachelor’s degree.
Population Growth	Counties experiencing rapid growth lack available quality housing; its social services network may not have had time to adjust to increased populations. New migrants may not speak the language and not be familiar with bureaucracies for obtaining relief or recovery information, all of which increases vulnerability.	Stockton has grown considerably over the past 10-15 years. The population has grown by about 20%--nearly double the state’s population growth rate. Rapid growth is highly correlated with low community cohesion. The sense of belonging, cooperation, and community pride are dynamic factors which help with community resilience but which may not be as strong in cities that have experienced rapid growth.
Medical Services	Health care providers, including physicians, nursing homes, and hospitals are important post-event sources of relief. The lack of proximate medical services will lengthen immediate relief and result in longer recovery from disasters.	The residents of Stockton would have access to medical facilities in nearby areas, which include the greater Sacramento metropolitan area approximately 45 miles to the north.

LIFE SAFETY EVALUATION

A life safety evaluation was conducted for both the No Action alternative and Alternative LS-7a. Life safety was evaluated based on the following variables: (1) the probability of an annual chance exceedance (ACE) event occurring; (2) the probability of levee failure given the occurrence of an ACE event; (3) the depth of flooding that would occur following a levee failure; and (4) the population density in the flooded area.

Life safety risk was evaluated in two parts. First, a risk matrix was developed based on flood probabilities and inundation depths. Probabilities range from the highly improbable to the very likely, while flood depths range from very shallow to catastrophically deep. The risk matrix and associated qualitative risk factors are shown in Figure 1 below. Table 3 provides plain language explanations of the risk factors that appear in each cell of matrix.

FIGURE 1: FLOOD RISK MATRIX



-RISK-		DEPTH					
		0-1	1-2	2-5	5-10	10-15	15-20
PROBABILITY	1:10,000	VERY LOW	VERY LOW	VERY LOW	LOW	MEDIUM	MEDIUM
	1:1,000	VERY LOW	VERY LOW	LOW	MEDIUM	MEDIUM	MEDIUM
	1:500	VERY LOW	VERY LOW	LOW	MEDIUM	MEDIUM	MEDIUM
	1:250	VERY LOW	LOW	LOW	MEDIUM	HIGH	HIGH
	1:100	LOW	LOW	MEDIUM	HIGH	HIGH	VERY HIGH
	1:25	LOW	MEDIUM	MEDIUM	HIGH	VERY HIGH	VERY HIGH
	1:10	MEDIUM	MEDIUM	HIGH	VERY HIGH	VERY HIGH	VERY HIGH

TABLE 3: EXPLANATION OF RISK FACTORS

-RISK-		DEPTH OF FLOODING (FT)					
		0-1	1-2	2-5	5-10	10-15	15-20
PROBABILITY OF EVENT + FAILURE	1:10,000	A 1:10,000 chance of receiving 0-1 feet of flooding in a given year is considered VERY LOW risk.	A 1:10,000 chance of receiving 1-2 feet of flooding in a given year is considered VERY LOW risk.	A 1:10,000 chance of receiving 2-5 feet of flooding in a given year is considered VERY LOW risk.	A 1:10,000 chance of receiving 5-10 feet of flooding in a given year is considered LOW risk.	A 1:10,000 chance of receiving 10-15 feet of flooding in a given year is considered MEDIUM risk.	A 1:10,000 chance of receiving 15-20 feet of flooding in a given year is considered MEDIUM risk.
	1:10,00	A 1:10,00 chance of receiving 0-1 feet of flooding in a given year is considered VERY LOW risk.	A 1:10,00 chance of receiving 1-2 feet of flooding in a given year is considered VERY LOW risk.	A 1:10,00 chance of receiving 2-5 feet of flooding in a given year is considered LOW risk.	A 1:10,00 chance of receiving 5-10 feet of flooding in a given year is considered MEDIUM risk.	A 1:10,00 chance of receiving 10-15 feet of flooding in a given year is considered MEDIUM risk.	A 1:10,00 chance of receiving 15-20 feet of flooding in a given year is considered MEDIUM risk.
	1:500	A 1:500 chance of receiving 0-1 feet of flooding in a given year is considered VERY LOW risk.	A 1:500 chance of receiving 1-2 feet of flooding in a given year is considered VERY LOW risk.	A 1:500 chance of receiving 2-5 feet of flooding in a given year is considered LOW risk.	A 1:500 chance of receiving 5-10 feet of flooding in a given year is considered MEDIUM risk.	A 1:500 chance of receiving 10-15 feet of flooding in a given year is considered MEDIUM risk.	A 1:500 chance of receiving 15-20 feet of flooding in a given year is considered MEDIUM risk.
	1:250	A 1:250 chance of receiving 0-1 feet of flooding in a given year is considered VERY LOW risk.	A 1:250 chance of receiving 1-2 feet of flooding in a given year is considered LOW risk.	A 1:250 chance of receiving 2-5 feet of flooding in a given year is considered LOW risk.	A 1:250 chance of receiving 5-10 feet of flooding in a given year is considered MEDIUM risk.	A 1:250 chance of receiving 10-15 feet of flooding in a given year is considered HIGH risk.	A 1:250 chance of receiving 15-20 feet of flooding in a given year is considered HIGH risk.
	1:100	A 1:100 chance of receiving 0-1 feet of flooding in a given year is considered LOW risk.	A 1:100 chance of receiving 1-2 feet of flooding in a given year is considered LOW risk.	A 1:100 chance of receiving 2-5 feet of flooding in a given year is considered MEDIUM risk.	A 1:100 chance of receiving 5-10 feet of flooding in a given year is considered HIGH risk.	A 1:100 chance of receiving 10-15 feet of flooding in a given year is considered HIGH risk.	A 1:100 chance of receiving 15-20 feet of flooding in a given year is considered VERY HIGH risk.
	1:25	A 1:25 chance of receiving 0-1 feet of flooding in a given year is considered LOW risk.	A 1:25 chance of receiving 1-2 feet of flooding in a given year is considered MEDIUM risk.	A 1:25 chance of receiving 2-5 feet of flooding in a given year is considered MEDIUM risk.	A 1:25 chance of receiving 5-10 feet of flooding in a given year is considered HIGH risk.	A 1:25 chance of receiving 10-15 feet of flooding in a given year is considered VERY HIGH risk.	A 1:25 chance of receiving 15-20 feet of flooding in a given year is considered VERY HIGH risk.
	1:10	A 1:10 chance of receiving 0-1 feet of flooding in a given year is considered MEDIUM risk.	A 1:10 chance of receiving 1-2 feet of flooding in a given year is considered MEDIUM risk.	A 1:10 chance of receiving 2-5 feet of flooding in a given year is considered HIGH risk.	A 1:10 chance of receiving 5-10 feet of flooding in a given year is considered VERY HIGH risk.	A 1:10 chance of receiving 10-15 feet of flooding in a given year is considered VERY HIGH risk.	A 1:10 chance of receiving 15-20 feet of flooding in a given year is considered VERY HIGH risk.

The tables and figures below are provided to compare flood risk to the population of the LSJRFS study area under the No Action alternative and Alternative LS-7a. Tables 4 and 5 list the number of people in each risk category for the existing and future condition. Tables 6 and 7 further illustrate the potential impact of Alternative LS-7a on flood risk by showing the number of people affected by each combination of the No Action alternative and Alternative LS-7a flood risk categories. The maps in figures 2 through 9 show existing and future flood risk for both alternatives based on the probability and depth of flooding.

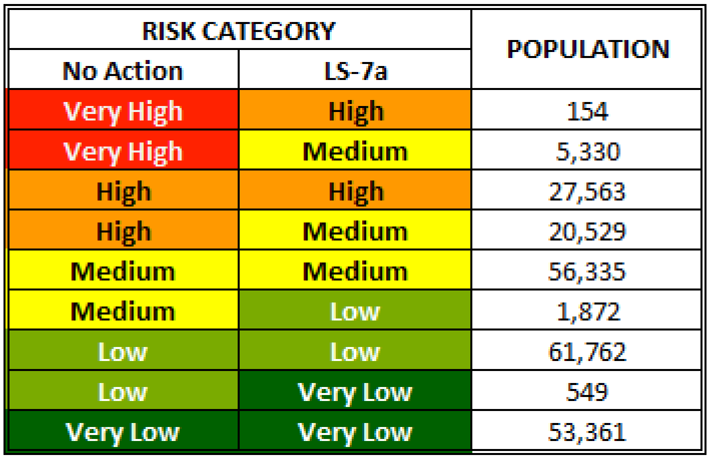
TABLE 4: POPULATION BY FLOOD RISK CATEGORY—EXISTING CONDITION

FLOOD RISK	ALTERNATIVE	
	NO ACTION	LS-7A
Very Low	53,361	53,910
Low	62,311	63,633
Medium	58,207	82,194
High	48,092	27,717
Very High	5,484	0

TABLE 5: POPULATION BY FLOOD RISK CATEGORY—FUTURE CONDITION

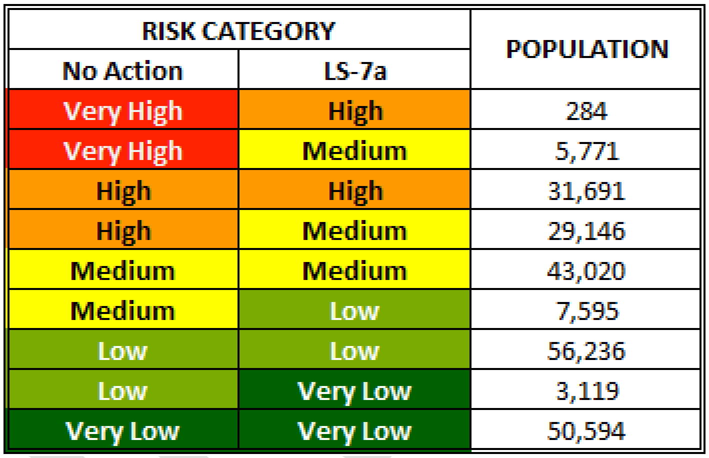
FLOOD RISK	ALTERNATIVE	
	NO ACTION	LS-7A
Very Low	50,594	53,713
Low	59,355	63,831
Medium	50,615	77,937
High	60,837	31,975
Very High	6,054	0

Table 6: Project Impact on Flood Risk—Existing condition



RISK CATEGORY		POPULATION
No Action	LS-7a	
Very High	High	154
Very High	Medium	5,330
High	High	27,563
High	Medium	20,529
Medium	Medium	56,335
Medium	Low	1,872
Low	Low	61,762
Low	Very Low	549
Very Low	Very Low	53,361

Table 7: Project Impact on Flood Risk—Future condition



RISK CATEGORY		POPULATION
No Action	LS-7a	
Very High	High	284
Very High	Medium	5,771
High	High	31,691
High	Medium	29,146
Medium	Medium	43,020
Medium	Low	7,595
Low	Low	56,236
Low	Very Low	3,119
Very Low	Very Low	50,594

FIGURE 2: FLOOD RISK—STUDY AREA—EXISTING CONDITION

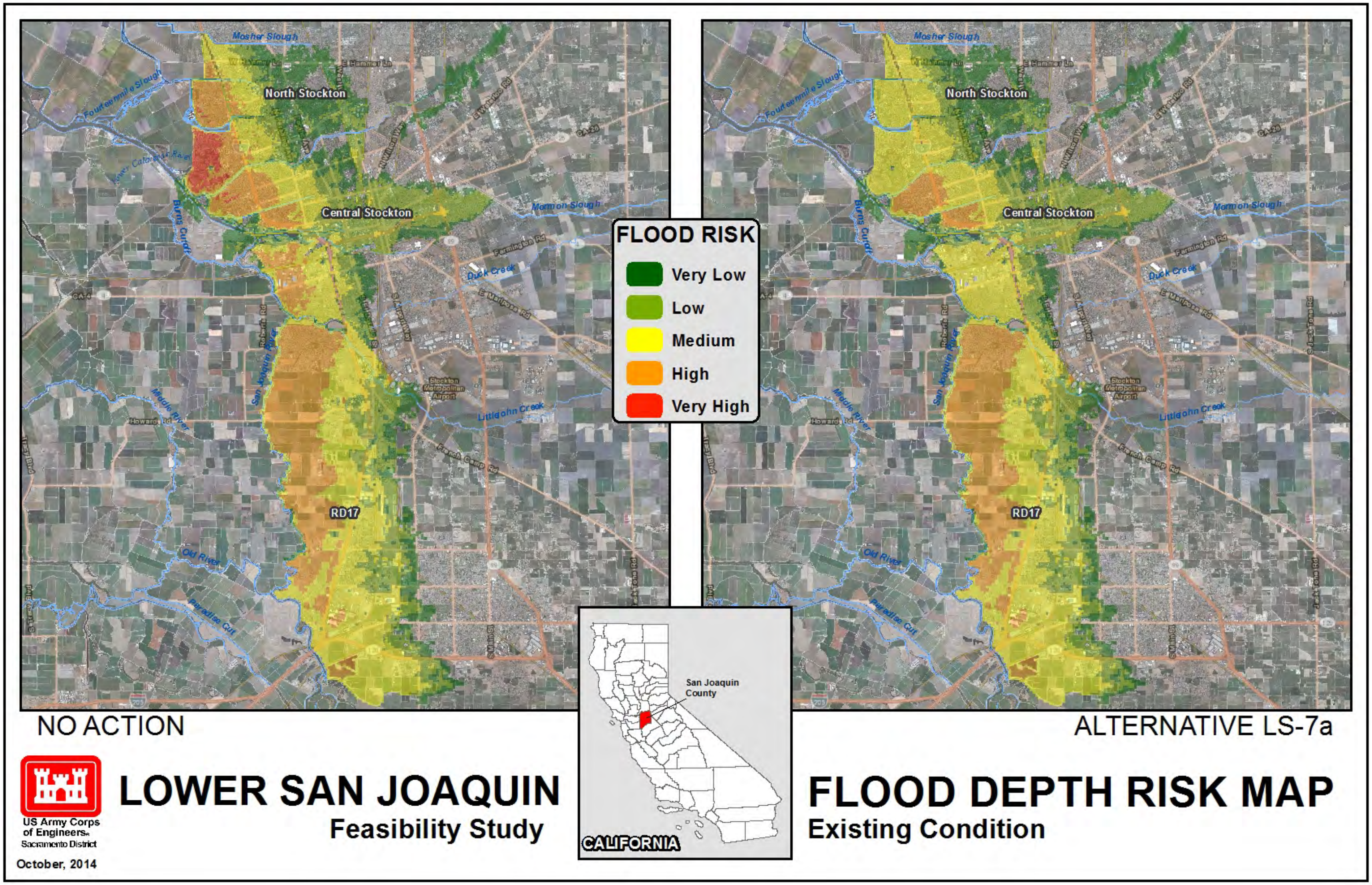


FIGURE 3: FLOOD RISK—STUDY AREA—FUTURE CONDITION

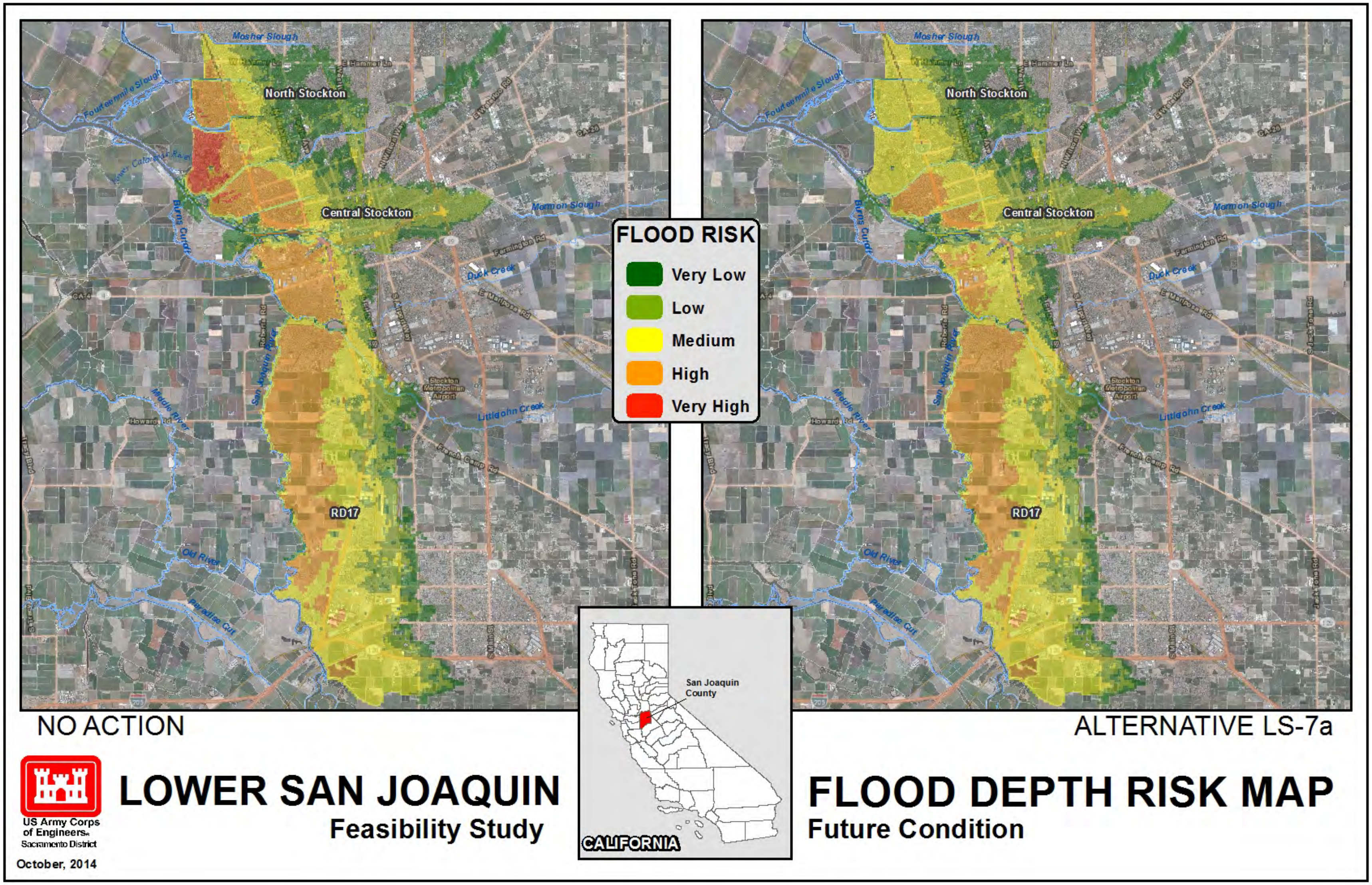


FIGURE 4: FLOOD RISK—NORTH STOCKTON—EXISTING CONDITION

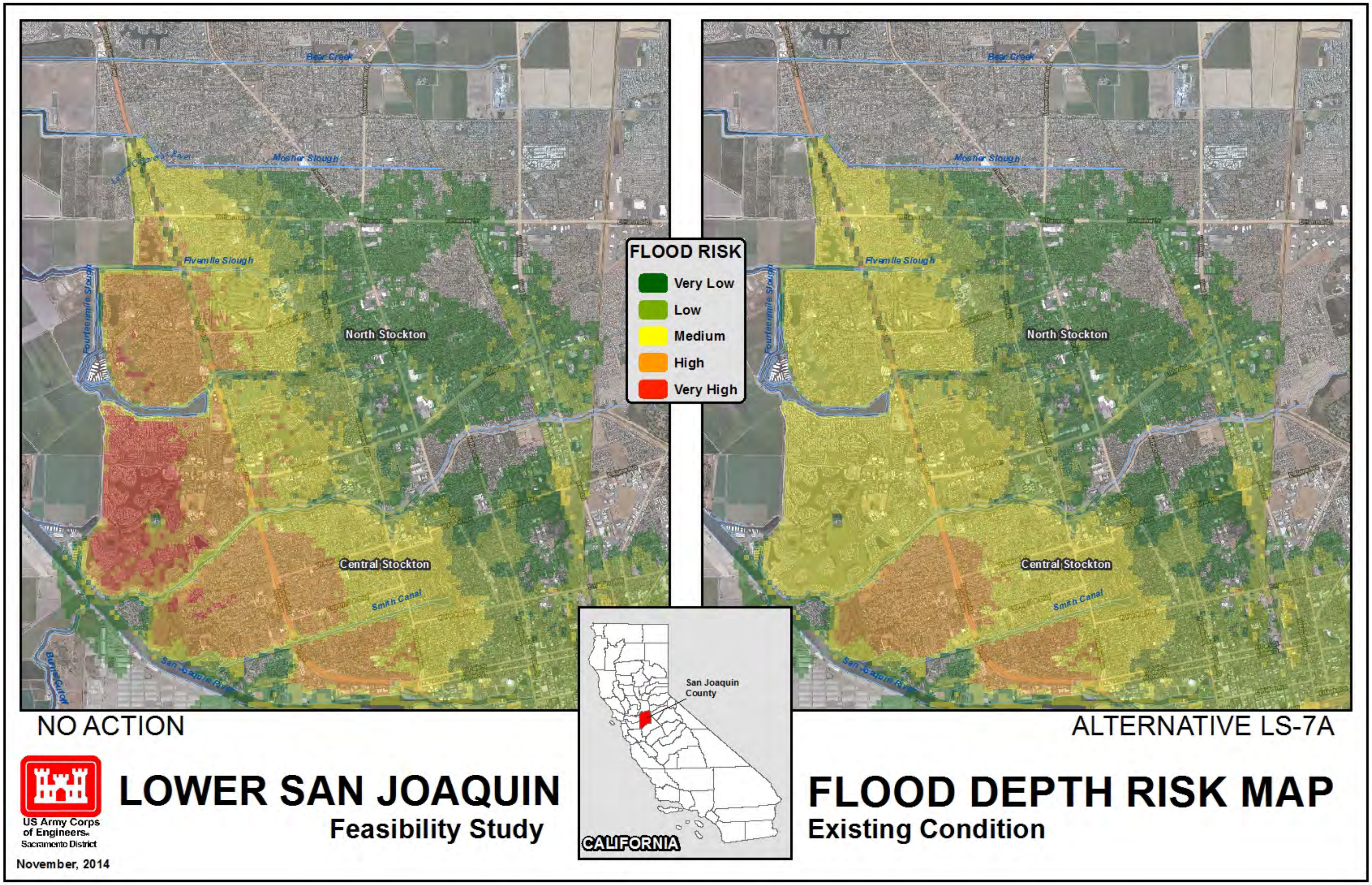


FIGURE 5: FLOOD RISK—NORTH STOCKTON—FUTURE CONDITION

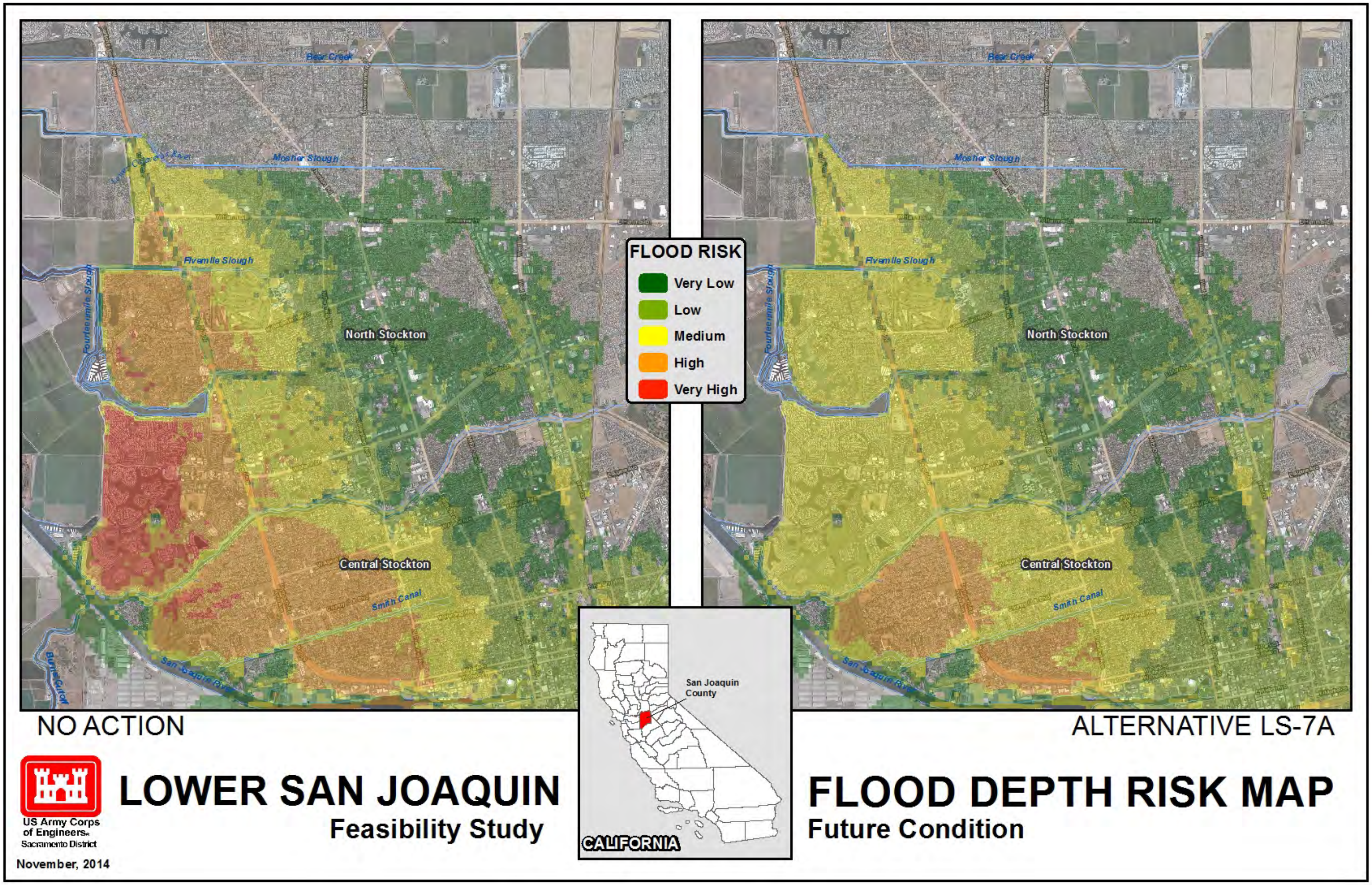


FIGURE 6: FLOOD RISK—CENTRAL STOCKTON—EXISTING CONDITION

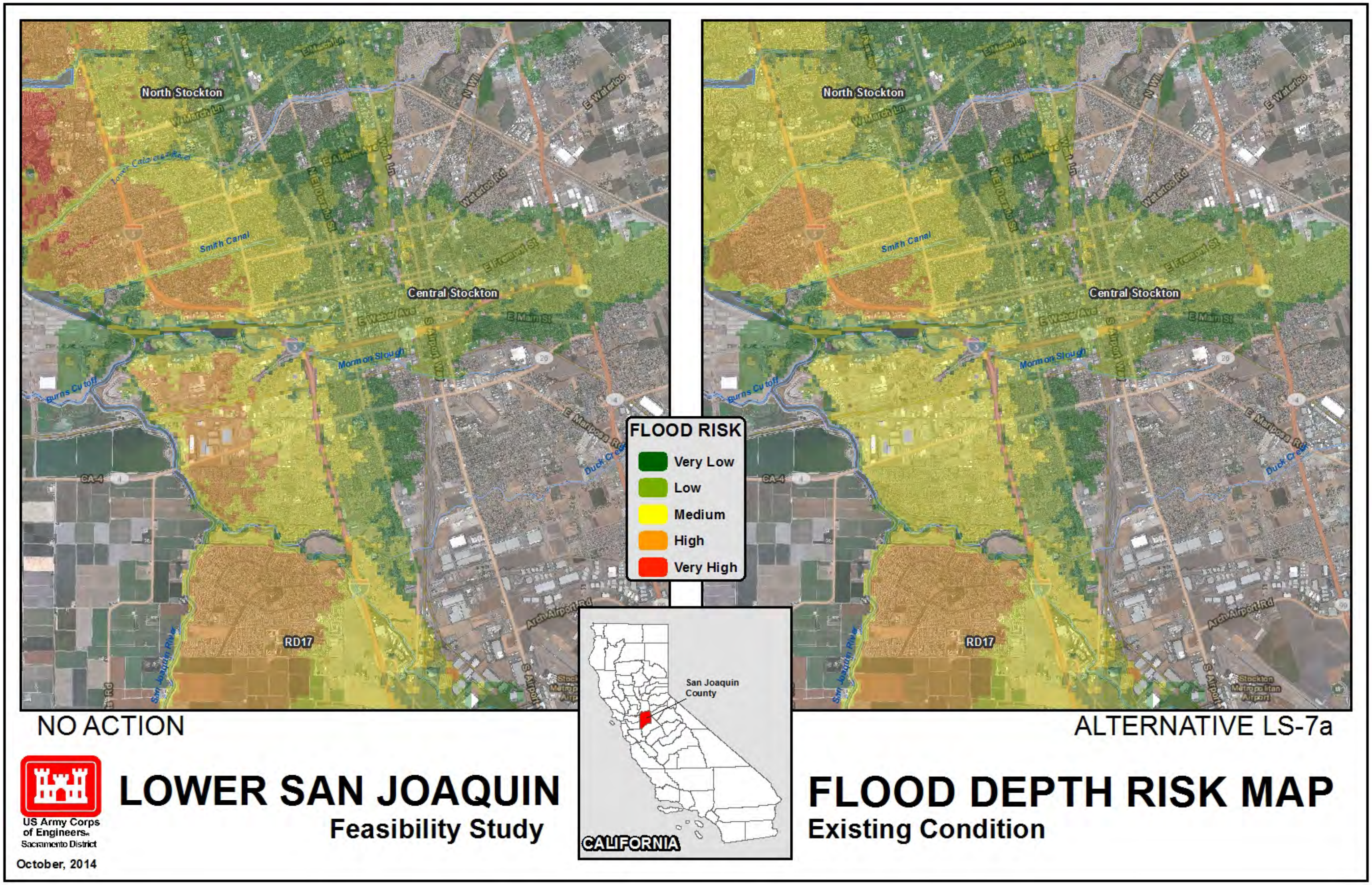


Figure 7: Flood Risk—Central Stockton—Future Condition

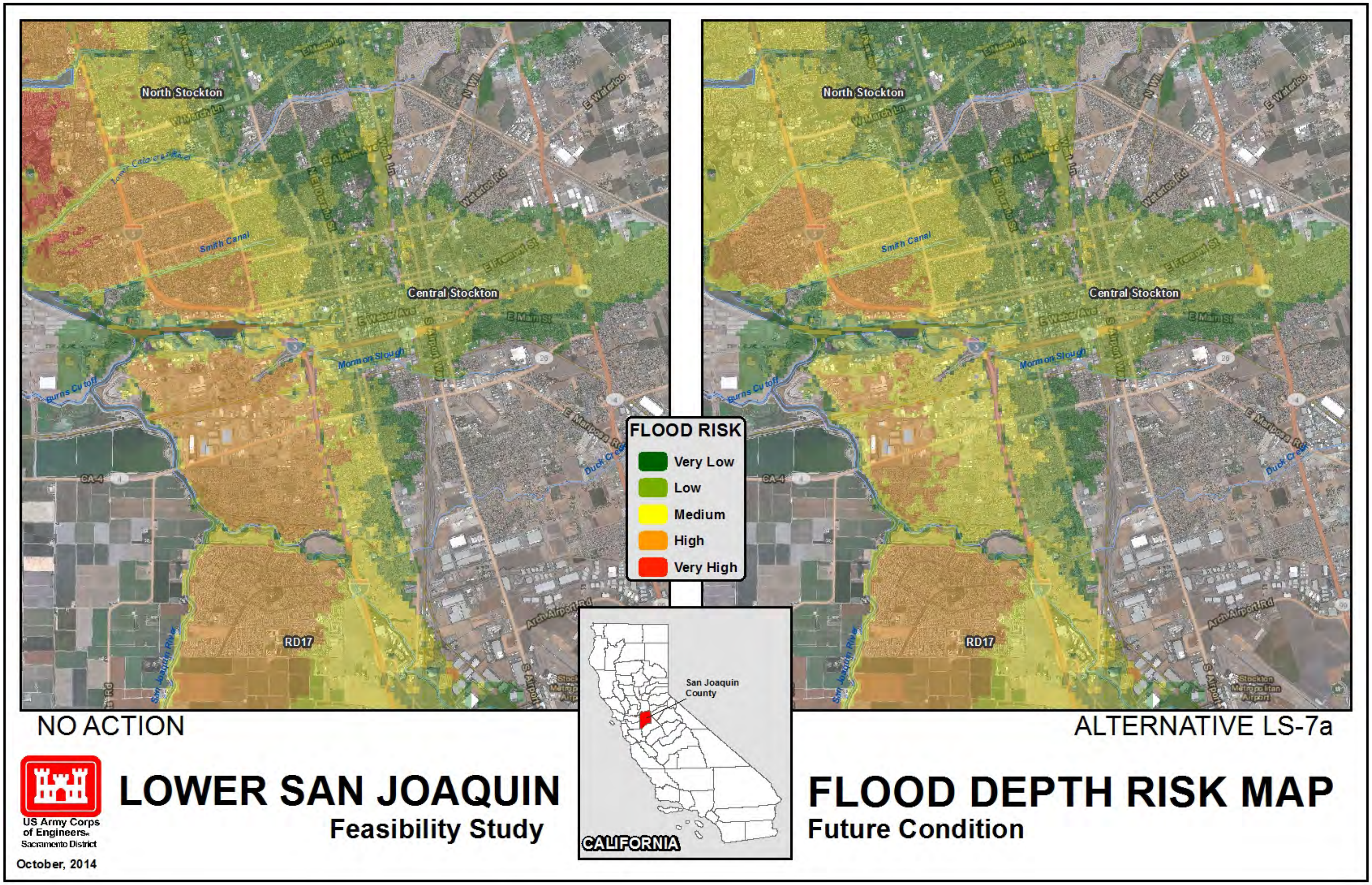


FIGURE 8: FLOOD RISK—RD17—EXISTING CONDITION

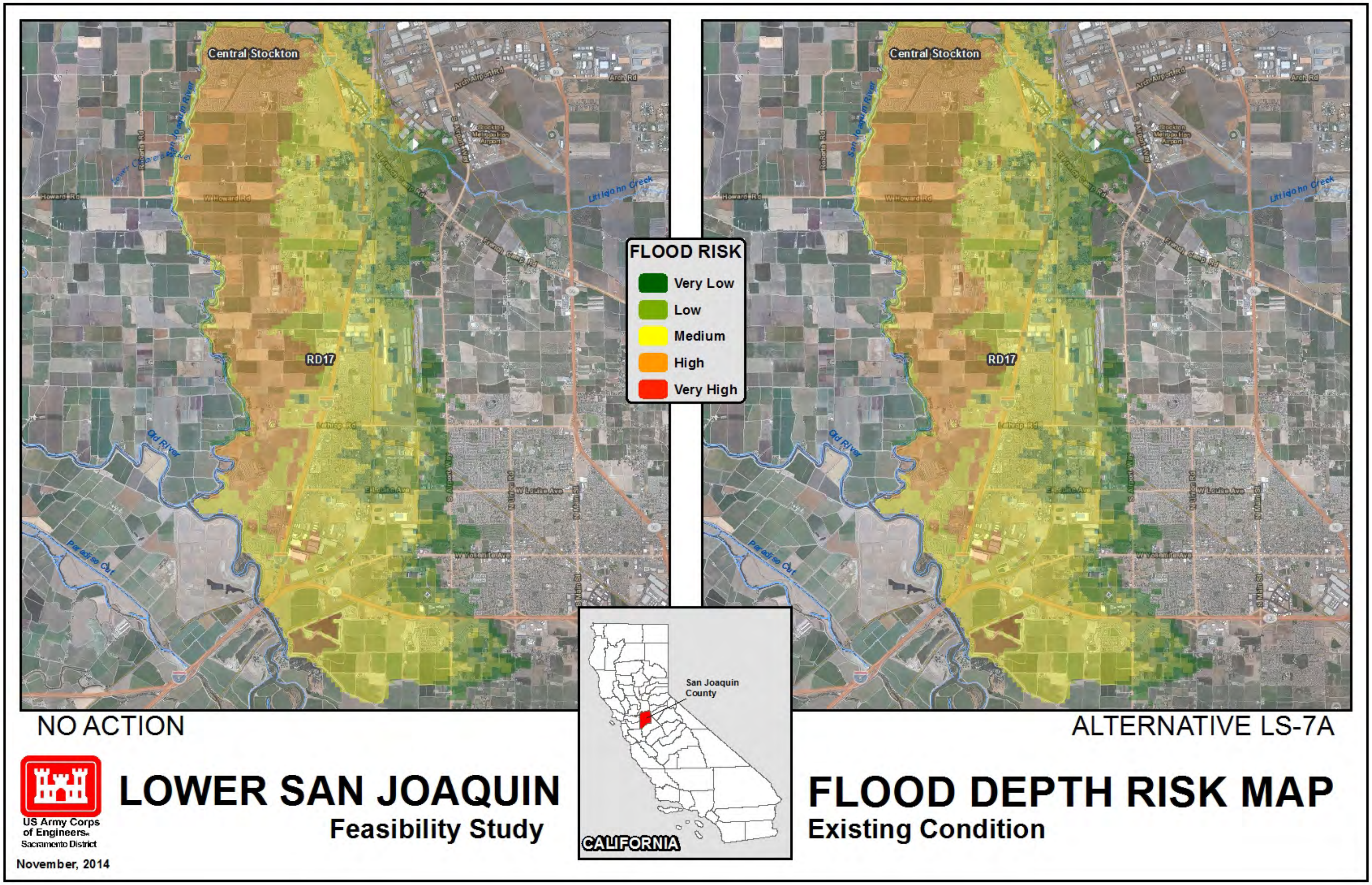
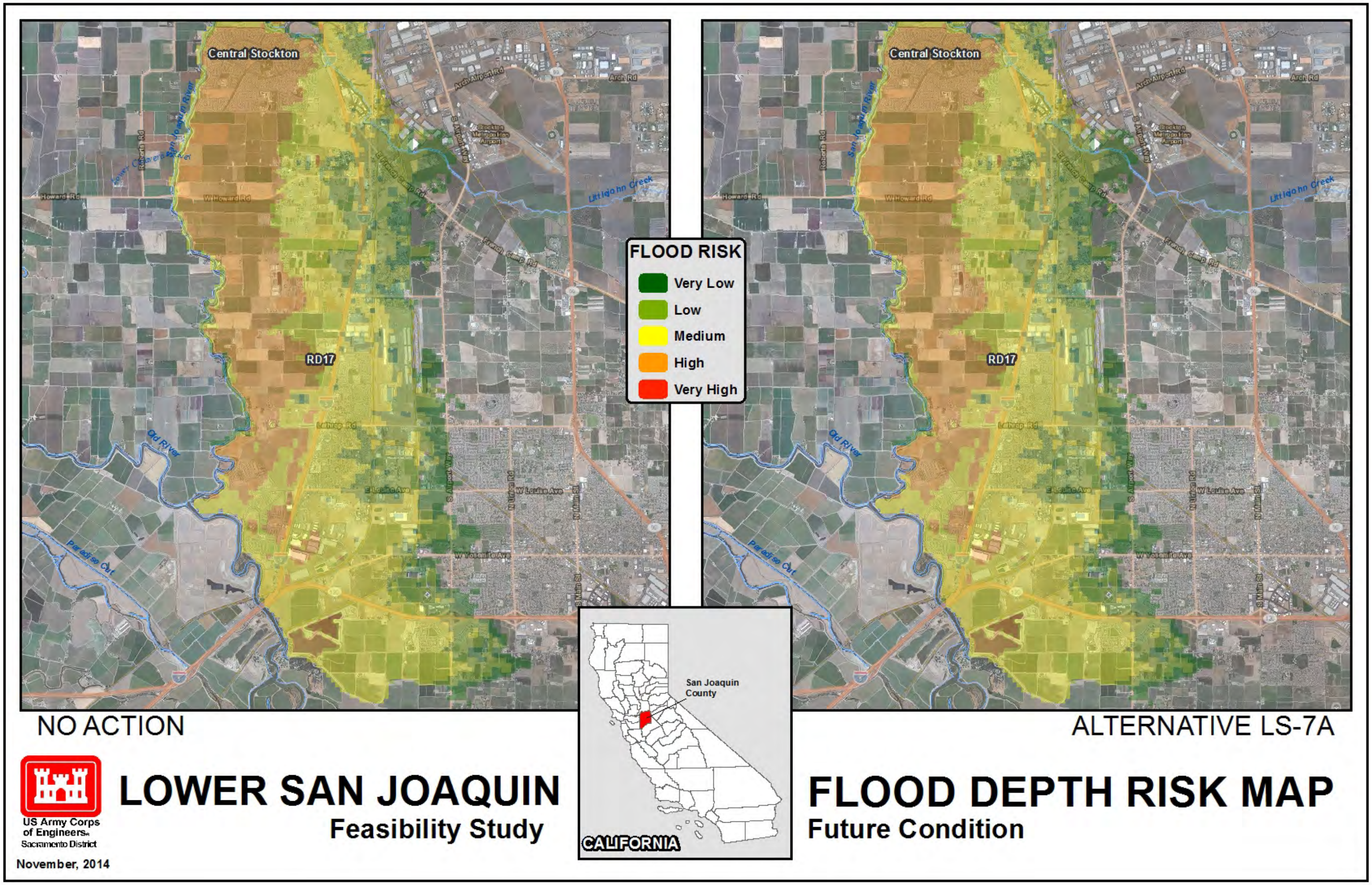


FIGURE 9: FLOOD RISK—RD17—FUTURE CONDITION



The second part of the life safety evaluation was to adjust the flood risk factors up or down based on population density in the affected area. The population density metric was selected because it represents the severity of consequences in the risk equation. In other words, the more people living in a flooded area, the higher the life safety risk, *ceteris paribus*. Conversely, the fewer people living in a flooded area, the lower the life safety risk, *ceteris paribus*.

According to the US Census Bureau, the average metropolitan statistical area (MSA) has a population density of roughly 4,400 people per square mile[1]. The population density of the LSJRFS study area is reasonably close to that estimate with an average of 4,126 people per square mile.

The risk matrix on page 11 is designed to describe flood risk in an area of average population density. For life safety risk estimation purposes, portions of the study area with a population density within one standard deviation below or two standard deviations above the mean population density were deemed average. Flood risk was assessed for these areas using the risk factors as shown in the matrix.

For areas more than two standard deviations above the mean, the risk factor was increased by one increment (medium becomes high, high becomes very high, etc.) For areas more than one standard deviation below the mean[2], the risk factor was reduced by one increment (medium becomes low, low becomes very low, etc.) Table 8 summarizes the risk adjustment factors and the total population affected by each factor adjustment. The maps in figures 10 through 13 provide graphic representations of the population density classifications shown in Table 8.

TABLE 8: RISK ADJUSTMENT BY DEVIATION FROM NATIONAL MEAN POPULATION DENSITY

POPULATION DENSITY DEVIATIONS FROM MEAN	RISK FACTOR ADJUSTMENT	POPULATION IMPACTED
More than 1 below	-1	8,978
1 below to 1 above	0	37,053
1 above to 2 above	0	62,547
2 above to 3 above	+1	45,618
More than 3 above	+1	73,258

---

[1] Data is from the report *Distance Profiles for U.S. Metropolitan Statistical Areas: 2000 and 2010* (US Census Bureau).

[2] Zero is 1.05 standard deviations below the mean. Therefore one standard deviation below the mean was deemed an appropriate threshold to define areas of low population density.

FIGURE 10: POPULATION DENSITY MAP—STUDY AREA

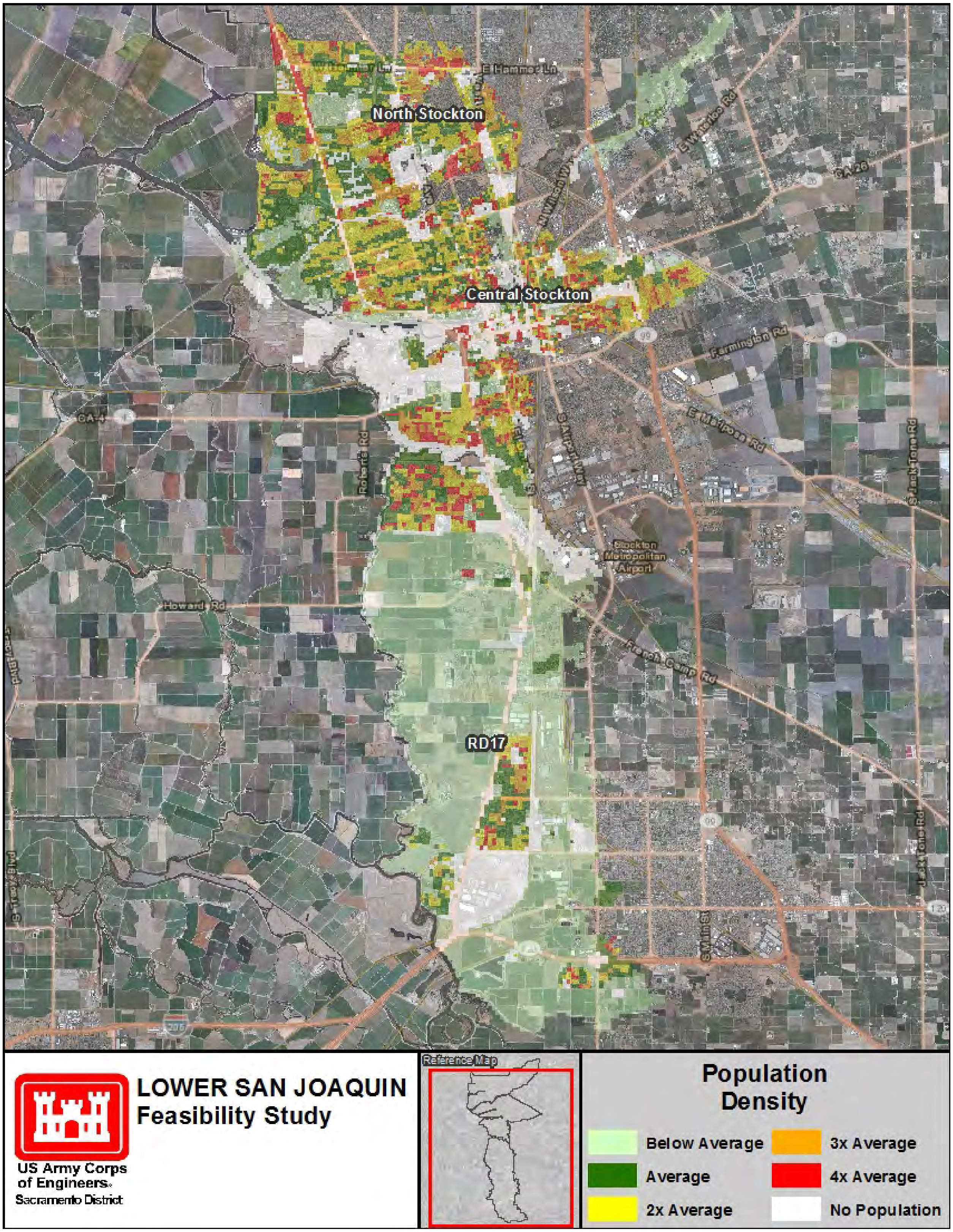


FIGURE 11: POPULATION DENSITY MAP—NORTH STOCKTON

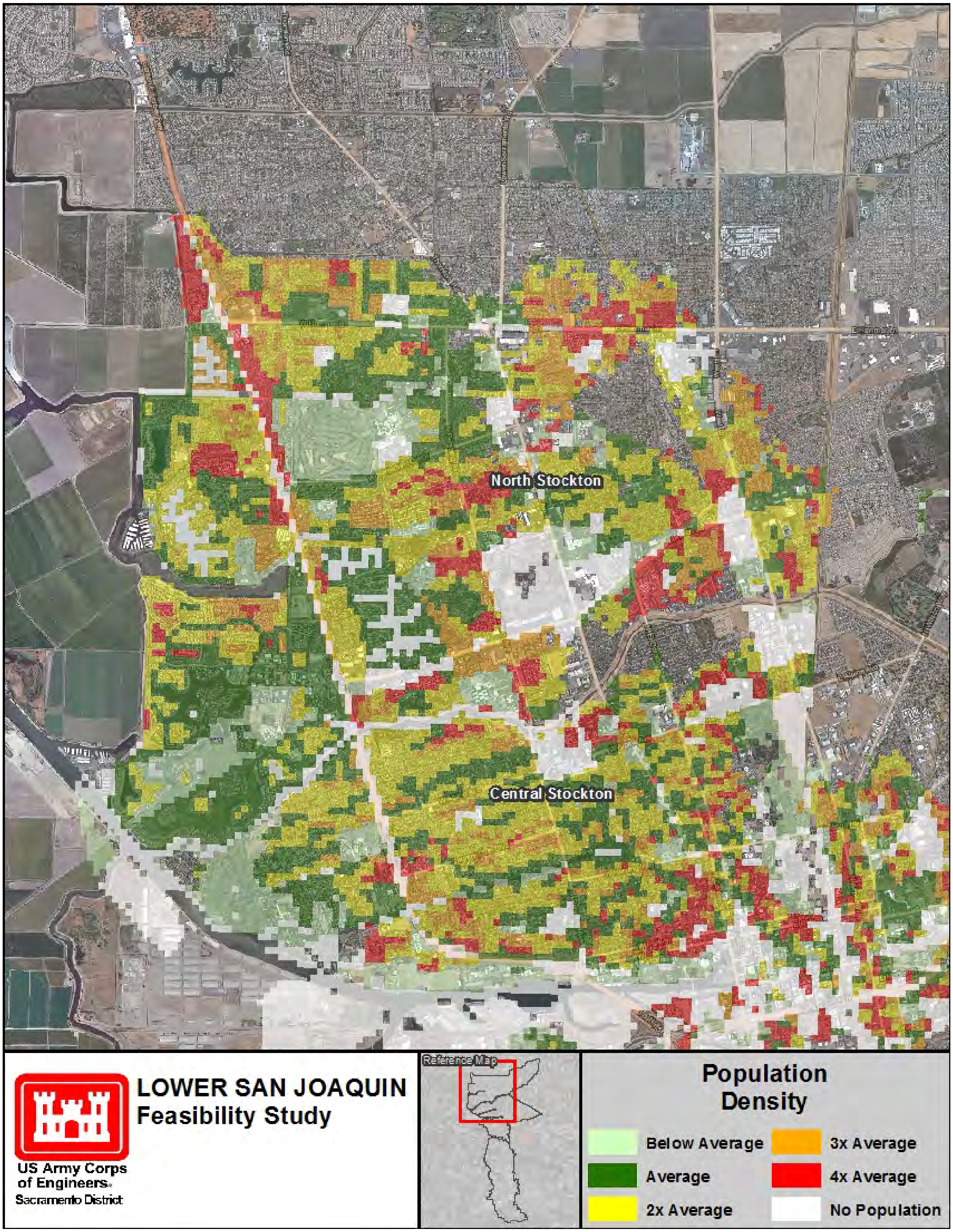


FIGURE 12: POPULATION DENSITY MAP—CENTRAL STOCKTON

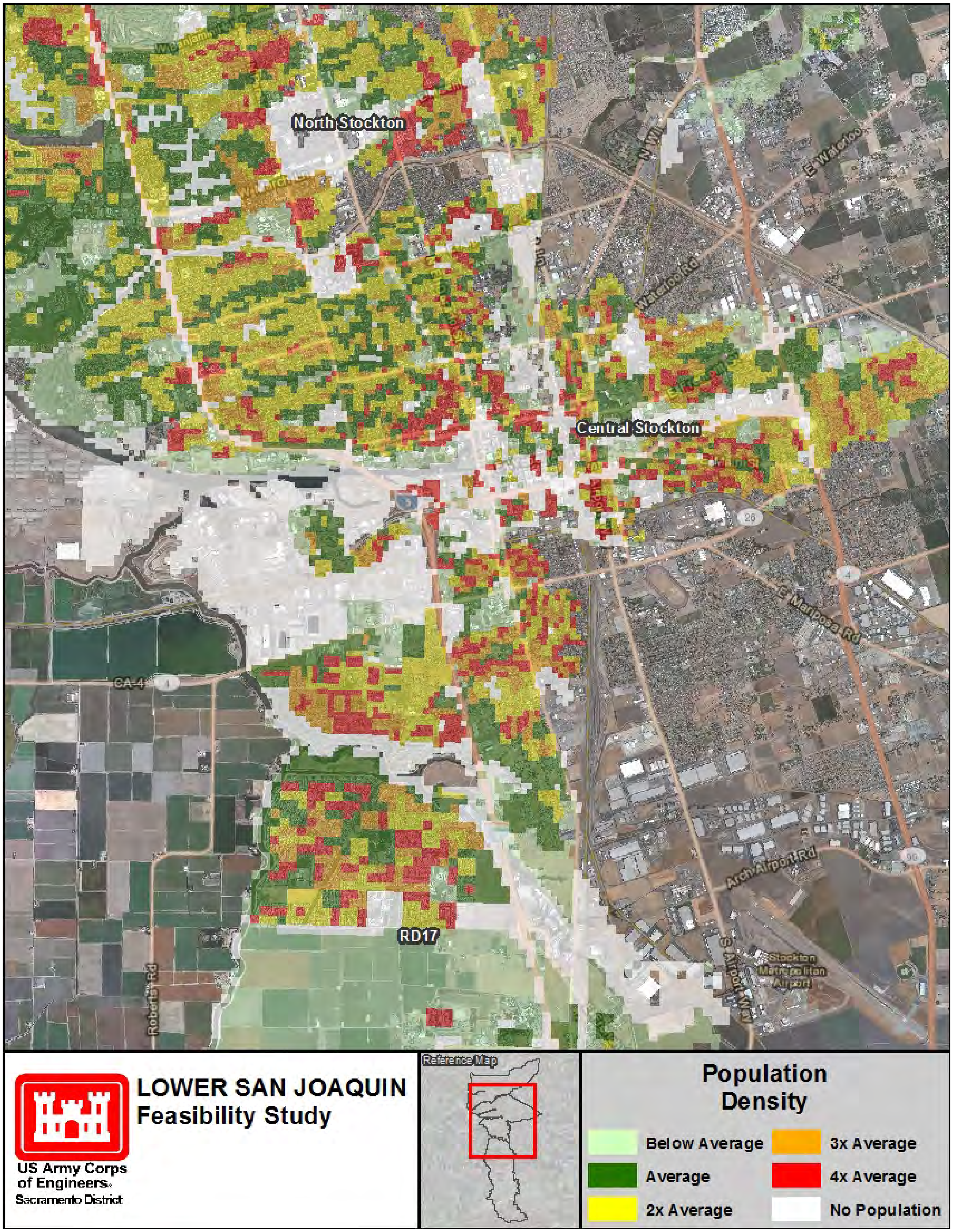
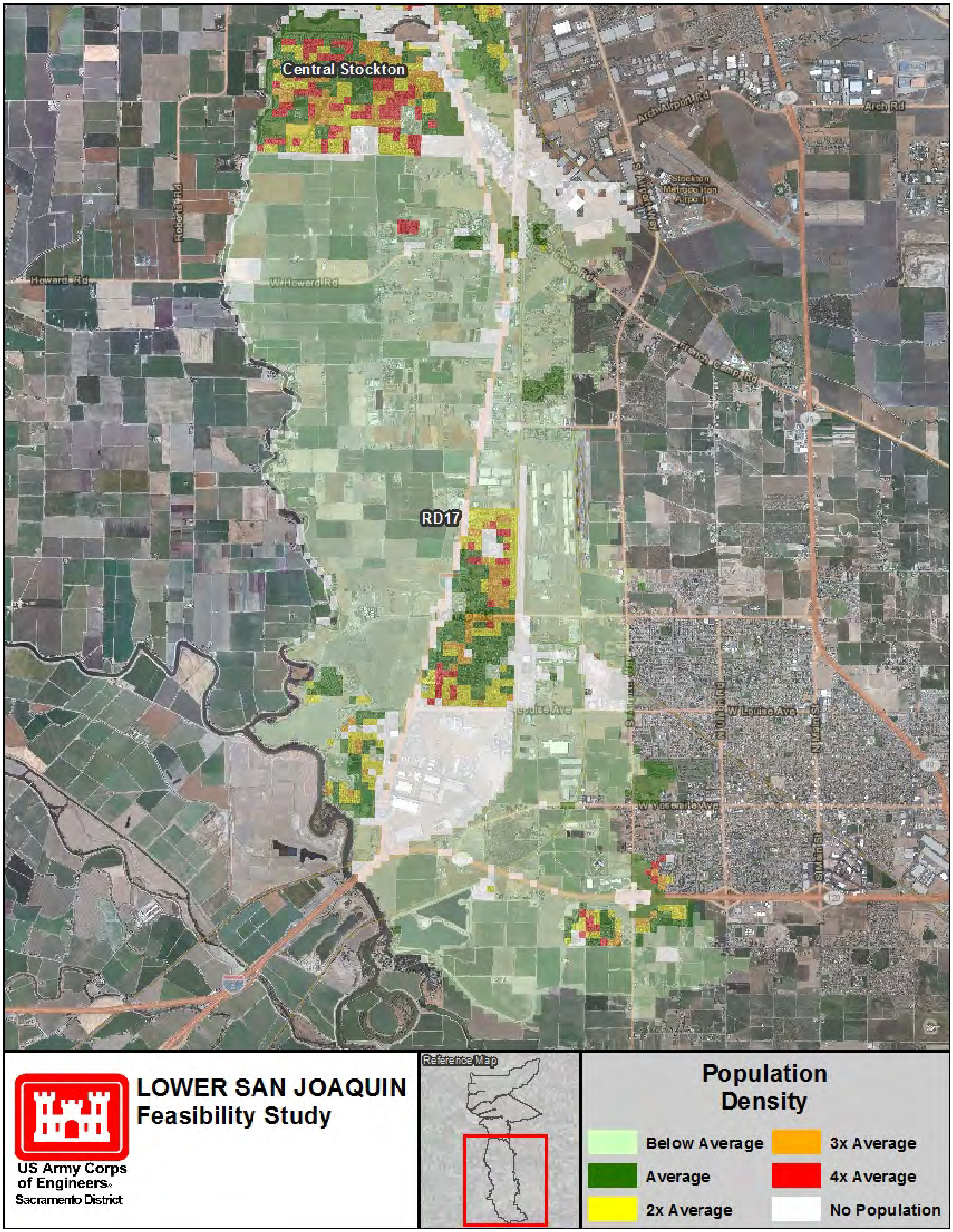


FIGURE 13: POPULATION DENSITY MAP—RD17



In this analysis, flood risk adjusted for population density will be referred as life safety risk. The tables and figures below compare life safety risk for the No Action alternative and Alternative LS-7a. Tables 9 and 10 list the number of people in each risk category for the existing and future condition. Tables 11 and 12 show the number of people affected by each combination of No Action and Alternative LS-7a life safety risk categories. The maps in figures 14 through 21 show existing and future life safety risk for both alternatives.

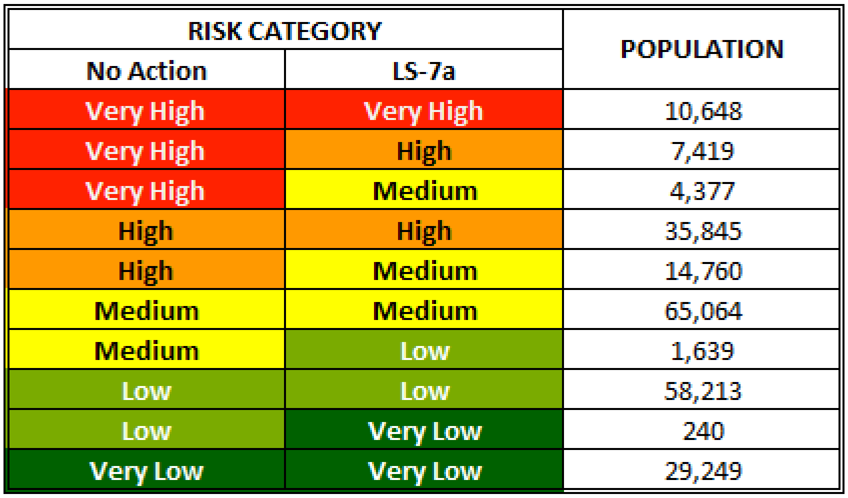
TABLE 9: POPULATION BY LIFE SAFETY RISK CATEGORY—EXISTING CONDITION

FLOOD RISK	ALTERNATIVE	
	NO ACTION	LS-7A
Very Low	29,249	29,489
Low	58,453	59,853
Medium	66,703	84,201
High	50,605	43,264
Very High	22,444	10,648

TABLE 10: POPULATION BY LIFE SAFETY RISK CATEGORY—FUTURE CONDITION

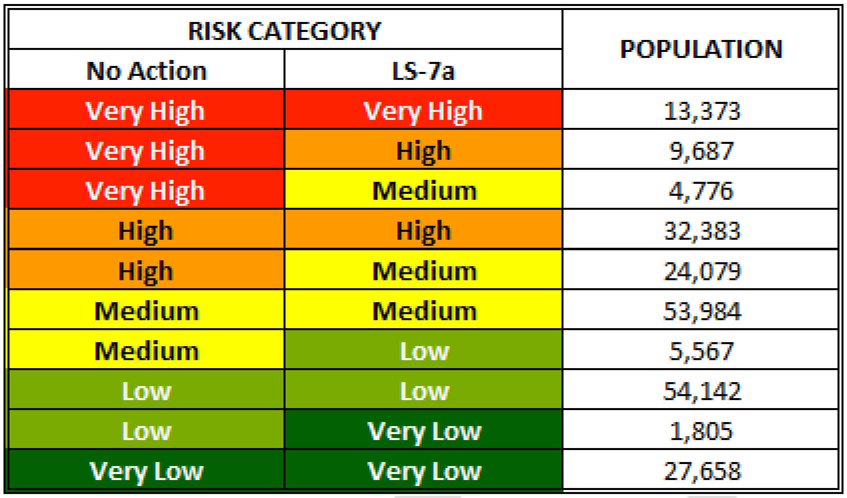
FLOOD RISK	ALTERNATIVE	
	NO ACTION	LS-7A
Very Low	27,658	29,462
Low	55,947	59,709
Medium	59,551	82,839
High	56,463	42,071
Very High	27,837	13,373

TABLE 11: PROJECT IMPACT ON LIFE SAFETY RISK—EXISTING CONDITION



RISK CATEGORY		POPULATION
No Action	LS-7a	
Very High	Very High	10,648
Very High	High	7,419
Very High	Medium	4,377
High	High	35,845
High	Medium	14,760
Medium	Medium	65,064
Medium	Low	1,639
Low	Low	58,213
Low	Very Low	240
Very Low	Very Low	29,249

TABLE 12: PROJECT IMPACT ON LIFE SAFETY RISK—FUTURE CONDITION



RISK CATEGORY		POPULATION
No Action	LS-7a	
Very High	Very High	13,373
Very High	High	9,687
Very High	Medium	4,776
High	High	32,383
High	Medium	24,079
Medium	Medium	53,984
Medium	Low	5,567
Low	Low	54,142
Low	Very Low	1,805
Very Low	Very Low	27,658

FIGURE 14: LIFE SAFETY RISK—STUDY AREA—EXISTING CONDITION

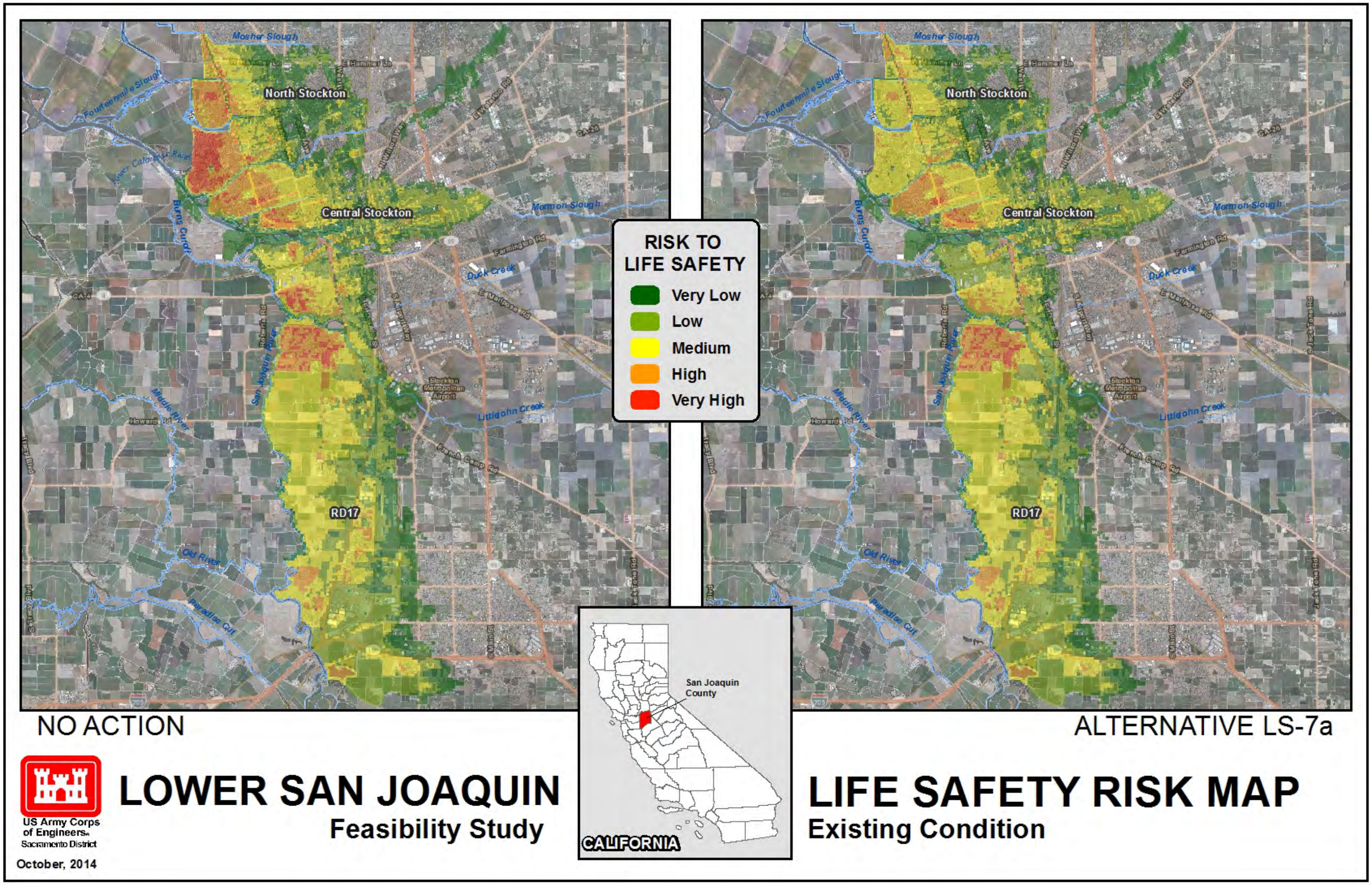


FIGURE 15: LIFE SAFETY RISK—STUDY AREA—FUTURE CONDITION

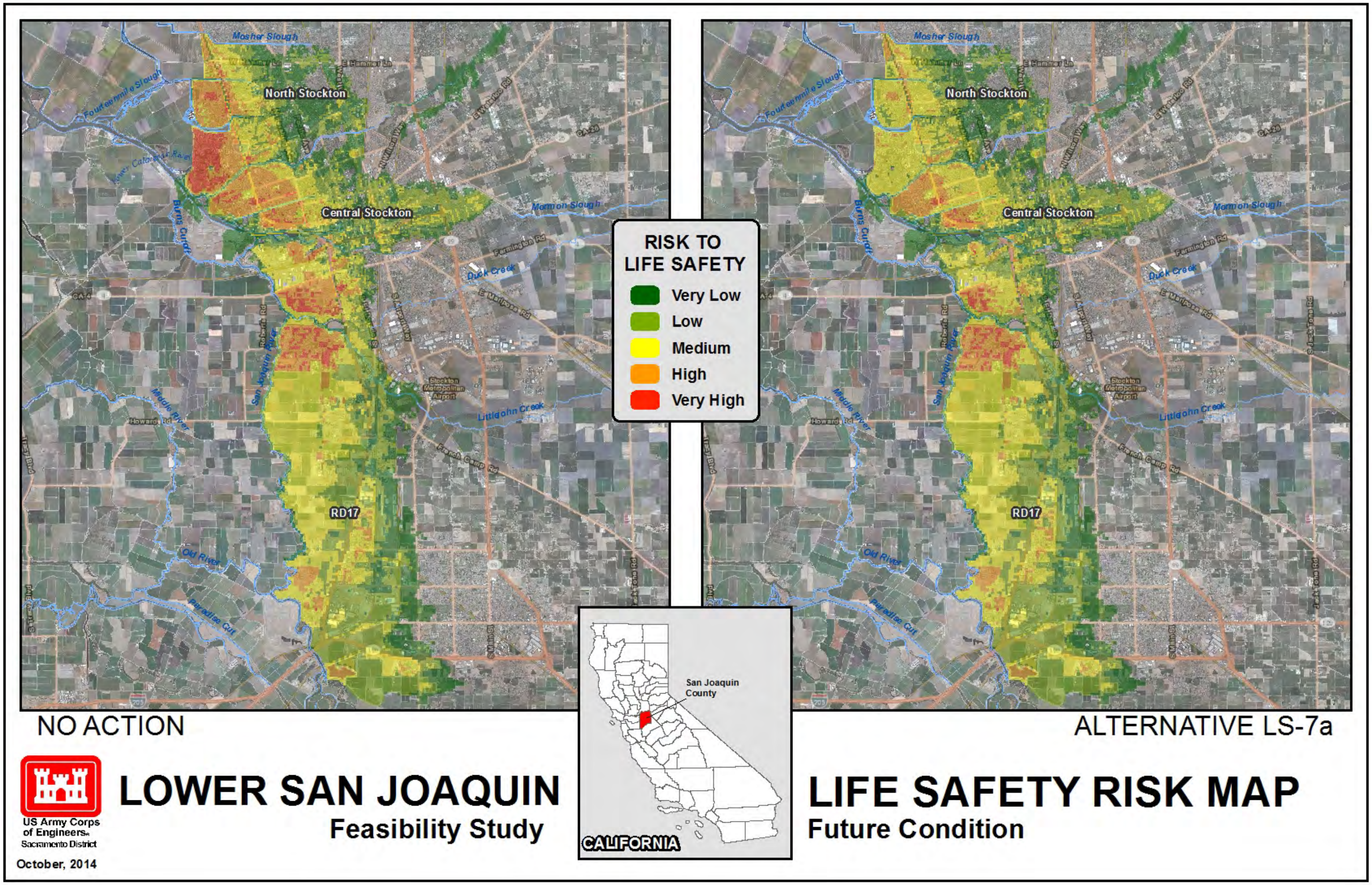


FIGURE 16: LIFE SAFETY RISK—NORTH STOCKTON—EXISTING CONDITION

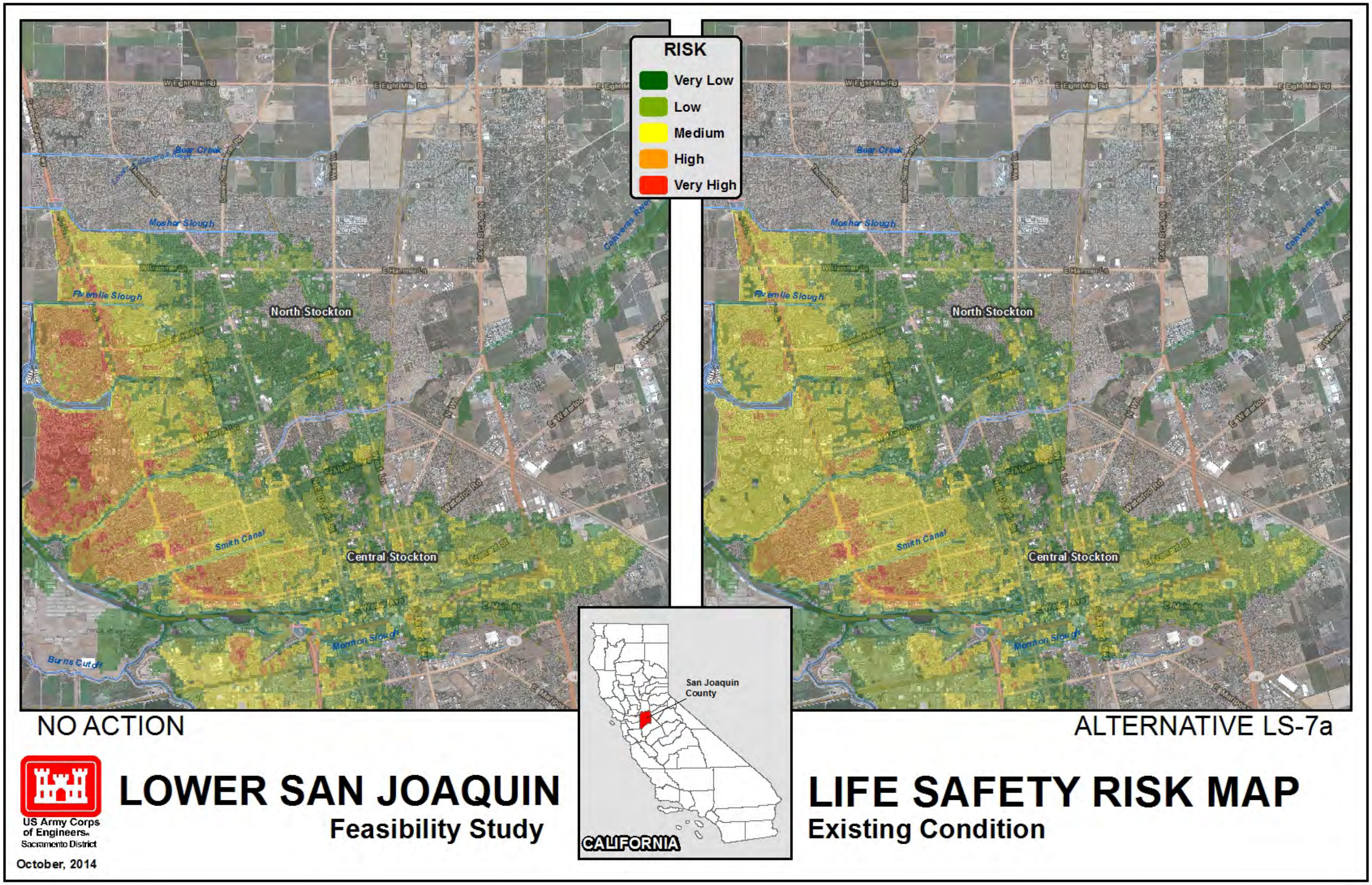


FIGURE 17: LIFE SAFETY RISK—NORTH STOCKTON—FUTURE CONDITION

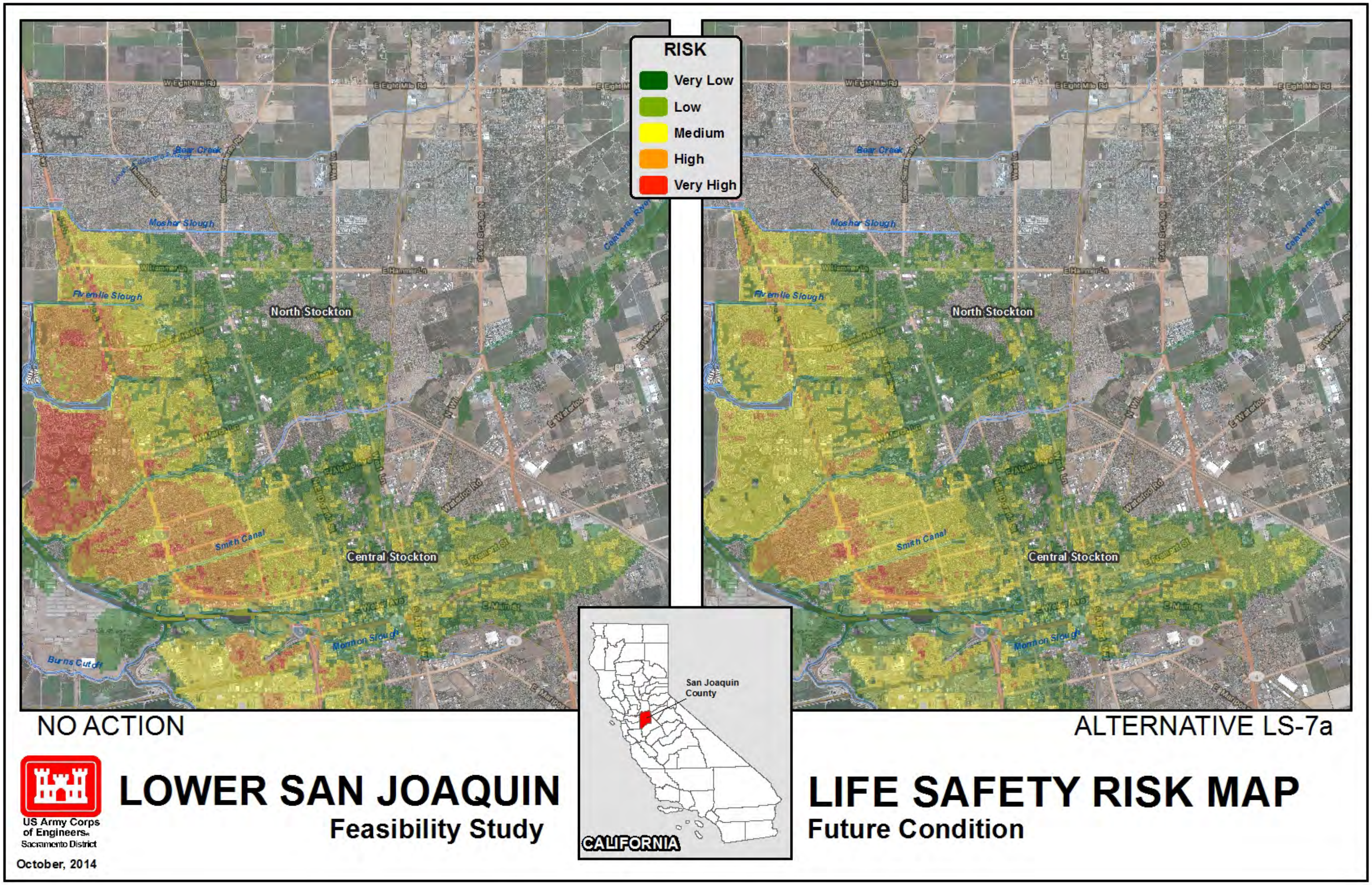


FIGURE 18: LIFE SAFETY RISK—CENTRAL STOCKTON—EXISTING CONDITION

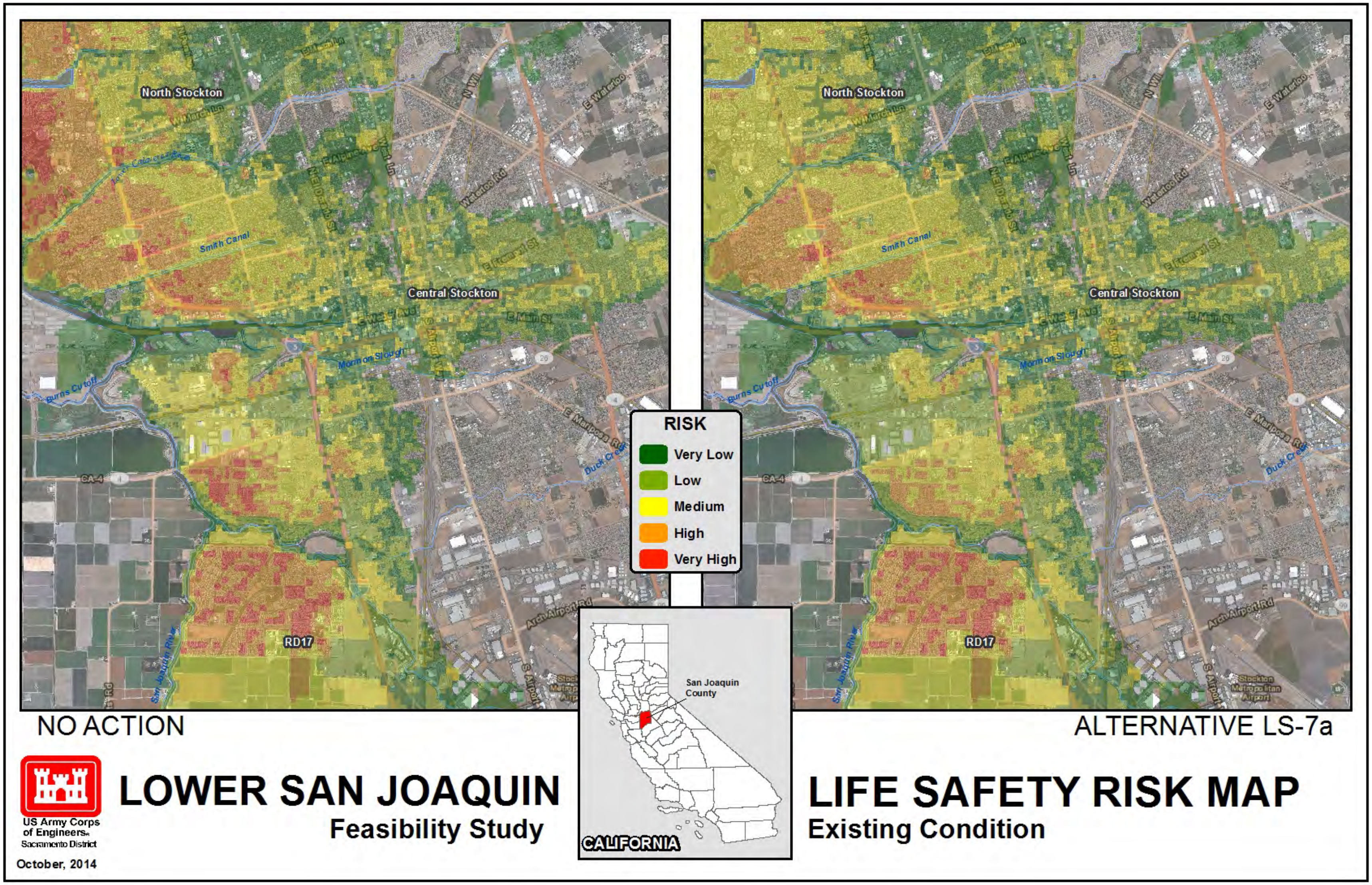


FIGURE 19: LIFE SAFETY RISK—CENTRAL STOCKTON—FUTURE CONDITION

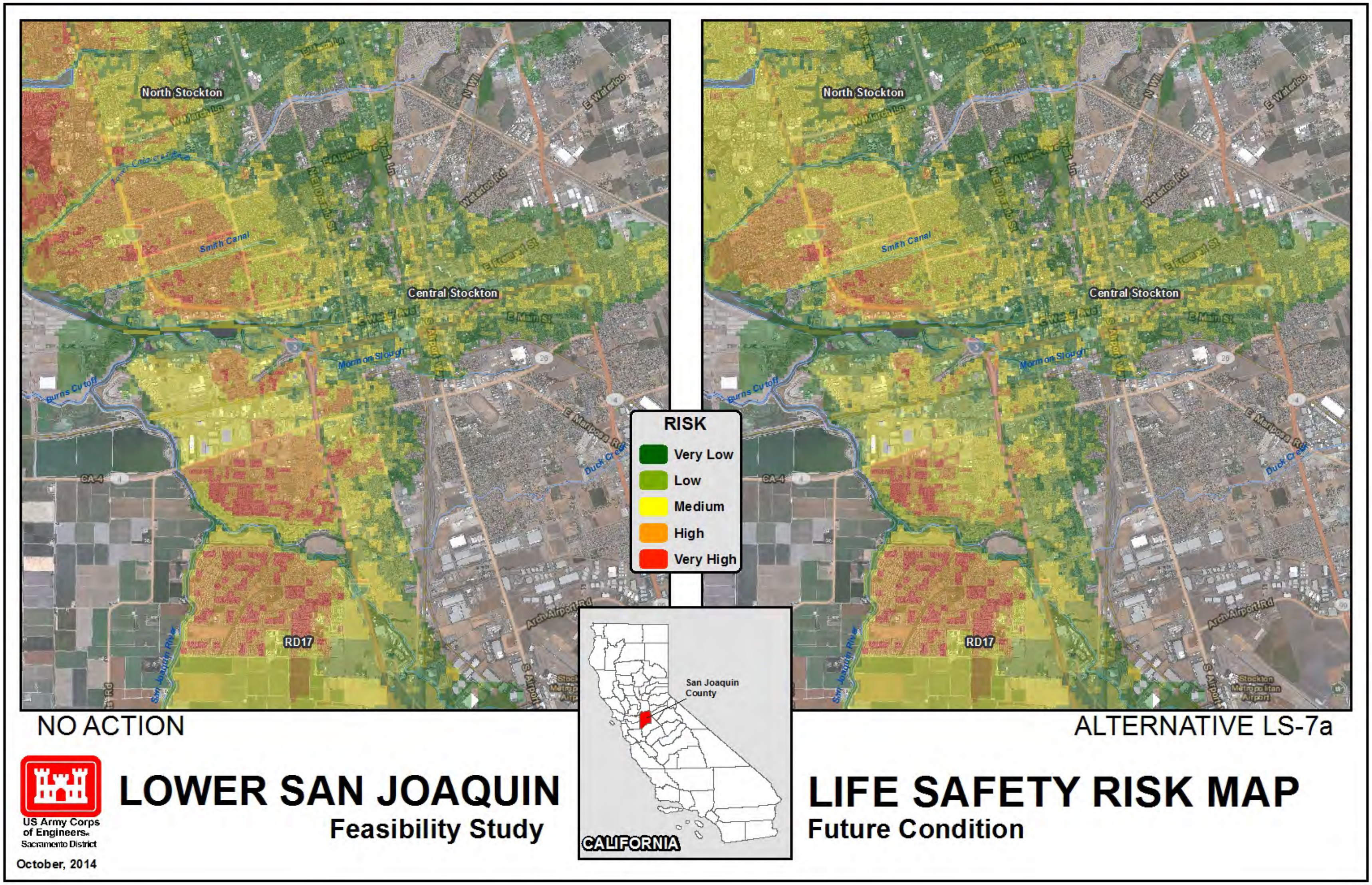


FIGURE 20: LIFE SAFETY RISK—RD17—EXISTING CONDITION

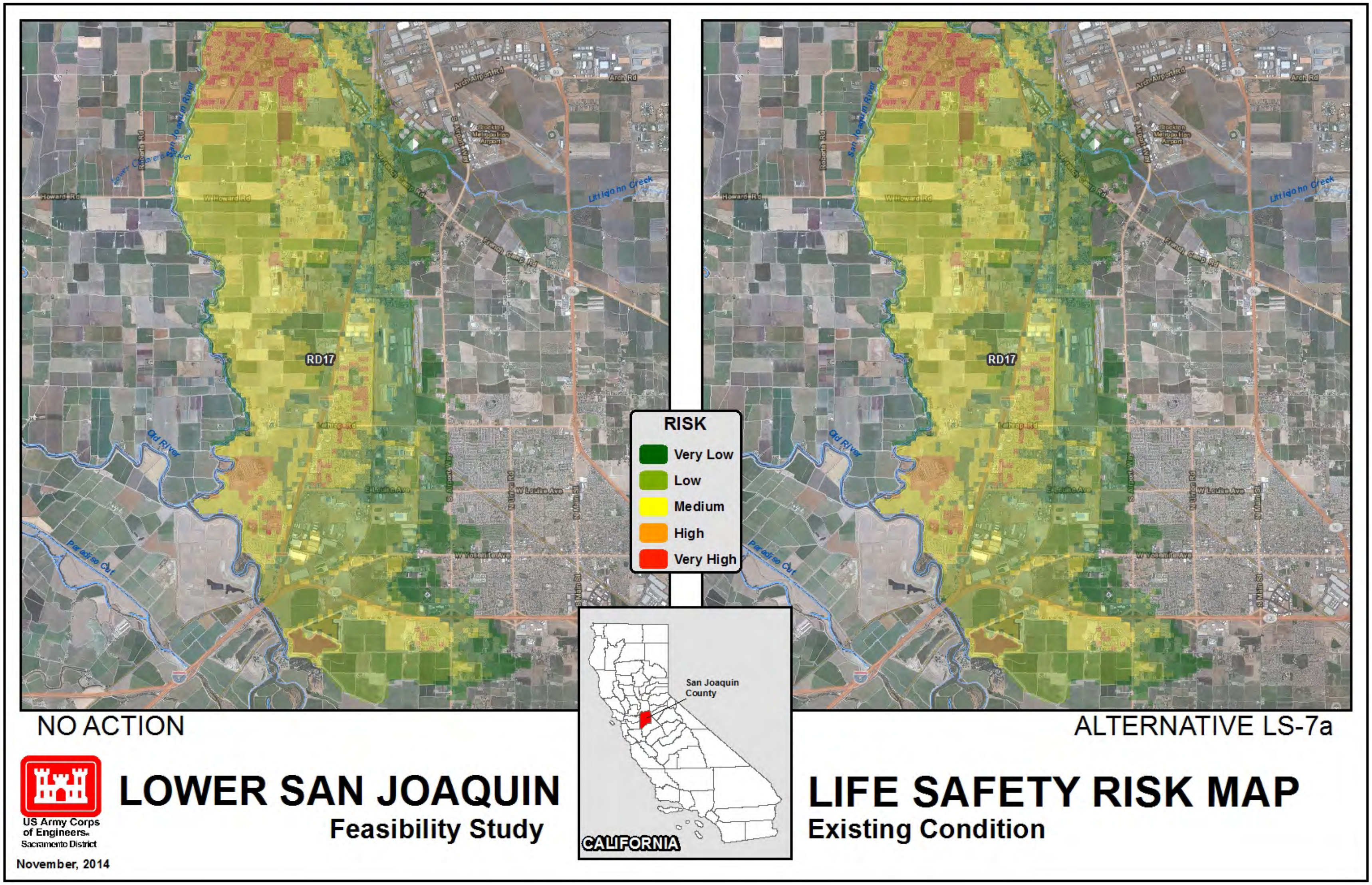
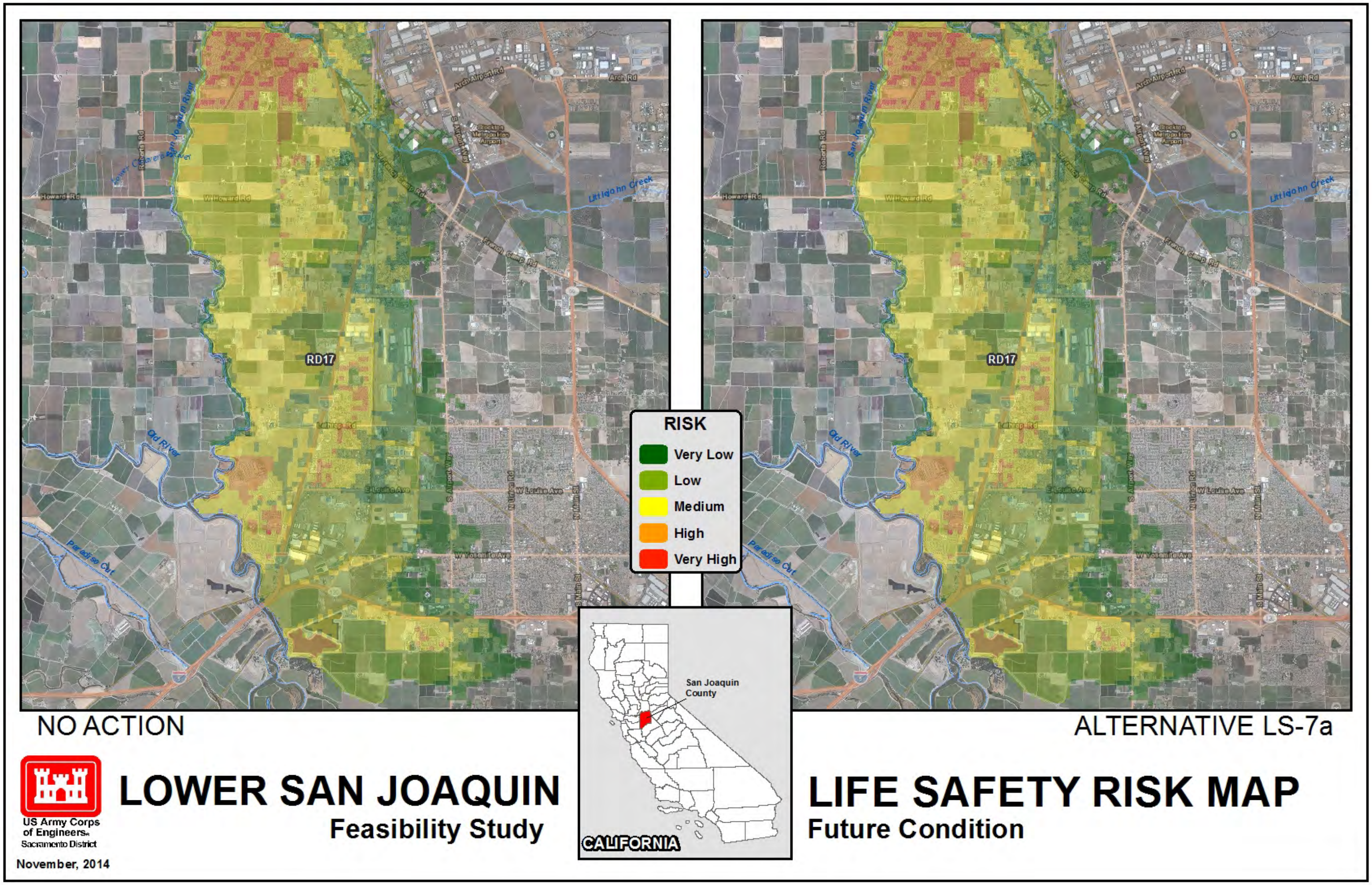


FIGURE 21: LIFE SAFETY RISK—RD17—FUTURE CONDITION



PART II — REGIONAL ECONOMIC DEVELOPMENT

PURPOSE AND METHODOLOGY

The U.S. Army Corps of Engineers (USACE) Planning Guidance Notebook (ER 1105-2-100) states that while the National Economic Development (NED) and Environmental Quality (EQ) accounts are required, display of the Regional Economic Development (RED) effects are discretionary. The Corps' NED procedures manual affirms that RED benefits are real and legitimate; however, the concern (from a Federal perspective) is that they are often offset by RED costs in other regions. Nevertheless, for the local community these benefits are important and can help them in making their preferred planning decisions.

Although the RED account is often examined in less detail than NED, it remains useful. For example, Hurricane Katrina caused a significant economic hardship to not just the immediate Gulf Coast but for entire counties, watersheds, and the state of Louisiana. Besides the devastating damage to homes (which are often captured by the NED account), hundreds of thousands of people lost their jobs, property values fell, and tourism and tax revenues declined significantly and were transferred to other parts of the U.S. In this example, the RED account can provide a better depiction of the overall impact to the region.

The distinction between NED and RED is a matter of perspective, not economics. A non-federal partner may consider the impacts at the state, regional, and local levels to be a true measure of a project's impact or benefit, whereas from the Corps' perspective, this may not constitute a national benefit. Gains in RED to one region may be partially or wholly offset by losses elsewhere in the nation. For example, if a Federal project enables a firm to leave one state to relocate to a newly-protected floodplain of another state, the increase in regional income for the project area may come at the expense of the former area's loss. In this case, there is no net increase in the value of the nation's output of goods and services and should be excluded from NED computations.

The following sections describe the impacts of the tentatively selected plan (TSP) a regional perspective. The impacts were evaluated using the Corps' certified RECONS software.

KEY RED CONCEPTS

Econometric analysis allows for the evaluation of a full range of economic impacts related to specific economic activities by calculating effects of the activities in a specific geographic area. These effects are:

- Direct effects, which consist of economic activity contained exclusively within the designated sector. This includes all expenditures made by the companies or organizations in the industry and all employees who work directly for them.
- Indirect effects, which define the creation of additional economic activity that results from linked business, suppliers of goods and services, and provisions of operating inputs.
- Induce effects, which measure the consumption expenditures of direct and indirect sector employees.

Input-output (I/O) models are characterized by their ability to evaluate the effects of industries on each other. Unlike most typical measures of economic activity that examine only the total output of an industry or the final consumption demand provided by a given output, I/O models provide a much more comprehensive view of the interrelated economic impacts. I/O analysis is based on the notion that there is a fundamental relationship between the volume of output of an industry and the volume of the various inputs used to produce that output. Industries are often grouped into production, distribution, transportation, and consumption categories. Additionally, the I/O model can be used to quantify the multiplier effect, which refers to the idea that an increase in spending can lead to an even greater increase in income and consumption, as monies circulate (or multiply) throughout the economy.

FLOOD RISK MANAGEMENT RED CONSIDERATIONS

There are particular effects for each type of project improvement as they relate to the RED account. The estimation of RED flood-related effects can be very complex. At a minimum, the RED analysis should include a qualitative description of the types of businesses at risk from flooding, particularly those that could have a significant adverse impact (output, employment, etc.) upon the community or regional economies if their operations should be disrupted by flooding and how this would be affected by the recommended project. The potential RED effects to flood risk management projects are summarized in Table 13below.

TABLE 13: POTENTIAL RED EFFECTS TO FLOOD RISK MANAGEMENT

RED FACTOR	POTENTIAL RED EFFECTS
Construction	Additional construction related activity and resulting spillovers to suppliers
Revenues	Increased local business revenues as a consequence of reduced flooding, particularly from catastrophic floods
Tax Revenues	Increased income and sales taxes from the direct project and spillover industries
Employment	Short-term increase in construction employment; with catastrophic floods, significant losses in local employment (apart from the debris and repair businesses, which may show temporary gains)
Population Distribution	Disadvantage groups may benefit from the creation of a flood-free zone
Increased Wealth	Potential increase in wealth for floodplain residents as less is spent on damaged property, repairs, etc.; potential increase in property values.

RECONS SOFTWARE

A variety of software programs are available to measure the RED impacts of a project. The Corps of Engineers' Institute for Water Resources (IWR) along with the Louis Berger Group has developed a regional economic impact modeling tool called Regional Economic System (RECONS) that computes estimates of regional and national job creation, retention, and other economic measures. The expenditures made by the USACE for various services and products generate economic activity that can be measured in jobs, income, sales, and gross regional product. The software automates calculations and generates estimates of economic measures associated with USACE's annual civil works program spending. RECONS was built by extracting multipliers and other economic measures from more than 1,500 regional economic models that were built specifically for USACE's project locations by the Minnesota IMPLAN Group. These multipliers were then imported into a database. The software ties various spending profiles to the matching industry sectors by location to produce economic impact estimates. The RECONS program is used to document the performance of direct investment spending of the USACE, and allows users to evaluate project and program expenditures associated with annual expenditures.

REGIONAL PROFILE

The economic impacts presented below show the Lower San Joaquin River Feasibility study area and the state of California's interrelated economic impacts resulting from an injection of flood risk management construction funds. For this assessment, the study area and the state of California were both used as the geographic designation to assess the overall impacts to the regional economy from constructing the

TSP. This places a frame around the economic impacts where the activity is internalized; leakages, which are payments made to imports or value added sectors that do not in turn re-spend the dollars within the area, are not included in the total impacts.

Table 14 summarizes the complex nature of the regional economy of the Stockton, CA Metropolitan Statistical Area (MSA), which has a population of approximately 750,000. There are approximately 288,000 people employed in the MSA who provide an output to the nation of nearly $40 billion annually.

TABLE 14: REGIONAL PROFILE – STOCKTON, CA MSA (DOLLAR VALUES IN $MILLIONS, OCTOBER 2014 PRICE LEVEL)

INDUSTRY	OUTPUT	LABOR INCOME	GRP	EMPLOYMENT
Accommodations and Food Service	$968	$328	$495	17,075
Administrative and Waste Management Services	$929	$482	$606	16,388
Agriculture, Forestry, Fishing and Hunting	$2,197	$614	$1,046	19,679
Arts, Entertainment, and Recreation	$227	$64	$104	2,872
Construction	$2,773	$1,151	$1,260	18,849
Education	$823	$609	$681	14,617
Finance, Insurance, Real Estate, Rental and Leasing	$3,348	$783	$2,222	18,799
Government	$3,041	$2,348	$2,665	34,727
Health Care and Social Assistance	$2,735	$1,503	$1,762	30,375
Imputed Rents	$3,022	$447	$1,904	17,145
Information	$1,787	$196	$387	3,219
Management of Companies and Enterprises	$303	$132	$176	1,492
Manufacturing	$9,093	$1,335	$2,155	21,820
Mining	$74	$23	$45	230
Professional, Scientific, and Technical Services	$1,215	$505	$682	9,394
Retail Trade	$2,362	$1,015	$1,616	32,939
Transportation and Warehousing	$2,033	$897	$1,268	16,116
Utilities	$1,082	$176	$408	1,235
Wholesale Trade	$1,871	$703	$1,208	11,425
Total	**$39,883**	**$13,311**	**$20,690**	**288,396**

INPUT COSTS

The RED analysis requires the adjustment of costs for two items: (1) interest during construction (IDC) and (2) purchases of land. Interest during construction is used in the NED analysis to estimate the opportunity cost of using money for one economic endeavor (*e.g.*, building a FRM project) instead of another (*e.g.*, building a bullet train); IDC is not actually expended within the region and therefore is not included in the RED analysis. Similarly, the purchase of land, not including administrative costs, is considered a transfer payment from one party to another and therefore is also not included in the RED analysis. The total remaining costs of the TSP is $517,801,000.

Table 15 shows the regional expenditures expected over the 11 year construction period. The expected annual expenditure is roughly $47 million. Local capture rates are provided by RECONS and show where the output from expenditures is realized.

TABLE 15: TSP INPUTS ASSUMPTIONS—STOCKTON, CA MSA

CATEGORY	SPENDING	SPENDING AMOUNT	LOCAL PERCENTAGE CAPTURE		
			LOCAL	STATE	NATIONAL
Aggregate Materials	8.3%	$43,076,775	74%	77%	97%
Other Materials	1.1%	$5,916,871	100%	100%	100%
Equipment	29.2%	$150,993,640	82%	99%	100%
Construction Labor	46.1%	$238,602,790	100%	100%	100%
Explosives Materials	0.1%	$439,572	8%	47%	86%
Cement Materials	0.3%	$1,794,919	7%	73%	92%
Metals and Steel Materials	1.2%	$6,263,901	18%	56%	90%
Machinery Materials	0.5%	$2,710,694	13%	46%	79%
Electrical Materials	0.6%	$3,150,266	19%	44%	80%
Lumber Materials	0.1%	$439,572	24%	56%	90%
Cultural Resources Protection Activities	2.8%	$14,592,000	40%	99%	99%
Fish Hatcheries, Wildlife Facilities, and Sanctuaries	9.6%	$49,820,000	100%	100%	100%
Total	**100%**	**$517,801,000**	**88.5%**	**96.4%**	**99.3%**

RECONS OUTPUT

The expenditures made by the Corps of Engineers for various services and products are expected to generate additional economic activity, which can be measured in jobs, income, sales, and GRP. These impacts are summarized in Tables 16 through 18 (economic activity on regional, state, and national basis).

TABLE 16: SUMMARY OF ECONOMIC IMPACTS

		REGIONAL	STATE	NATIONAL
Direct Impact	Output	$457,920,499	$499,184,217	$513,950,423
	Jobs	$6,152	$6,318	$6,390
	Labor Income	$318,105,873	$332,625,180	$339,076,586
	GRP	$363,579,956	$386,604,753	$394,679,283
Total Impact	Output	$802,934,646	$1,016,660,600	$1,371,534,378
	Jobs	$8,624	$9,761	$11,675
	Labor Income	$433,463,030	$510,646,814	$624,475,268
	GRP	$571,957,806	$694,794,105	$888,588,856

TABLE 17: REGIONAL ECONOMIC IMPACTS

Industry Sector	Sales	Jobs	Labor Income	GRP
Direct Effects				
Wholesale trade businesses	$1,483,655	8	$560,373	$1,118,727
Transport by rail	$1,151,469	3	$353,202	$610,689
Transport by water	$327,013	1	$83,163	$158,500
Transport by truck	$14,937,266	107	$7,252,626	$8,543,227
Construction of other new nonresidential structures	$5,916,871	33	$2,487,517	$3,096,192
Commercial and industrial machinery and equipment rental and leasing	$123,305,157	375	$34,237,323	$69,882,421
Labor	$238,602,790	5,198	$238,602,790	$238,602,790
All other chemical product and preparation manufacturing	$4,559	0	$373	$726
Cement manufacturing	$0	0	$0	$0
Steel product manufacturing from purchased steel	$405,949	1	$84,742	$100,655
Other industrial machinery manufacturing	$51,246	0	$16,483	$19,256
Mining and quarrying sand, gravel, clay, and ceramic and refractory minerals	$15,674,892	72	$9,334,131	$10,444,986
Switchgear and switchboard apparatus manufacturing	$233,406	1	$52,055	$107,983
Retail Stores - Furniture and home furnishings	$22,508	0	$8,307	$14,383
Retail Stores - Electronics and appliances	$69,323	1	$22,252	$37,112
Retail Stores - Building material and garden supply	$3,772	0	$1,767	$2,593
Transport by air	$1,473	0	$25	$450
Engineered wood member and truss manufacturing	$51,089	0	$16,814	$21,259
Scientific research and development services	$5,882,202	42	$2,446,999	$2,450,038
Maintenance and repair construction of nonresidential structures	$49,795,863	311	$22,544,933	$28,367,970
Total Direct Effects	$429,375,535	5,776	$298,883,003	$342,335,820
Secondary Effects	$322,890,493	2,312	$107,993,154	$194,987,838
Total Effects	**$752,266,028**	**8,089**	**$406,876,156**	**$537,323,658**

TABLE 18: STATE ECONOMIC IMPACTS

Industry Sector	Sales	Jobs	Labor Income	GRP
Direct Effects				
Wholesale trade businesses	$2,582,269	15	$1,043,716	$1,974,155
Transport by rail	$1,151,469	3	$353,202	$610,689
Transport by water	$340,031	1	$86,475	$164,809
Transport by truck	$14,937,266	107	$7,252,626	$8,543,227
Construction of other new nonresidential structures	$5,916,871	33	$2,487,517	$3,096,192
Commercial and industrial machinery and equipment rental and leasing	$149,354,072	456	$41,470,151	$84,645,479
Labor	$238,602,790	5,198	$238,602,790	$238,602,790
All other chemical product and preparation manufacturing	$161,566	0	$25,076	$36,601
Cement manufacturing	$1,121,507	2	$251,323	$510,405
Steel product manufacturing from purchased steel	$2,455,936	5	$512,677	$608,952
Other industrial machinery manufacturing	$742,013	3	$238,661	$278,817
Mining and quarrying sand, gravel, clay, and ceramic and refractory minerals	$16,536,071	78	$9,846,949	$11,018,834
Switchgear and switchboard apparatus manufacturing	$767,908	2	$172,330	$356,004
Retail Stores - Furniture and home furnishings	$32,899	0	$12,598	$21,283
Retail Stores - Electronics and appliances	$108,039	1	$41,858	$62,750
Retail Stores - Building material and garden supply	$3,772	0	$1,767	$2,593
Transport by air	$11,337	0	$2,803	$5,310
Engineered wood member and truss manufacturing	$162,827	1	$53,587	$67,755
Scientific research and development services	$14,399,714	102	$7,624,143	$7,630,138
Maintenance and repair construction of nonresidential structures	$49,795,863	311	$22,544,933	$28,367,970
Total Direct Effects	$405,833,177	5,551	$283,929,227	$328,982,424
Secondary Effects	$410,515,217	2,787	$141,300,173	$244,905,267
Total Effects	**$816,348,394**	**8,337**	**$425,229,400**	**$573,887,691**

TABLE 19: NATIONAL ECONOMIC IMPACTS

Industry Sector	Sales	Jobs	Labor Income	GRP
Direct Effects				
Wholesale trade businesses	$2,617,282	15	$1,059,120	$2,001,417
Transport by rail	$1,359,488	4	$419,610	$723,076
Transport by water	$492,478	1	$125,244	$238,699
Transport by truck	$15,727,307	113	$7,636,223	$8,995,083
Construction of other new nonresidential structures	$5,916,871	33	$2,487,517	$3,096,192
Commercial and industrial machinery and equipment rental and leasing	$150,773,053	462	$41,864,150	$85,449,678
Labor	$238,602,790	5,198	$238,602,790	$238,602,790
All other chemical product and preparation manufacturing	$330,782	1	$55,227	$80,316
Cement manufacturing	$1,464,137	3	$328,104	$666,338
Steel product manufacturing from purchased steel	$4,537,693	9	$947,244	$1,125,126
Other industrial machinery manufacturing	$1,633,695	7	$525,461	$613,873
Mining and quarrying sand, gravel, clay, and ceramic and refractory minerals	$23,911,121	124	$14,238,665	$15,933,209
Switchgear and switchboard apparatus manufacturing	$1,899,499	5	$446,617	$921,609
Retail Stores - Furniture and home furnishings	$33,882	0	$13,004	$21,936
Retail Stores - Electronics and appliances	$108,313	1	$41,997	$62,931
Retail Stores - Building material and garden supply	$3,800	0	$1,780	$2,612
Transport by air	$15,114	0	$3,876	$7,187
Engineered wood member and truss manufacturing	$310,936	2	$102,330	$129,386
Scientific research and development services	$14,406,385	102	$7,628,198	$7,634,196
Maintenance and repair construction of nonresidential structures	$49,805,796	0	$0	$0
Total Direct Effects	$513,950,423	6,079	$316,527,156	$366,305,654
Secondary Effects	$857,583,955	4,755	$256,080,670	$443,786,015
Total Effects	**$1,371,534,378**	**10,834**	**$572,607,826**	**$810,091,669**

GEOTECHNICAL ENGINEERING EVALUATION
LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY
STOCKTON, CALIFORNIA

Prepared By:

U.S. Army Corps of Engineers

Sacramento District

Geotechnical Branch

February 2015

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ENCLOSURES

E1: Geomorphology Maps

E2: Calculation Package

E3: Risk and Uncertainty Analyses

E4: Seismic and Liquefaction Analyses

E5: Template Options for Assigned Mitigation Measures

E6: Meeting Minutes for Expert Elicitation

ABBREVIATIONS

ASTM	American Society of Testing and Materials
BTA	blanket theory analysis
bgs	below ground surface
c	cohesion
CB	cement bentonite
cfs	cubic feet per second
CGS	California Geological Survey
cm	centimeters
CPT	cone penetrometer test
CR	Calaveras River
CW	cutoff wall
CVFPB	Central Valley Flood Protection Board
DBSA	Delta Brookside Study Area

DLVSA	Delta Lincoln Village Study Area
DMM	deep mix method
DSM	deep soil mixing
DWR	Department of Water Resources
EM	Engineer Manual
ER	Engineer Regulation
ETL	Engineer Technical Letter
FCS	French Camp Slough
FOS	factor(s) of safety
FOSM	First Order Second Moment
ft	foot/feet
ft/s	feet per second
GDR	Geotechnical Data Report
GER	Geotechnical Engineering Report
GMS	Groundwater Modeling System
H:V	horizontal to vertical ratio
HQ	Headquarters U.S. Army Corps of Engineers
IBC	International Building Code
IWM	in-stream woody material
k	coefficient of permeability
Ka	kiloannum – one thousand years
k_H	horizontal hydraulic conductivity under fully saturated conditions
k_H/k_V	ratio between vertical and horizontal conductivities; anisotropic ratio
k_V	vertical hydraulic conductivity under fully saturated conditions
LiDAR	Light Detection and Ranging
LM	Levee Mile
LSJRFS	Lower San Joaquin River Feasibility Study
LSJR	Lower San Joaquin River
Ma	million years
MCE	Maximum Credible Earthquake
MSWL	Mean Summer Water Level
M_W	Moment Magnitude
NAD83	North American Datum of 1983
NAVD88	North American Vertical Datum of 1988
NCEER	National Center for Earthquake Engineering Research
NGA	Next Generation Attenuation
NGVD29	National Geodetic Vertical Datum of 1929
NLD	National Levee Database
NULE	Nonurban Levee Evaluations
PCF	per cubic foot
PDT	Project Delivery Team
PED	pre-construction engineering and design
PGA	peak ground acceleration
Pr(f)	probability of failure
Pr(U)	probability of poor performance
PSHA	Probabilistic Seismic Hazard Analysis

P1GDR	Phase 1 Geotechnical Data Report
P1GER	Phase 1 Geotechnical Engineering Report
PI	Periodic Inspection
RD	Reclamation District
RM	River Mile
SAFCA	Sacramento Area Flood Control Agency
SJAFCA	San Joaquin Area Flood Control Agency
SB	soil-bentonite
SCB	soil cement bentonite
SDC	Stockton Diverting Canal
SGDR	Supplemental Geotechnical Data Report
SOP	Standard Operating Procedure
SPT	Standard Penetration Test
SRBPP	Sacramento River Bank Protection Project
TEC	Topographic Engineering Center
TM	Technical Memorandum
ULE	Urban Levee Evaluations
ULDC	Urban Levee Design Criteria
USACE	U.S. Army Corps of Engineers
USGS	United States Geological Society
V_S30	velocity of the upper 30 meters
VVR	vegetation variance request
WRDA	Water Resources Development Act
WSE	water surface elevation

1. INTRODUCTION

This report is the geotechnical appendix to the Lower San Joaquin River Feasibility Study (LSJRFS). The LSJRFS area includes portions of the Lower San Joaquin River (LSJR), French Camp Slough (FCS), Stockton Diverting Canal (SDC), Calaveras River (CR), the Delta Brookside Study Area (DBSA), and the Delta Lincoln Village Study Area (DLVSA). The flood plain includes most of the developed portions of North Stockton, Central Stockton, and South Stockton, including areas of Lathrop and Manteca. The San Joaquin watershed drains approximately 31,000 square miles of land, covering an area nearly the expanse of South Carolina, and a population of approximately 4,000,000.

1.1 PURPOSE AND SCOPE

This Report presents the results of geotechnical analyses and feasibility level geotechnical recommendations to address levee height, geometry, erosion, access, vegetation, seepage, and slope stability deficiencies within the LSJRFS area. Due to the evolving Planning process and the implementation of the 3x3x3 paradigm, this Report was prepared using existing information provided by the Department of Water Resources (DWR), San Joaquin Area Flood Control Agency (SJAFCA), URS Corporation, and Kleinfelder. For this geotechnical engineering evaluation of the LSJRFS area, the following tasks were performed and are summarized in this report:

- review currently available geology, geomorphology, and geotechnical information
- review past performance and flood control system construction history/improvements
- identification of levee performance deficiencies through geotechnical analysis and engineering judgment
- probabilistic geotechnical analysis and development of levee performance curves
- seismic study of existing levees
- development of geotechnical conclusions and recommendations

1.2 PROJECT DESCRIPTION

The Lower San Joaquin River and Tributaries Project was first authorized by the Flood Control Act of 1944. The Lower San Joaquin River Feasibility Study was authorized by the Water Resources Development Act (WRDA) of 1986 following the feasibility studies authorized by the Flood Control Act of 1962 and following appropriations in 2004. The Cost-Share agreement signed in February 2009 initiated the multi-year feasibility study of the LSJR between the Corps, Central Valley Flood Protection Board (CVFPB) represented by the State of California Department of Water Resources, SJAFCA, and its partners.

The LSJRFS area, shown in Figure 1-1, has been divided into three basins: North Stockton, Central Stockton, and South Stockton.

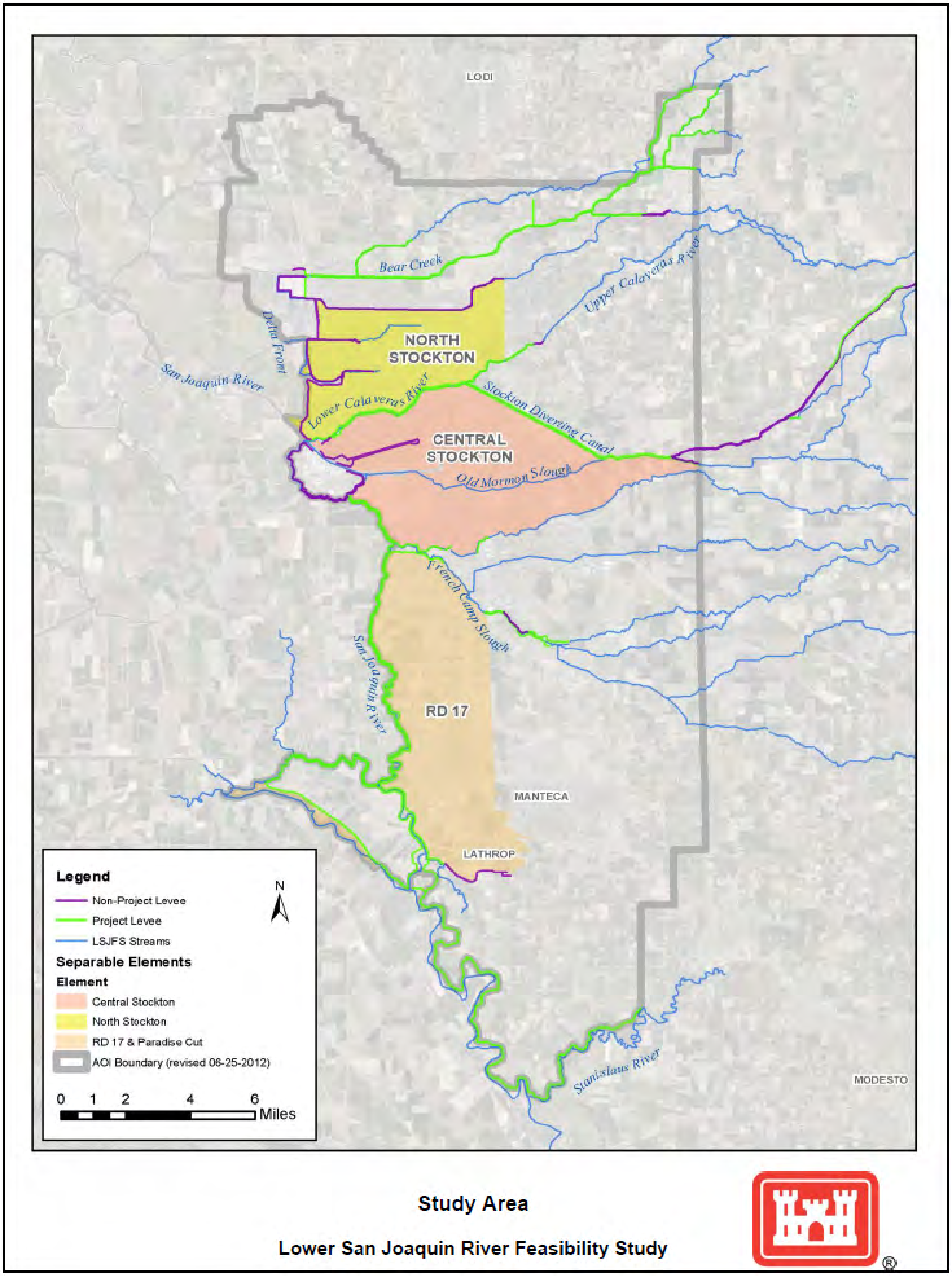


Figure 1-1: Lower San Joaquin Project Study Area

These three areas include the following stretches of levee, which are covered by this report:

- approximately 15 miles of levee along the east bank of the Lower San Joaquin River, Reclamation District 17 (RD-17), immediately downstream of Weatherbee Lake, north to the confluence of French Camp Slough
- approximately 2 miles of levee along the north (RD-404) and south banks (RD-17) of French Camp Slough (total 4 miles), immediately downstream of I-5, west to the confluence of the Lower San Joaquin River
- approximately 5 miles of levee along the west bank of the Stockton Diverting Canal (SJAFCA), immediately downstream of the confluence of Mormon Slough, northwest downstream to the confluence of Calaveras River
- approximately 6 miles of levee along the north (SJAFCA, RD-2074) and south (SJAFCA, RD-1614) banks of the Calaveras River (total 12 miles), immediately downstream of the Stockton Diverting Canal, southwest downstream to the confluence of the Lower San Joaquin River
- approximately 3.5 miles of levee west and north (RD-2074) of the Brookside Community along the Lower San Joaquin River and Fourteen Mile Slough, respectively
- approximately 2.5 miles of levee west and south (RD-1608) of the Lincoln Village Community along Fourteen Mile Slough

The extents of the areas listed above were developed further by the Project Delivery Team (PDT) over the duration of the study (for example, in identifying with project alternatives).

1.3 REACH IDENTIFICATION

Reach identification (i.e., LR-1, FR-1, etc.) is the primary method used to describe the index point locations; however; for the purposes of the feasibility planning process, these reaches were further subdivided based on common properties, such as geographic features. In general, as stated above, this report presents information either by basin or reach; however, in some cases the report structure deviates from basin or reach-based organization. For instance, geology and geomorphology, construction history, and past performance are better related to channel features than basin related reaches. Therefore, for those topics, the information has been presented in the following groups: North Stockton, Central Stockton, South Stockton, RD-17, RD-404, French Camp Slough, Stockton Diverting Canal, Calaveras River, Tenmile Slough, and Fourteen-Mile Slough.

2. SITE CONDITIONS

2.1 SOURCES OF DATA

The subsurface conditions and material properties of the levee embankments and foundation soils have been characterized by several studies in the past. These studies have been prepared as part of reconnaissance and feasibility efforts by the USACE, DWR, SAFCA, and SJAFCA among others. Following the 1986 flood event and the severe flooding of 1997 that resulted in dozens of levee failures throughout the San Joaquin River Basin, several studies were initiated which generated geotechnical data including:

- RD-17 – Phase 1 Geotechnical Engineering Report (P1GER), December 2007, Phase 1 Geotechnical Data Report (P1GDR), September 2008; Supplemental Geotechnical Data Report (SGDR), December 2010; all reports prepared by URS for DWR
- RD-404 – Supplemental Geotechnical Data Report (SGDR), April 2011; prepared by URS for DWR
- Stockton Diverting Canal/Calaveras River – Phase 1 Geotechnical Data Report (P1GDR), July 2008; Phase 1 Geotechnical Engineering Report (P1GER), July 2011, Draft Supplemental Geotechnical Data Report (SGDR), March 2013; all reports prepared by URS for DWR
- Delta Brookside Study Area – Draft Geotechnical Data Report (GDR), August 2012; prepared by Kleinfelder for DWR
- Delta Lincoln Village Study Area – Draft Geotechnical Data Report (GDR), June 2012; prepared by Kleinfelder for DWR
- Geotechnical Assessment Report (GAR) South NULE Study Area, Volumes 1 through 4, May 2011; prepared by Kleinfelder for DWR

These studies consisted of feasibility geotechnical data and design reports that presented the results of engineering studies and investigations prior to plans and specifications for remedial construction of levees within the LSJ Basin.

The available geotechnical data from the above mentioned sources included subsurface geotechnical borings and Cone Penetrometer Tests (CPT) performed along the levee crest, waterside toe, landside toe, and within 500-feet of the landside toe; other data included geology and geomorphology studies, and geophysical surveys. The levee geometry was based on the existing data in the National Levee Database (NLD) supplemented by recent Light Detection and Ranging (LiDAR) survey and bathymetric survey provided by the DWR as part of the Urban Levee Evaluations (ULE) program.

Elevation references in this report are in feet and are based on the North American Vertical Datum of 1988 (NAVD88) unless otherwise noted. Conversion factors ranging between +2.26 to +2.42 were applied by the organizations mentioned above to convert Geodetic Vertical Datum of 1929 (NGVD29) elevations to NAVD88. All horizontal references in this report are in feet and are based on the California State Plane, Zone III, North American Datum of 1983 (NAD83).

2.2 GEOLOGY, GEOMORPHOLOGY, AND SEISMICITY

2.2.1 Geologic Setting

This section will summarize the geologic and geomorphic assessment developed by USACE, Fugro William Lettis & Associates (FWLA), and Kleinfelder for the LSJRFS area. The complete assessment report(s) are included as Appendix O in each report listed in Section 2.1; except for the GAR South NULE report.

This area of California was part of the early Cretaceous to Paleocene convergent tectonic margin and associated Sierran magmatism. The basement rock in this area consists of Sierran granite or granitoid rocks on the eastern side of the basin and Coast Range ophiolite to the west. Age-dated profiles suggest a migration of plutonism from west to east with the oldest rocks occurring on the margin of the San Joaquin Valley and the youngest appearing on the eastern flank of the Sierra Nevada (Hosford Scheirer and Magoon, 2008). With the end of plutonism, came the beginning of the flat slab subduction mega-sequence about 5 Ma (million years) subsequent. During the late Cretaceous through the beginning of the Paleocene, the Panoche and Moreno formations indicate dominantly marine conditions with periods of scattered and non-aerially extensive terrestrial deposition. The geologic record is incomplete from the late Paleocene to the early Eocene in the Northern Sub-province during which time the Lodo (marine) and Yokut (near shore fluvial deltaic) formations were deposited. The Yokut deposition was followed (conformably) in the north sub-province by the Domengine sand (shallow marine transgressive). Deposition of the Kreyenhagen formation (marine) began concurrently with the Domengine formation and continued long after into the middle Eocene (37 Ma). The geologic record is incomplete in the north sub-province until the deposition of the late Oligocene to early Miocene Zilch formation (terrestrial - period of worldwide regression) which lies unconformably above the Kreyenhagen. The Zilch is unconformably overlain by the upper Miocene Santa Margarita Sandstone (shallow marine clastic). The remaining sequence of sediments are generally Pliocene and Pleistocene terrestrial deposits derived from the uplift of the Sierra Nevada and Coast Range. These younger sediments include the Pliocene Mehrten formation (terrestrial fluvial - derived from volcanic sources), and the Pliocene China Hat formation (terrestrial fluvial – Sierran origin). These are overlain by the Pleistocene Merced, Turlock Lake, Riverbank, and Modesto formations; all of which thin to the west of the basin and interfinger with sediments derived from the coast range to the west. These are in turn incised by Holocene alluvial channels and covered by Holocene fan deposits.

The RD-17 basin follows the Lower San Joaquin River as it flows into the San Joaquin Delta. The LSJR is near a contact of young, fluvial deposits within the Delta (in the west) and a gently west sloping alluvial fan formed by the Stanislaus and Calaveras Rivers (in the east). Upstream of the RD-17 study area, the LSJR splits into multiple channels including Tom Paine Slough and Paradise Cut. All major channels are characterized by several overflow and secondary channels that typically diverge to the north and west from the LSJR. Before agricultural development was introduced into these areas, the channels flowed into and through tidal marshes. Tidal effects, sea-level changes, and subsidence within the Delta have influenced the events along the LSJR over the past thousands of years.

The RD-404 study area occupies a lowland area along the east bank of the Lower San Joaquin

River just north of French Camp Slough headed north-west to the Port of Stockton. This area is situated between two large Pleistocene alluvial fans that originated from the Sierra Nevada Range. Lone Tree and Littlejohns Creek fill in the low lying areas of these two large fans with their own alluvial fan sediment and then drain to French Camp Slough traversing the southern boundary of the study area.

The Stockton Diverting Canal and Calaveras River study areas are similar in setting to the other areas in this study. They are situated within two large alluvial fans underlain by materials that originated from the Sierra Nevada Range. The Calaveras River flows along the lateral margin of the Calaveras alluvial fan. The western extents of the study area, west of Highway I-5, are within the eastern part of a tidally influenced Delta. Elevations in this area are at or below sea level. This area at or below sea level is a transition zone of low energy where alluvial materials and organic rich sediment string together (Marchand and Atwater, 1979; Cosby and Carpenter, 1937).

The Delta Brookside study area shares the same geologic setting as the Lincoln Village study area. The majority of the entire study area is underlain by the Delta geomorphic domain except for the southeast portion of the Lincoln Village study area that trends east beyond Highway I-5 onto alluvial fans underlain by materials that originated from the Sierra Nevada Range. The Delta geomorphic domain consists of saucer-shaped islands separated by fluvial channels and tidal sloughs that were connected prior to dredging and levee construction. The western extents of the study areas, including Buckley Cove and Fourteen Mile Slough, are part of the tidally influenced Delta that, prior to reclamation, was part of the inundated Delta characterized by organic-rich peat and peaty mud sediments (Atwater, 1982).

2.2.2 Geomorphology

For a summary description of area geomorphology, the LSJRFS area was broken up into the following areas: Lower San Joaquin River RD-17, RD 404/French Camp Slough, Stockton Diverting Canal, Calaveras River, and North Stockton Delta Brookside and Lincoln Village. Site-specific geomorphology maps produced by FWLA and Kleinfelder are included in Enclosure 1.

Historical deposits along the RD-17 basin overlay Holocene alluvial deposits. The historical channel deposits mapped east of the RD-17 levees suggest a younger sandy material overlain with the RD-17 levee prism (Figure 2-1). Detailed maps completed by Atwater (1980, 1982) showed the deposits of the northward flowing San Joaquin River system are primarily Holocene in age with more recent data suggesting less than 7,000 years of age (Malamoud-Roam et al., 2007). The San Joaquin River deposits were defined by Atwater (1982) as undivided Holocene alluvial floodplain deposits with isolated areas of Holocene basin deposits. These shallow deposits are underlain by a much thicker sequence of alluvial deposits from the Stanislaus River drainage originating from the Sierra Nevada Range and eolian deposits from the Central Valley. The Pleistocene deposits in the east are primarily silts and clayey materials with lenses of gravel all grouped into the Modesto Formation; the age of these deposits have been estimated by Atwater (1980) to be between 14,000 and 40,000 years old. The RD-17 area contains minor historic debris resulting from hydraulic mining. A surficial geologic map created by FWLA for the RD-17 area is included as part of Enclosure 1.

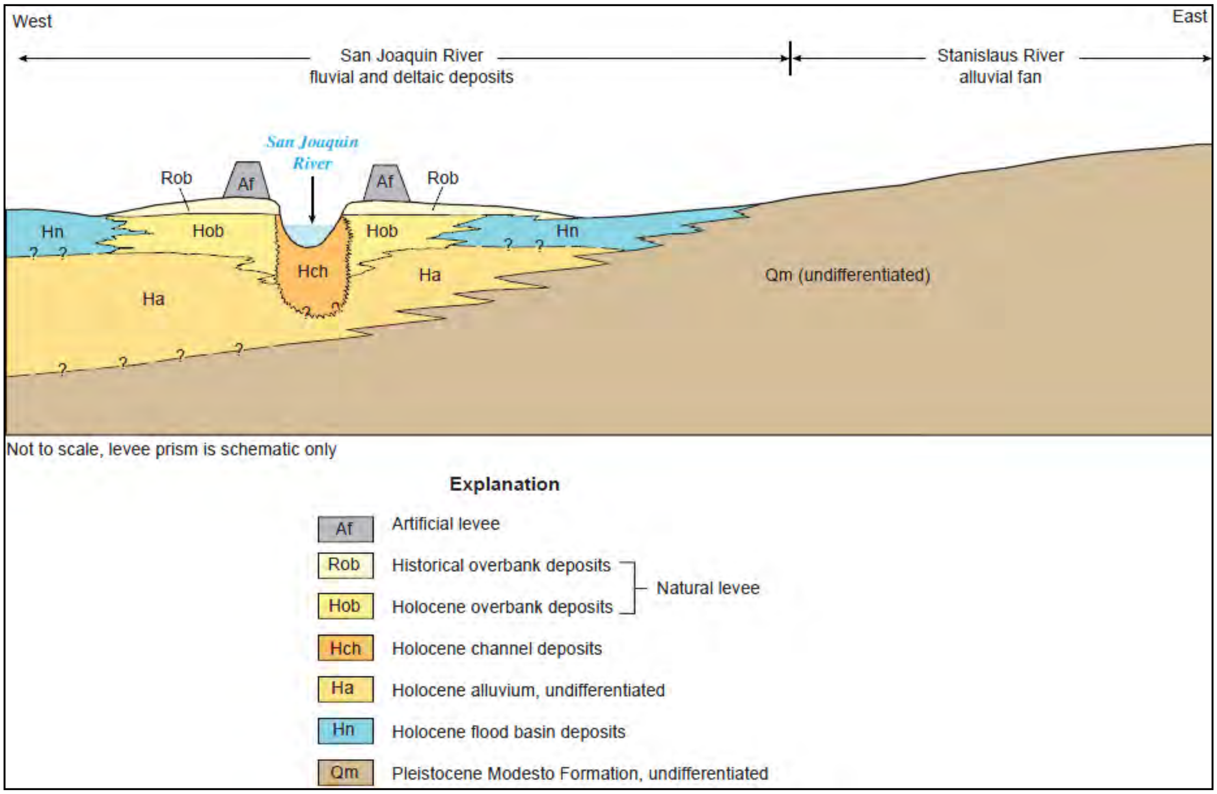


Figure 2-1: Geologic Units of Lower San Joaquin River RD-17

A surficial geologic map of RD-404 shows historical deposits along the Lower San Joaquin River suggesting a younger sandy material overlain with the levee prism along this section of RD-404. The map also shows a blend of silty, clayey, organic material overlain with the RD-404 levee prism along French Camp Slough (Figure 2-2). The oldest geologic unit in the study area is the late Pleistocene Modesto Formation that underlies a low gradient alluvial fan towards the eastern portion of the study area. It consists of unconsolidated to semi-consolidated sands, silts, and clayey materials and is part of a developed clay-rich duripan horizon. This clay-rich horizon likely forms extensive lateral zones of impermeable material in the shallow subsurface. The thickness and age of the Modesto Formation varies; however, the lower member is exposed in this study area and ranges from 29 to 42 Ka (Marchand and Allwardt, 1981). A surficial geologic map of this area is included as part of Enclosure 1.

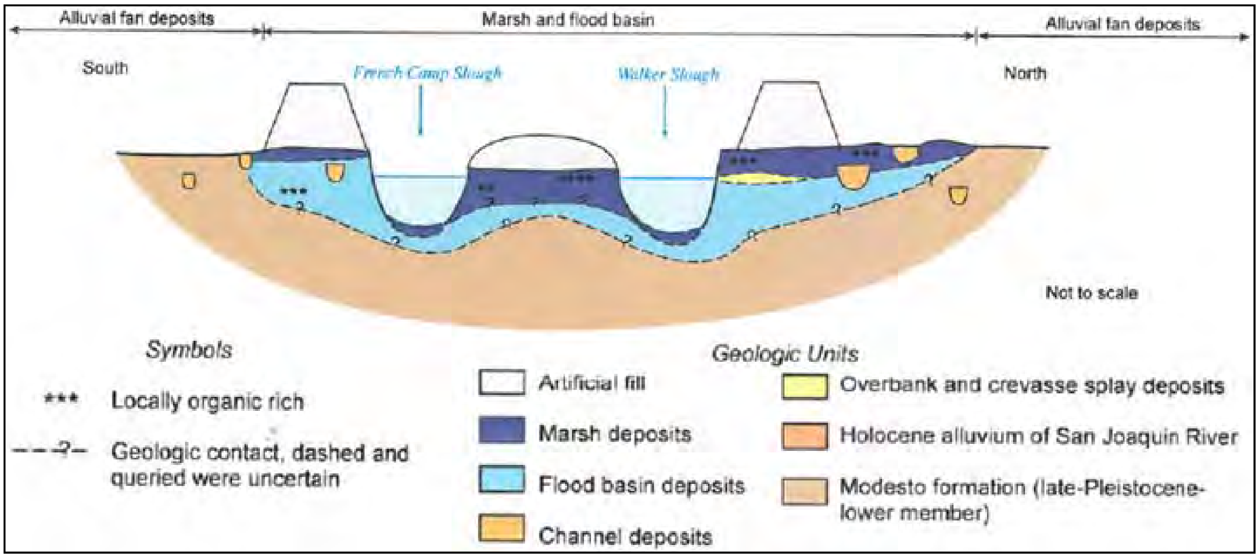


Figure 2-2: Geologic Units of RD-404/French Camp Slough

A surficial geologic map of the Stockton Diverting Canal and Calaveras River (Enclosure 1) show that SDC and a majority of the Calaveras River (from SDC to just east of Highway I-5) are within the domain of an alluvial fan. The area west of Highway I-5 resides within an intertidal domain. The SDC is a linear manmade channel that carries flows from Mormon Slough across the alluvial fan to the Calaveras River. The channel is filled with fine-grained silts and clays and crosses 15 channels that once flowed down the alluvial fan. The Modesto Formation underlies the levees along the canal to a depth of approximately 10 to 25 feet below the levee base; material at these depths consist of very stiff to hard silty clays to sandy clays, and silty sands. Underlying this material is a denser well consolidated Riverbank Formation (Figure 2-3).

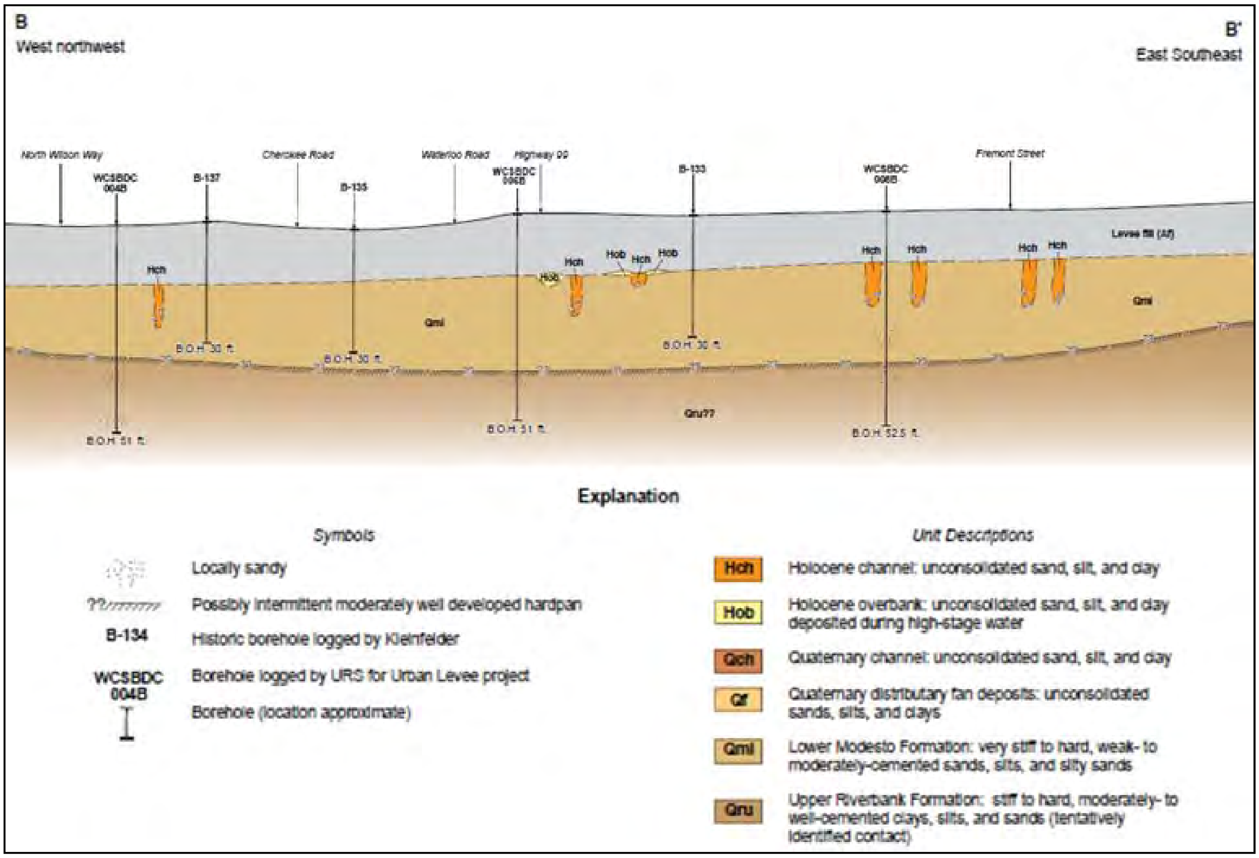


Figure 2-3: Geologic Units of Stockton Diverting Canal

The surficial geologic map shown for SDC (Enclosure 1) shows a portion of the Calaveras River within the alluvial fan. The Calaveras River ranges from 28-feet above sea level in the upstream portion (east) to less than 5 feet above sea level at the downstream end (west southwest). This portion of the Calaveras River crosses 8 channels that once flowed down the alluvial fan. Thin layers of unconsolidated Holocene sands and silts overlay more consolidated deposits of Modesto Formation. Additional deposits of Pleistocene, Holocene, historical channel, overbank, and historic overbank deposits underlie the levees in this portion of the Calaveras River; the Holocene and historic deposits most likely contribute to underseepage issues in these areas. The west-southwest portion of the Calaveras River extends westward from ¼ mile east of Highway I-5 to the confluence of the LSJR. This is a low lying intertidal area that was prone to depositional and erosional forces prior to levee construction. Levees in this area are underlain by Holocene peat and mud. Other materials such as, marsh, historic overbank, crevasse splay deposits, and channel deposits of varying age also exist in this portion of the river. The historic crevasse splay deposits and the historic overbank deposits most likely contribute to underseepage issues in these areas. The areas with the most potential for underseepage would be the crescent-shaped slivers of Holocene channel deposits. Figure 2-4 shows the geologic units of this area.

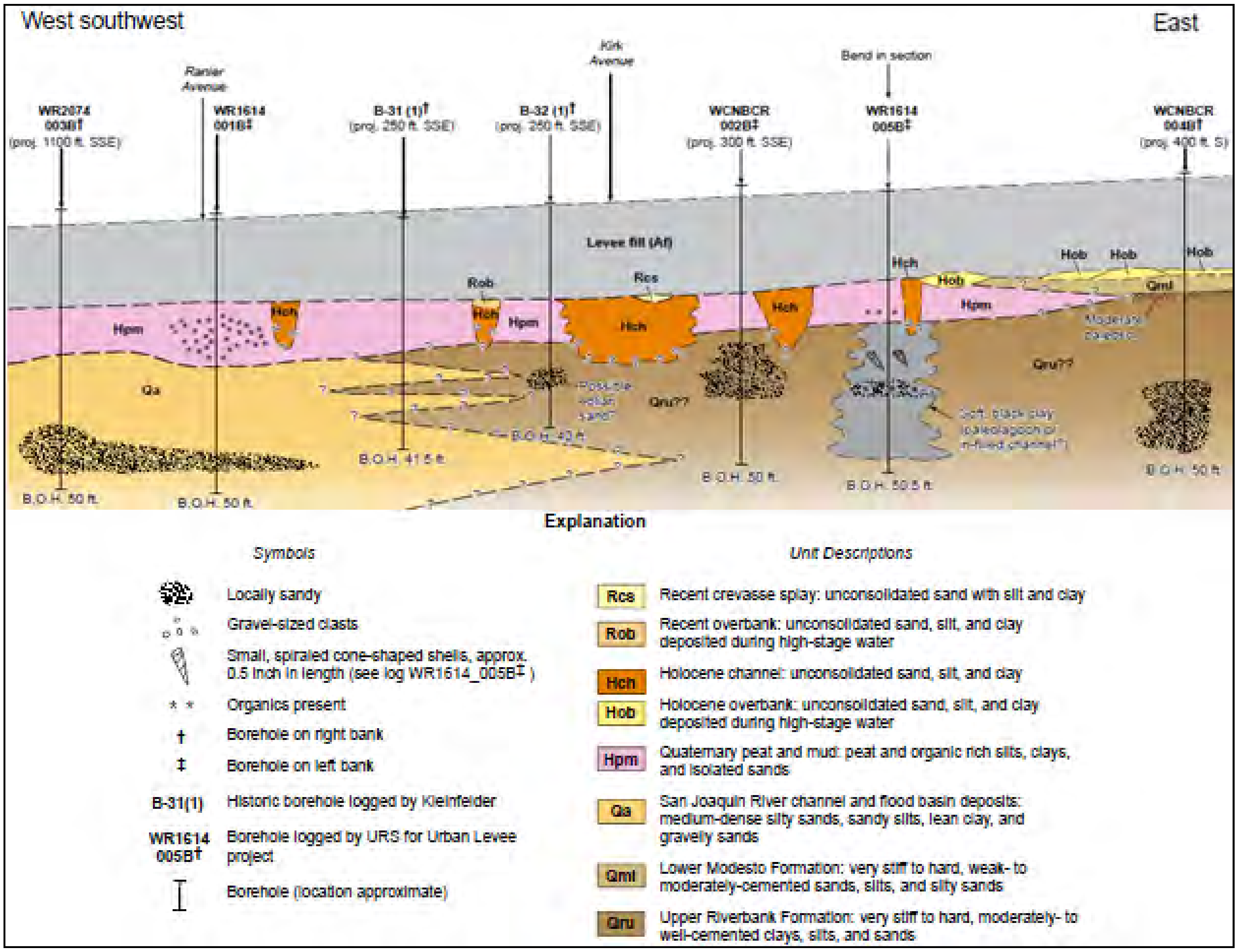


Figure 2-4: Geologic Units of Calaveras River

A surficial geologic map of the Delta Brookside/Delta Lincoln Village study areas (Enclosure 1) shows a northward trending contact just east of Highway I-5 that separates the Delta Geomorphic Domain to the west from the Pleistocene Modesto Formation in the east. The mapped contact between these two domains roughly follows the 1850 tidal line of Atwater (1982). Figure 2-5 shows a cross-sectional view running east to west of the various geologic units. The oldest underlying portions of the Delta islands are late Holocene consisting of unconsolidated organic-rich silts, clays, peat, and mud deposits; these materials accumulated in this intertidal area at or near sea level in these low-flow areas. This material is highly concentrated in both the Delta Brookside and Delta Lincoln Village study areas. Multiple channels of Holocene channel deposits, isolated Holocene overbank deposits, and historical recent overbank deposits crosscut this material flowing across the alluvial fans in a west-southwest orientation; the Holocene and historic deposits most likely contribute to underseepage issues in these areas. The oldest unit within the study area is the late Pleistocene Modesto Formation; this material is unconsolidated, slightly weathered gravels, sands, silts, and clays from upper alluvial fans. The Modesto Formation is exposed along the eastern portions on the study area trending northwest.

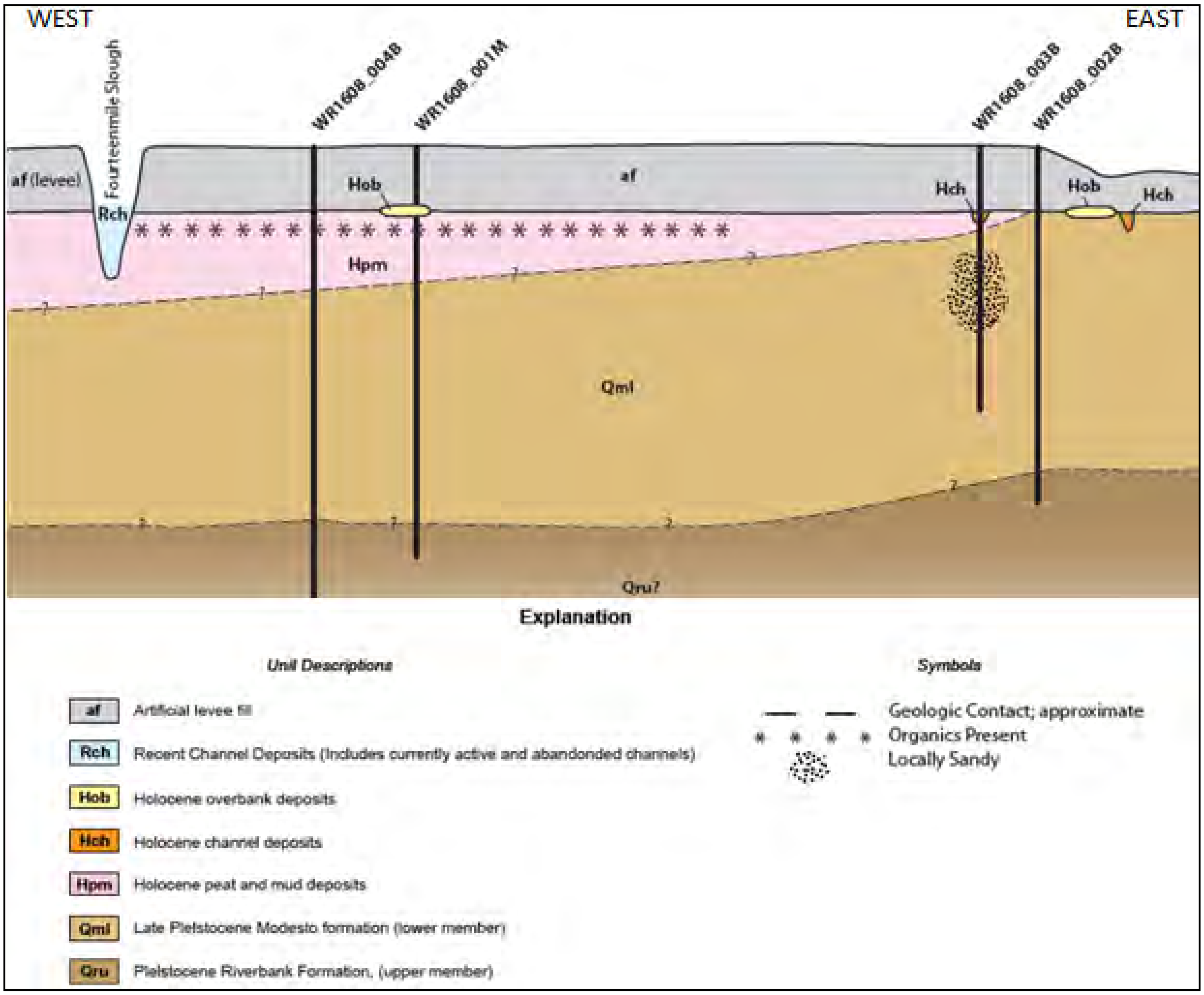


Figure 2-5: Geologic Units of Delta Brookside / Delta Lincoln Village

2.2.3 Seismic Setting

The LSJRFS area lies within the San Joaquin Valley and is exposed to less seismic response during a maximum credible earthquake (MCE) on the nearest active fault than sites in the San Andreas, Hayward, or Calaveras fault zones. Stockton is approximately 65 miles east of the San Andreas Fault. The San Andreas Fault is one of the longest active faults in the world at roughly 600 miles in length, stretching from the coast line in Northern California to the Gulf of California. The San Andreas Fault is capable of generating a moment magnitude (M_W) 8.5 MCE. The last major event of record for this strike-slip fault was the moment magnitude (M_W) 6.9 MCE Loma Prieta earthquake on October 17, 1989. One of the largest events of record for the San Andreas Fault was the moment magnitude (M_W) 7.9 MCE San Francisco earthquake that occurred April 18, 1906.

Stockton is approximately 45 miles east of the Hayward Fault. The Hayward Fault borders the hills of Berkeley and Hayward and extends southeast where it meets up with the Calaveras Fault. The Hayward Fault is capable of generating a moment magnitude (M_W) 7.5 MCE. The last major event of record for this right-lateral, strike-slip fault was on October 21, 1868. The moment magnitude (M_W) of this event is not known, however, it was very destructive.

Stockton is approximately 40 miles east of the Calaveras fault system. The Calaveras fault is approximately 90 to 100 miles in length, extending from central Contra Costa County southeast to where it meets up with the San Andreas Fault just south of Hollister, CA. The Calaveras Fault is capable of generating a moment magnitude (M_W) 7.0 MCE. The last major event of record for this right lateral, strike-slip fault was the moment magnitude (M_W) 6.2 MCE Morgan Hill earthquake on April 24, 1984.

The nearest active fault is the Great Valley 7 fault (part of the San Joaquin Fault zone) located approximately 19 miles southwest of Stockton, CA. The San Joaquin Fault marks the physiographic boundary between the Diablo Range and the Central Valley (Unruh and Krug, 2007). The San Joaquin fault parallels the range-front from the Corral Hollow Creek outlet in the north to the Garzas Creek outlet in the south. Estimates of motion for this fault are in the range of 60 meters of west-side uplift over the last 200 to 300-thousand years. Maulchin (1996) has estimated a M_W 6.5 MCE for this fault; however, there is little evidence that this fault has moved in Holocene times.

Figure 2.6 displays the various Northern California fault zones as shown in a 2010 fault map from California Geological Survey (CGS).

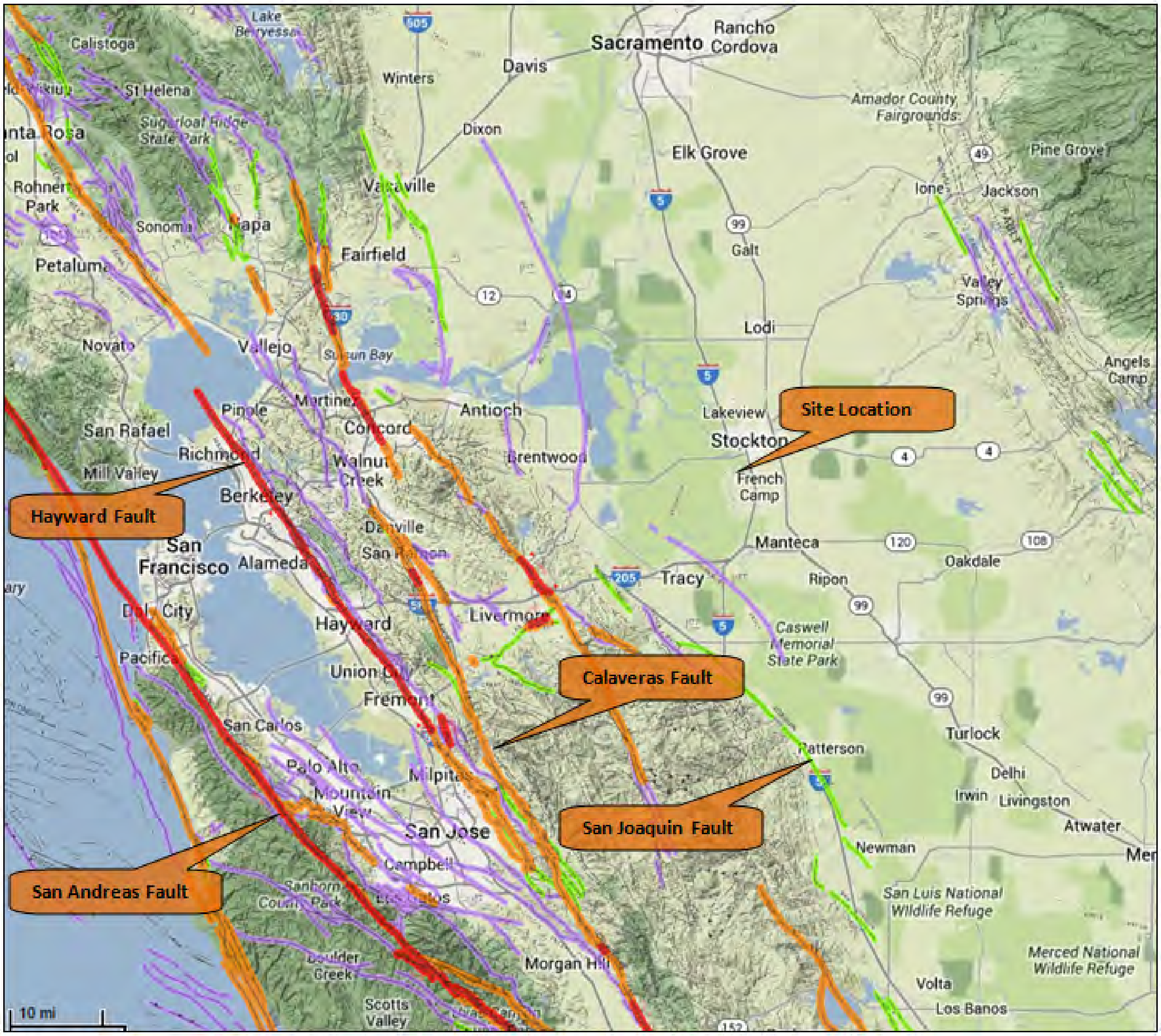


Figure 2-6: Northern California Fault Activity Map, CGS 2010

2.3 LEVEES

2.3.1 Construction History

A mix of Federal, State, and local agencies have been involved in flood control project construction and operation since levees were first constructed in California in the mid to late-1800's. Since the creation of the State Reclamation Board (now the Central Valley Flood Protection Board or CVFPB) in 1911 and the authorization of the California Central Valley Project Act in 1933, most levee improvements have been first Federally authorized by Congress, and then subsequently authorized by the State Legislature.

The first levees along the Lower San Joaquin River were most likely constructed under the California Central Valley Project Act or the Lower San Joaquin River Flood Control Project using clamshell dredges with material sourced from the channel. The levees were usually constructed at least 20 to 50 feet from the river with dredge material placed in the form of a pyramid. The base of the pyramidal shape was up to eighty (80) feet wide built to a height four (4) feet above the 1862 high-water mark. Willows were usually planted along the banks of the river and alfalfa was grown on the slopes of the levee to control erosion. This method of construction usually resulted in loose, sandy fill material that was deepest below the center of the levee. Historic logs show the levee sections were composed of silt to sandy silt, silty sand sometimes interbedded with lean clay, poorly graded sand, and well graded sand. Figure 2-7 shows an example of clamshell dredging performed along the Sacramento River at RM 57.3 in 1942.

Figure 2-7: Clamshell Dredge Along Sacramento River 1942

Many of these levees were then reconstructed, repaired, or reshaped with materials sourced from waterside borrow pits using scrapers, dozers, and compactors between 1947 and 1957. Figure 2-8 below represents a typical levee section constructed on the Lower San Joaquin River in the 1940's through 1950's.

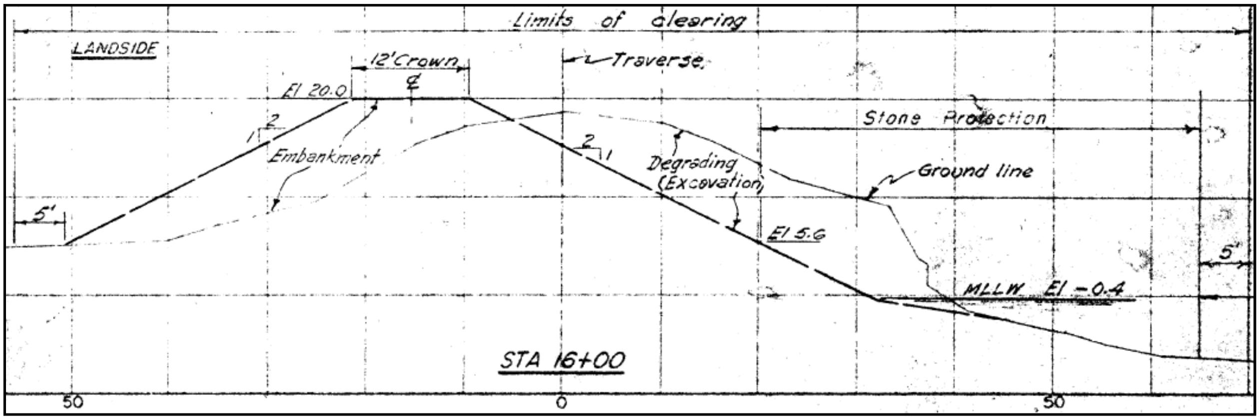


Figure 2-8: Lower San Joaquin River Typical Section, 5 March 1957

It should be noted that because of the construction history outlined above, the upper portion of the semi-pervious blanket beneath the center of the levee has been removed and commonly replaced with sand. Typically, the sand core extends to a greater depth beneath the center of the levee than beneath either of the flanks or the surrounding ground. Most of the levee material was hydraulically dredged from the Lower San Joaquin River and piled or pushed into place with no mechanical compaction. Some mechanical shaping of the upper and outer portions of the sand core likely occurred during establishment of the general levee geometry.

2.3.2 Past performance

The LSJRFS area has experienced several high-water events in recorded history. Journals and legends from Native Americans and explorers document flood events as far back as the 1800's. One of the larger events of record occurred in the winter of 1950 with another following in 1955, and the most recent notable flooding occurring in 1986 and 1997. Though these flood events were documented, past performance history of the individual study areas was not always documented and/or preserved for future use. The following past performance history was obtained from NULE and ULE data reports.

The RD-17 basin has experienced several large flood events. Data reports document interviews with local residents that state several floods occurred in the early 1900's before local farmers purchased their own dredging equipment in attempts to protect their land. Early records document significant seepage erosion, flood fighting, and a levee breach during the flood of December 1950. The failure, approximately 300 feet in length, occurred south of Dos Reis Road. The levees were subjected to record levels again in the 1997 flood event. Emergency flood fighting was initiated when large amounts of seepage and boils were discovered along the landside of the levee. The waterside experienced undercutting, and erosion related to wave runup. An intentional breach upstream in RD-2094 was made in an effort to halt backwater from outflanking the Dryland Levee and entering RD-17 and flooding significantly populated areas. Figures 2-9 and 2-10 documented landside seepage between River Mile (RM) 8.0 and 10.0 of the RD-17 levee along the east bank of the Lower San Joaquin River in 1997.

Figure 2-9: Areas of Seepage 1997, RD-17 (≈RM 8.5)

Figure 2-10: Seepage and Sack Rings 1997, RD-17 (≈RM 9.5)

Data reports document some historical performance issues of the levees along RD-404 from interviews of local residents. The most notable events of record for this area are the January 1997, February 1998, and the early 2006 flood events. The levees experienced landside seepage, boils, and waterside erosion.

Data documenting historical levee performance along the left bank of the Stockton Diverting Canal and Calaveras River are sparse; however, existing data reports document erosion along the left bank of the Stockton Diverting Canal between Waterloo Road and East Fremont Street. The South NULE report addresses the right bank of the Stockton Diverting Canal; the report lists five high-water events (1967, 1969, 1997, 1998, and 2006) for which there were no documented reports of seepage, instability, boils, breaches, or overtopping. Data reports document erosion along both the right and left banks of the Calaveras River between North El Dorado Street, and Brookside Road. Isolated areas of seepage were observed along the Calaveras River (areas were not specified) and did not require emergency flood fighting. A section of levee was reconstructed along the north bank of the Calaveras River (approximately 100 feet in length just south of Brookside School) due to settlement.

Data reports indicate the predominant performance issues for the Delta Brookside Study Area to be settlement, seepage, bank erosion, and rodent activity. Past levee raises, as a result of dredging the Deep Water Channel, induced settlement of the organic soil layers along Tenmile Slough. Areas of historic seepage were documented during the 1997 event and include areas along the San Joaquin River Deep Ship Channel, Buckley Cove, and the south and east banks for Fourteen Mile Slough.

Data reports indicate the predominant performance issues for the Delta Lincoln Village Study Area to be seepage and bank erosion. Bank erosion has steadily increased as boating activities have increased on Fourteen Mile Slough. Bank protection has been an ongoing maintenance activity mitigated with the installation of rip-rap bank protection. The extents of the existing bank protection are not known. Historic seepage has been documented along the southern portion of Lincoln Village along Fourteen Mile Slough (Station 136+70 and 154+10). The data report states that seepage mitigation in the form of cutoff walls were installed in the vicinity of these areas in 1999; however, no As-Builts were obtained to confirm the installation of these measures.

2.4 HYDRAULIC LOADING CONDITIONS

Water surface profiles for the LSJRFS area were obtained from developed cross-sections within existing P1GDR's, P1GER's, and SGDR's provided by the DWR, URS, and Kleinfelder. The cross-sections provided 200 year and sometimes 500 year flood frequencies.

During the preparation of this report, the hydraulic models for these areas were in the process of being revised and updated. Due to the detailed review process required of the hydraulic model update, the decision was made to use design water surface elevations developed in the earlier reports prepared by URS and Kleinfelder as stated in Section 2.1.

Tables 2-1, 2-2, and 2-3 below summarize the water surface elevations deterministically analyzed at each index point, by basin (i.e., South Stockton, Central Stockton, and North

Stockton). Subsequent sections of this report provided more information regarding water surface elevations used for geotechnical analyses. Index points are further described in Section 3.3.4 of this report. All water surface elevations are in NAVD 88.

Table 2-1: South Stockton Basin Analyses Water Surface Elevations (RD-17)

Index Point	Event	Stage	Head
LR-1 RD-17 LSJR	Crest	25.0	15.7
	El.22.4	22.4	14.1
	200yr	19.8	12.6
	El.17.0	17.0	10.9

Index Point	Event	Stage	Head
LR-2 RD-17 LSJR	Crest	27.8	14.7
	El.24.6	24.6	14.3
	200yr	21.5	13.8
	El.17.0	17.0	13.0

Index Point	Event	Stage	Head
LR-3 RD-17 LSJR	Crest	31.0	29.9
	El.28.9	28.9	28.0
	200yr	26.9	26.1
	El.24.0	24.0	23.4

Index Point	Event	Stage	Head
LR-4 RD-17 LSJR	Crest	33.9	23.3
	200yr	31.3	22.4
	El.27.5	27.5	21.1
	El.23.7	23.7	19.9

Index Point	Event	Stage	Head
FL-1 RD-17 French Camp Slough	Crest	21.4	12.2
	El.18.6	18.6	11.5
	200yr	15.9	10.9
	El.13.0	13.0	10.3

Table 2-2: Central Stockton Basin Analyses Water Surface Elevations (RD-404, Stockton Diverting Canal, Left Bank of Calaveras River)

Index Point	Event	Stage	Head
FR-1 RD-404 French Camp Slough	Crest	21.8	5.7
	El.18.8	18.8	5.3
	200yr.	15.9	4.8
	El.12.9	12.9	4.3

Index Point	Event	Stage	Head
SL-1 Stockton Diverting Canal	Crest	39.2	30.5
	El.36.1	36.1	29.3
	El.33.1	33.1	28.0
	200yr.	30.2	26.7

Index Point	Event	Stage	Head
SL-2 Stockton Diverting Canal	Crest	44.6	39.5
	200yr	40.4	37.5
	El.38.8	38.8	36.7
	El.37.2	37.2	35.9

Index Point	Event	Stage	Head
CL-1/CL-2 Calaveras River	Crest	31.4	23.3
	El.29.4	29.4	22.9
	El.27.4	27.4	22.4
	200yr.	25.5	21.7

Index Point	Event	Stage	Head
D-5 Calaveras River	Crest	17.5	9.2
	200yr.	13.2	7.4
	El.10.0	10.0	6.1
	El.7.2	7.2	4.9

Table 2-3: North Stockton Basin Analyses Water Surface Elevations (Right Bank of Calaveras River, Delta Brookside Community and Delta Lincoln Village)

Index Point	Event	Stage	Head
CR-1/CR-2 Calaveras River	Crest	29.7	25.2
	El.28.2	28.2	24.8
	200yr	26.9	24.2
	El.25.3	25.3	23.1

Index Point	Event	Stage	Head
D-4 Calaveras River	Crest	18.8	12.3
	El.16.5	16.5	11.1
	200yr.	14.2	9.9
	El.11.8	11.8	8.6

Index Point	Event	Stage	Head
D-BS Delta Brookside Community	Crest	18.0	3.3
	El.14.0	14.0	2.0
	El.10.0	10.0	0.7
	El.6.0	6.0	0.6

Index Point	Event	Stage	Head
D-LV Delta Lincoln Village	Crest	13.2	3.2
	El.11.0	11.0	2.8
	El.8.5	8.5	2.4
	El.6.0	6.0	2.0

3. WITHOUT PROJECT CONDITIONS

Levee construction and remediation has occurred within the study area since the middle of the 19th century. While the modern levee systems were constructed in the early 20th century and remediated in the 1940's through 1950's, the vast majority of the construction and remediation consisted of crest widening and slope flattening. Beginning in the early 1990s and continuing through present day, some internal improvements have been, and continue to be constructed in the form of cutoff walls and other improvements consisting of seepage and/or stability berms. The without project conditions documented by the sources listed in Section 2.1 are given below.

3.1 POTENTIAL FAILURE MODES

For the purposes of problem identification and alternatives analysis, several different failure modes have been evaluated for the without-project condition. The failure modes included: erosion, overtopping, seepage (under and through), slope stability, and seismic.

3.1.1 Overtopping

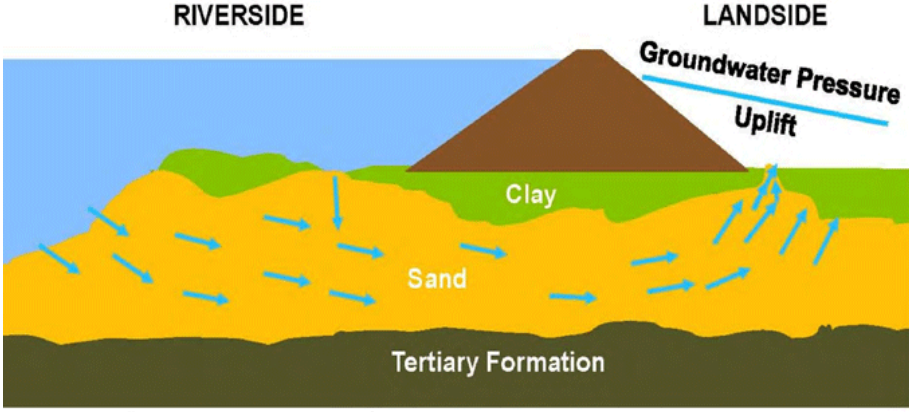
Overtopping occurs when the water surface elevation is greater than the elevation of the levee crest. In this case, water will flow over the crest and onto the landside of the levee. As the levee is overtopped, the action of the water flowing down the landside levee slope and into the basin may cause backside erosion of the landside levee slope and levee toe. This backside erosion may lead to sloughing of the levee and/or a breach condition. For the LSJRFS, the assumption is made that if a levee overtops it fails.

3.1.2 Erosion

Erosion is the wearing away of the riverbank and/or waterside levee slope due to high flows. Erosion can also cause the degradation of the channel invert (scour) causing slope instability. Erosion can occur on the landside of the levee due to overtopping. Erosion occurs when the velocity of the river generates an effective hydraulic shear stress greater than the critical shear stress of the soil over which it flows. As the critical shear stress of the soil is exceeded, soil-particle movement begins. Loosely compacted cohesionless soils are more susceptible to erosion; whereas, cohesive engineered fill is less susceptible. The LSJRFS did not perform explicit analyses for this potential failure mode; erosion was captured as a judgment based curve as part of the performance curves based on historical information and Periodic Inspection (PI) reports.

3.1.3 Seepage

Seepage is subdivided into two categories: seepage through the levee embankment (through-seepage) and seepage beneath the levee embankment through foundation layers (underseepage). Through-seepage occurs when water from the river passes through a pervious levee and weakens the interior of the existing levee causing internal erosion that leads to slope instability or movement of embankment material. Concentrated underseepage that carries silt and sand up to the surface through a more or less open channel in the top stratum (usually of clays and/or silts) is known as a sand boil. Active erosion of sand or other soils from under a levee or top stratum, as a result of substratum pressure and concentration of seepage in localized channels, is known as piping. If the hydrostatic pressure in the pervious substratum landward of a levee becomes greater than the submerged weight of the top stratum, the excess pressure will cause heaving of the top stratum or a rupture at one or more weak spots. This results in a concentration of seepage flow that may cause sand boils and/or underground piping as shown in Figure 3-1.



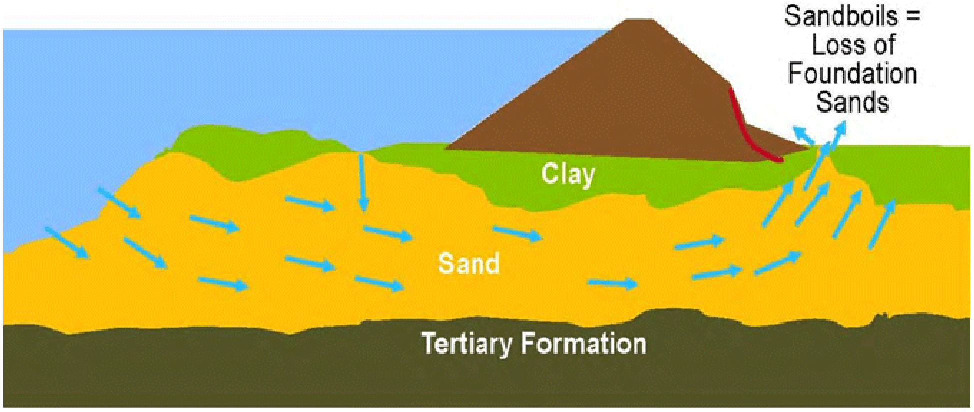
Source: Cory Williams, P.E. – U.S. Army Corps of Engineers

Figure 3-1: Underseepage Distress

3.1.4 Slope Stability

Hydraulic loading of the levee during a flood event reduces the strength of the levee embankment materials causing instability in the embankment slope. Additionally, uplift pressures caused by an excess in pore water pressure at the landside levee toe can lead to the movement of embankment material within the levee due to seepage causing levee instability, as shown in Figure 3-2.

Levee instability can occur on both the waterside and landside of the embankment. Slope stability of the landside slope is typically analyzed, and in instances where the waterside slope is somewhat steep, waterside slope stability may be analyzed as well. Cases will also exist where a rapid drawdown condition occurs. Rapid drawdown conditions arise when a submerged slope experiences a sudden reduction in water level. This change in water surface elevation causes a change in pore water pressure within the embankment. The excess pore water pressure contained in the embankment may lead to a waterside slope stability failure. Even though waterside slope stability and rapid drawdown are potential failure modes, they typically have limited affect on feasibility level designs and are therefore considered design-level analysis.



Source: Cory Williams, P.E. – U.S. Army Corps of Engineers

Figure 3-2: Underseepage Induced Slope Instability Distress

3.1.5 Seismic

Levees can fail as result of a seismic load which may cause degradation due to liquefaction. Liquefaction can lead to detrimental consequences such as loss of freeboard due to embankment instability, transverse crack-induced piping, and loss of freeboard due to settlement. Evaluations are typically completed to determine the liquefaction resistance of soils; this is known as liquefaction triggering. Other seismically induced failure modes include lateral spreading, which can cause vertical displacement of the levee leading to loss of freeboard and levee stability. The seismic analyses performed for this study focuses on liquefaction and vertical displacement as potential seismic failure modes; this analysis is included as Enclosure 4.

3.2 GEOTECHNICAL REACH DESCRIPTION

The following subsections describe the conditions that comprise, and were used to distinguish, reaches for this study that are represented by the Index Points. Table 3.1 summarizes the reach of levee represented by each Index Point.

Table 3-1: LSJRFS Area Levees

Basin	Reaches	Channel	Maintaining Agency	Length (mi)
South Stockton	LR-1	Lower San Joaquin River	RD-17	6.4
	LR-2			3.8
	LR-3			1.5
	LR-4			1.5
	FL-1	French Camp Slough		1.9
Central Stockton	FR-1	French Camp Slough	RD-404	2.1
	SL-1	Stockton Diverting Canal	SJAFCA	2.2
	SL-2		SJAFCA	2.9
	CL-1/CL-2	Calaveras River (left bank)	SJAFCA	2.9
	D-5		RD-1614	3.1
North Stockton	CR-1/CR-2	Calaveras River (right bank)	SJAFCA	2.9
	D-4		RD-2074	3.2
	D-BS	LSJR/Tenmile Slough/Fourteen Mile Slough	RD-2074	3.7
	D-LV	Fourteen Mile Slough	RD-1608	2.5

3.2.1 RD-17 Basin

The RD-17 levees, including the east bank of the Lower San Joaquin River and the left bank of French Camp Slough, extend for approximately 15 miles. The levee crest height ranges from 8 to 16 feet above the landside levee toe. The crest width varies from 12 to 20 feet. The landside and waterside slopes are predominantly 2H:1V or flatter (H:V, Horizontal: Vertical); however, there are areas throughout the system with slopes steeper than 2H:1V. The RD-17 levee system resides in both a high density housing urban area and rural agricultural area. In the northern area, there is significant waterside vegetation (mostly large trees and riparian habitat) that thins out to sparse waterside vegetation heading south along the embankment. In some areas, landside vegetation (mostly trees) exists near the levee toe or on the levee slopes. On the landside, numerous encroachments include: fences at or near the landside levee toe, out buildings, residences, parks, pump stations, agricultural land, power poles, road crossings, Highway/Freeway I-5, and 120, railroad crossings, ditches, treatment plants, and water bodies.

- At index point location FL-1 the levee embankment is predominantly sandy lean clay with a lean clay to sandy lean clay blanket underlain by an aquifer composed of silty sand. Geomorphology in this area shows stringers of Historical channel deposits.

- At index point location LR-1 the levee embankment varies from lean clay to silt with a lean clay blanket underlain by an aquifer composed of poorly graded sand with silt to silty sand. Geomorphology in this area shows stringers of Historical channel deposits.
- At index point location LR-2 the levee embankment varies from poorly graded sand with silt to clayey sand with a thin lean clay to silty sand blanket underlain by an aquifer composed of poorly graded sand with silt. Geomorphology in this area shows significant areas of overbank and basin deposits.
- At index point location LR-3 the levee embankment varies from lean clay to silty sand with a silty sand blanket underlain by an aquifer composed of poorly graded sand with silt to silty sand. Geomorphology in this area shows significant areas of Holocene and Historical alluvial fan deposits.
- At index point location LR-4 the levee embankment is predominantly clayey sand with a lean clay to sandy lean clay blanket underlain by an aquifer composed of poorly graded sand with silt. Geomorphology in this area shows significant areas of Holocene alluvial fan deposits.

3.2.2 RD-404

The RD-404 levee along the right bank of French Camp Slough extends for approximately 2 miles. The levee crest height ranges from 10 to 13 feet above the landside levee toe. The crest width varies from 15 to 25 feet. The landside slopes are predominantly 2H:1V or flatter. The waterside slopes are predominantly steeper than 2H:1V. There is vegetation along both the landside and waterside of the levee embankment; mostly shrubs, small trees, and riparian habitat along the waterside, and large trees along the landside levee toe and slopes. On the landside, there are some encroachments due to outbuildings, power poles, water bodies, and parking areas related to Van Buskirk Park Golf Course, as well as the I-5 Highway.

- At index point location FR-1 the levee embankment is predominantly lean clay and silt with a thin clayey sand blanket underlain by an aquifer composed of silty sand. Geomorphology in this area shows predominantly marsh deposits with stringers of Historical channel deposits.

3.2.3 Stockton Diverting Canal

The levee along the left bank of the Stockton Diverting Canal extends for approximately 5 miles. The levee crest height ranges from 10 to 16 feet above the landside levee toe. The crest width varies from 14 to 25 feet. The landside and waterside slopes are predominantly 2H:1V; however, there are areas throughout the system with slopes steeper than 2H:1V. A waterside bench, approximately 20 feet wide, is present. The levee system resides in both a high density housing urban area and an industrial area. Areas of landside vegetation are present in the urban area. In some areas, landside vegetation (mostly trees) exists near the levee toe or on the levee slope. Waterside vegetation consists of sparse grasses and shrubs. On the landside, numerous encroachments include: fences at or near the landside levee toe, out buildings, residences, railroad tracks/rail yard, pump stations, power poles, road crossings, railroad crossings, industrial areas, parking and storage areas, and Highway/Freeway 99, 88, and 26.

- At index point location SL-1 the levee embankment is predominantly sandy lean clay with a thin lean clay blanket underlain by an aquifer composed of silty sand. Geomorphology in this area shows stringers of Historical and Holocene channel deposits.
- At index point location SL-2 the levee embankment is predominantly sandy silt with a lean clay blanket underlain by an aquifer composed of silty sand. Geomorphology in this area shows stringers of Holocene overbank and channel deposits.

3.2.4 Calaveras River South Bank

The levee along the left (south) bank of the Calaveras River extends for approximately 6 miles. The levee crest height ranges from 8 to 14 feet above the landside levee toe. The crest width is predominantly 12 feet that widens towards road crossings. The landside and waterside slopes are predominantly 2H:1V or flatter; however, there are areas throughout the southern alignment with slopes steeper than 2H:1V, and an area along the waterside that is roughly 1H:1V. A waterside bench from 10 to 20 feet wide is present throughout the southern alignment. The levee system resides in various settings. Urban area high density housing is present throughout most of the alignment; however, agricultural land, industrial areas, educational areas, and recreational areas are also present. Landside vegetation (mostly trees) is present in the urban, agricultural, educational, and recreational areas. In some areas, landside vegetation exists near the levee toe, on the levee slope, or on the crest of the levee. Waterside vegetation consists of sparse grasses, shrubs, and a few trees along the toe and slopes in the eastern portion of the alignment; more dense waterside vegetation (mostly trees) is present west of University of the Pacific to the confluence of the LSJR. On the landside, numerous encroachments include: fences at or near the landside levee toe, out buildings, residences, stairs on slopes, railroad crossings, pump stations, road crossings, power poles, industrial areas, parking lots, Highway I-5, recreational facilities including Stockton Golf and Country Club; waterside encroachments include: stairs on slopes, boat docks, and recreational facilities including Stockton Yacht Club.

- At index point location CL-1/CL-2 the levee embankment is predominantly sandy silt with an elastic silt blanket underlain by a deeper aquifer composed of poorly graded sand with silt. Geomorphology in this area shows stringers of Historical and Holocene channel deposits.
- At index point location D-5 the levee embankment is predominantly silt with a lean clay blanket underlain by an aquifer composed of silty sand. Geomorphology in this area shows an abundance of peat, mud, and organic material with stringers of Holocene overbank and channel deposits.

3.2.5 Calaveras River North Bank

The levee along the right (north) bank of the Calaveras River extends for approximately 6 miles. The levee crest height ranges from 6 to 12 feet above the landside levee toe. The crest width varies from 12 to 15 feet and widens towards road crossings. The waterside slopes are predominantly 2H:1V or flatter; however, there are a few areas throughout the northern alignment with slopes steeper than 2H:1V; and an area along the waterside that is roughly 1H:1V. The landside slopes are predominantly 2H:1V or steeper throughout the northern alignment. A waterside bench 30 to 50 feet wide is present throughout the northern alignment.

The levee system resides predominantly in an urban area with high density housing, churches, and several schools. Landside vegetation (mostly trees) is present in the urban and educational areas. In some areas, landside vegetation exists near the levee toe, on the levee slope, or on the crest of the levee. Waterside vegetation consists of sparse grasses, shrubs, and a few trees along the toe and slopes in the eastern portion of the alignment; more dense waterside vegetation (mostly trees) is present west of Stagg High School to the confluence of the LSJR. On the landside, numerous encroachments include: fences at or near the landside levee toe, fences on slopes, out buildings, residences, swimming pools, stairs on slopes, railroad crossings, pump stations, power poles, road crossings, parking lots, and Highway I-5; waterside encroachments include stairs on slopes, and boat docks.

- At index point location CR-1/CR-2 the levee embankment is predominantly sandy lean clay with a thin blanket of sandy lean clay underlain by an aquifer composed of sandy silt. Geomorphology in this area shows an abundance of alluvial deposits with stringers of Holocene channel deposits.
- At index point location D-4 the levee embankment varies from sandy silt to sandy lean clay with a thin blanket of sandy fat clay and sandy silt underlain by an aquifer composed of poorly graded sand with silt. Geomorphology in this area shows stringers of Holocene overbank and channel deposits.

3.2.6 Delta Brookside Study Area

The Delta Brookside Study Area levee extends approximately 3.5 miles along the west and north of the Brookside community, an urban high density housing development. The levees reside along the Stockton Deep Water Channel of the LSJR, Buckley Cove, Tenmile Slough, and Fourteen Mile Slough. Along the Deep Water Channel, the levee crest height ranges from 6 to 12 feet above the landside levee toe. The crest width varies from 12 to 16 feet and widens towards Buckley Cove. The landside and waterside slopes are predominantly 2H:1V or flatter; however, there are a few areas along the waterside with slopes steeper than 2H:1V. Along Buckley Cove, the levee crest height ranges from 8 to 18 feet above the landside levee toe. The crest width varies from 14 to 20 feet. The landside and waterside slopes are predominantly 2H:1V or flatter. Along Tenmile Slough, the levee crest height ranges from 16 to 20 feet above the landside levee toe. The crest width varies from 14 to 18 feet. The landside and waterside slopes are predominantly 2H:1V or flatter. Along Fourteen Mile Slough, the levee crest height ranges from 8 to 14 feet above the landside levee toe. The crest width varies from 18 to 40 feet. The landside and waterside slopes are predominantly 2H:1V or flatter. Landside vegetation (mostly trees) is present throughout the highly urbanized area at most residences, and in most cases near the levee toe. Waterside vegetation consists of a few trees at the toe within the Deep Water Channel, grasses, shrubs, and trees along Buckley Cove, shrubs and brush along Tenmile Slough, and a few trees along Fourteen Mile Slough. On the landside, numerous encroachments include: fences at or near the landside levee toe, fences on slopes, decks and/or retaining walls on slopes and crest, out buildings, residences, swimming pools, stairs on slopes, and pump stations; waterside encroachments include: stairs on slopes, concrete patios/decks, boat docks, road crossings, and Highway I-5.

- At index point location D-BS the levee embankment is predominantly lean clay with portions of an older levee constructed of organic clay. The thin blanket varies from organic clay to lean clay underlain by an aquifer composed of silty sand. Geomorphology in this area shows an abundance of peat, mud, and organic material with stringers of Holocene channel deposits and overbank deposits.

3.2.7 Delta Lincoln Village Study Area

The Delta Lincoln Village Study Area levee extends approximately 2.5 miles along the west and south of the Lincoln Village community on Fourteen Mile Slough, an urban high density housing development. The levee crest height ranges from 6 to 12 feet above the landside levee toe. The crest width varies from 12 to 14 feet and widens towards road crossings. The waterside slopes are predominantly 2H:1V or flatter; however, there are a few areas near Station 200+00 with slopes steeper than 2H:1V; and two areas roughly 1H:1V. The landside slopes are predominantly 2H:1V or flatter throughout the alignment. Landside vegetation (mostly trees) is present throughout the highly urbanized area at most residences, and in most cases near the levee toe. Waterside vegetation (mostly trees) begins moving south along the alignment just before Village West Yacht Club; the waterside vegetation (mostly trees) becomes denser heading south then east along Fourteen Mile Slough. On the landside, numerous encroachments include: fences at or near the landside levee toe, fences on slopes, fences on crest, decks and/or retaining walls on slopes and crest, out buildings, residences, power poles, swimming pools, stairs on slopes, and pump stations; waterside encroachments include: stairs on slopes, concrete patios/decks, boat docks, Village West Yacht Club, road crossings, and Highway I-5.

- At index point location D-LV the levee embankment is predominantly lean clay with a thin blanket of lean clay underlain by a deep aquifer was comprised of silty sand to poorly graded sand with silt. Geomorphology in this area shows an abundance of peat, mud, and organic material with stringers of Holocene channel deposits, overbank deposits, and marsh deposits.

3.3 SEEPAGE AND STABILITY METHODOLOGY

Deterministic seepage and stability analyses were performed for various water surface elevations, including top of levee. The probabilistic analyses were performed for a range of stages not correlated to flood frequency, but which represented stages from no head (landside toe of levee) to maximum head (top of levee). Refer to Section 2.4 for water surface elevations used at each Index Point for seepage and stability analyses.

3.3.1 Steady State Seepage Analysis

Deterministic steady state seepage analysis was performed using SEEP2D within GMS 6.5 (Groundwater Modeling System), a finite element program. Results from the seepage analysis were used to calculate average vertical exit gradients at the landside levee toe and/or at a more critical location near the levee toe if applicable; for example, at the invert of the empty drainage ditch. The pore pressures and/or phreatic surfaces were exported to UTEXAS4 for use in slope stability analysis.

Boundary conditions along the waterside ground surface from the waterside model extents to the levee slope were assigned as fixed total head conditions corresponding to the analyzed water elevation. On the landside, exit face boundary conditions are applied from the crest hinge point to landside extents of the model. All other boundaries not explicitly assigned a condition are assumed by the program to be no flow; this includes both vertical faces of the model and the bottom nodes. The landside model extents were extended 2,000 feet from the levee centerline and to the end of available topographic information on the waterside. Figure 3-3 shows a typical GMS SEEP2D seepage model.

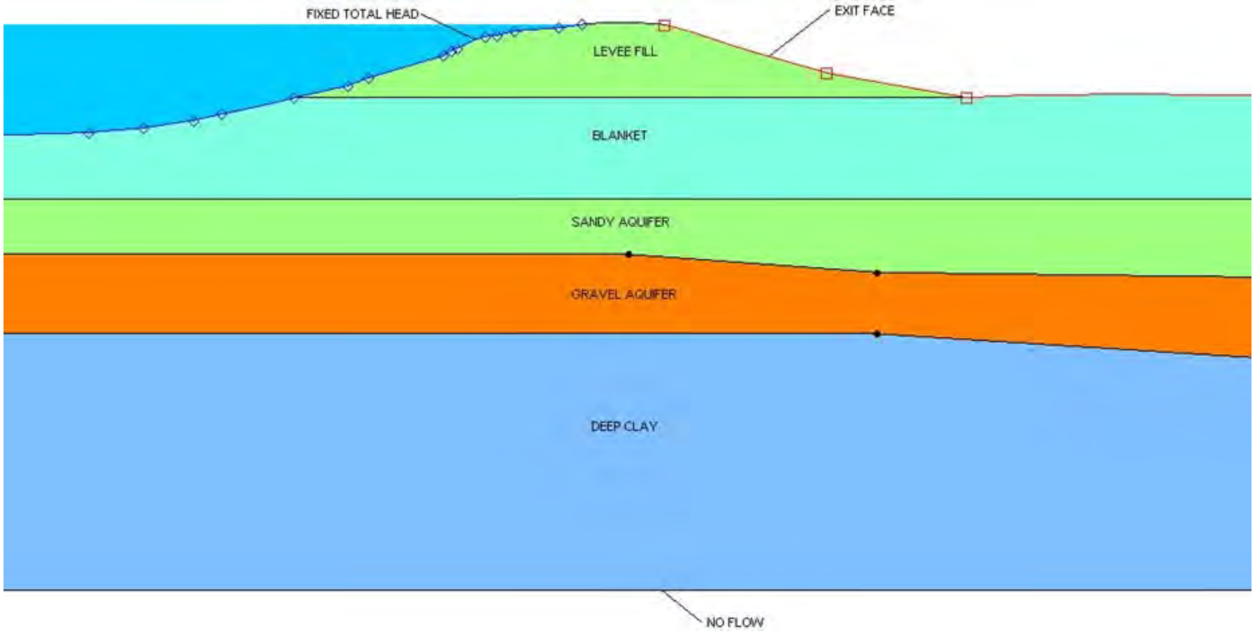


Figure 3-3: Typical GMS SEEP2D Seepage Analysis Model

Levees constructed of fine grained clays having stability berms with drainage layers that capture any seepage through the levee, or having cutoff walls constructed through the levee embankment, are unlikely to be susceptible to through-seepage caused internal erosion. Levees of silt, silty sand, and/or sand were considered to be susceptible to internal erosion caused by through-seepage and were considered as deficient from a through-seepage perspective.

3.3.2 Steady State Slope Stability Analysis

Embankment stability against shear failure was analyzed using the UTEXAS4 software package for steady state conditions. Analyses to find factors of safety against sliding were conducted using a floating grid automatic circular failure surface search routine to identify the critical failure surfaces with the Spencer Procedure within the embankment and/or foundation. The Spencer Procedure satisfies both force and moment equilibrium for each slice. A minimum weight restriction was applied to the slices within the failure surface to eliminate surficial failure surfaces. Where tensile stresses exist on the failure surface, a crack depth was introduced to eliminate the tensile stresses, but not compressive stresses. The appropriate depth for a crack is the one producing the minimum factor of safety (FOS), which corresponds to the depth where tensile, but no compressive stresses are eliminated. If a crack was required, the maximum crack depth was set to producing the lowest FOS; typically, two to four feet. Figure 3-4 shows a typical UTEXAS4 model.

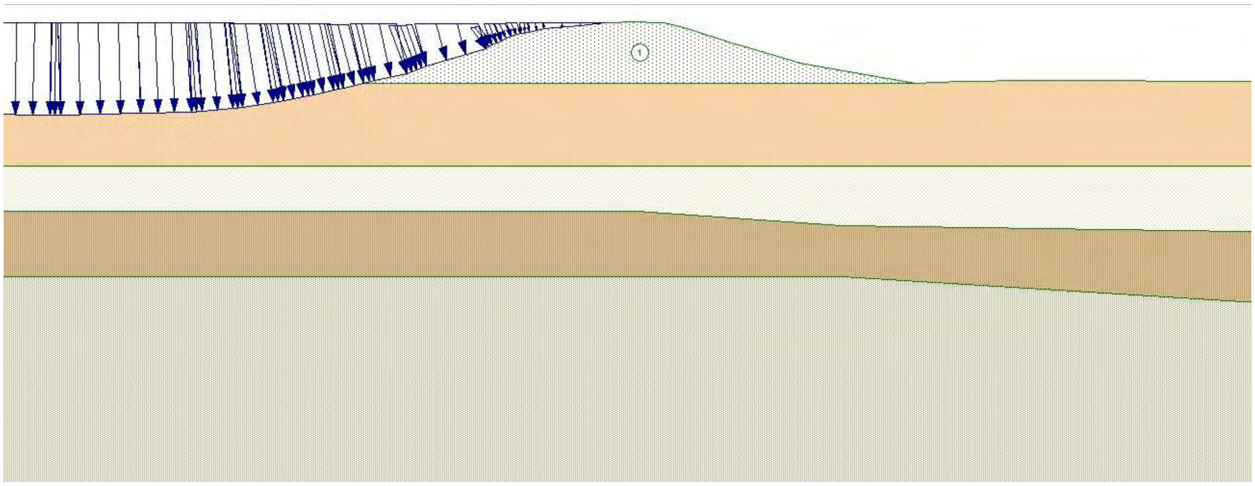


Figure 3-4: Typical UTEXAS4 Slope Stability Analysis Model

The long term evaluation was considered with steady state seepage and is based on the assumption of a fully developed phreatic surface through the embankment. Saturated unit weights are used in the embankment and the pore water pressure is imported from SEEP2D. External water pressures from the channel are applied as a distributed load against the landside slope. Effective shear strength parameters c' and Φ' were used for all materials.

3.3.3 Material Properties

In order to develop geotechnical products for the LSJRFS area in a timely manner, the PDT and Sponsors agreed to use existing subsurface information (i.e., Geotechnical Data Reports (GDR) and Geotechnical Engineering Reports (GER)) developed by both URS and Kleinfelder for DWR. Cross sections, material properties, including hydraulic conductivity for seepage analysis and drained (effective) shear strength and unit weight for slope stability analysis, were obtained from existing P1GER's provided by DWR, URS, and Kleinfelder. The stratigraphy of the existing levee cross-sections were divided into unique layers typically consisting of levee embankment fill, a foundation or blanket layer, pervious aquifer layers separated by an aquitard, and a deeper fine grained layer. The hydraulic conductivities, shear strengths, and unit weights used in the seepage and slope stability analysis are included in Enclosure 2.

The hydraulic conductivities developed in the earlier GER's were reevaluated and assigned based on soil classification and fines content using typical values developed and evolved from soil index property and hydraulic conductivity testing on samples gathered from numerous subsurface investigations coupled with limited in-situ testing and engineering judgment performed by USACE, DWR, URS, Kleinfelder, and others on similar levees and in similar geologic conditions to this project. These values have been adapted for this project and are presented in Table 3-2 below.

Many soil deposits have a different horizontal hydraulic conductivity than vertical hydraulic conductivity. The ratio of horizontal hydraulic conductivity divided by vertical hydraulic conductivity is referred to as anisotropy ratio (k_H/k_V). Anisotropy between horizontal and vertical conductivities is influenced by a number of factors including a variation in material properties within a modeled layer (inter-bedded lenses of sand in a silt or clay layer), cracks within the layer, etc. The analyses were performed using a soil anisotropy ratio of 4 for most naturally deposited layers. Thin clay blankets were given an anisotropy ratio of 1 to 0.10 (assumed to be cracked) and some sands and gravels were given an anisotropy ratio of 10.

Table 3-2: Hydraulic Conductivities

Material Type	**Soil Description**	**Hydraulic Conductivity**				
		k_H (cm/sec)	**k_H (ft/day)**	**k_H/k_V**	**k_V (cm/sec)**	**k_V (ft/day)**
Cutoff Wall	SCB, SB, CB	1.0E-06	0.0028	1	1.0E-06	0.0028
Clay	Engineered Embankment	1.0E-06	0.0284	1	1.0E-0.6	0.0284
	Non-Engineered Embankment	1.0E-05	0.0284	4	2.5E-06	0.007
	Blanket ≥10ft Thick or Embankments	1.0E-05	0.0284	4	2.5E-06	0.007
	Blanket 5ft<>10ft Thick	1.0E-05	0.0284	1	1.0E-05	0.0284
	Blanket ≤5ft Thick	1.0E-05	0.0284	0.10	1.0E-04	0.284
Silt	Elastic (plastic)	5.0E-05	0.14	4	1.3E-05	0.035
	Non-plastic	2.0E-04	0.57	4	5.0E-05	0.14
Clayey Sand to Sand	30-49% fines	5.0E-05	0.14	4	1.3E-05	0.035
	13-29% fines	1.0E-04	0.28	4	2.5E-05	0.071
	8-12% fines	1.0E-03	2.8	4	2.5E-04	0.71
	0-7% fines	5.0E-03	14	4	1.3E-04	3.5
Silty Sand to Sand	30-49% fines	5.0E-04	1.4	4	1.3E-04	0.35
	13-29% fines	1.0E-03	2.8	4	2.5E-04	0.71
	8-12% fines	5.0E-03	14	4	1.3E-03	3.5
	0-7% fines	1.0E-02	28	4	2.5E-03	7.1
Gravel	28-49% fines	4.0E-04	1.13	4	1.0E-04	0.28
	18-27% fines	1.0E-03	2.8	4	2.5E-04	0.71
	13-17% fines	6.0E-03	17	10	6.0E-04	1.7
	8-12% fines	1.2E-02	34	10	1.2E-03	3.4
	0-7% fines	2.5E-02	71	10	2.5E-3	7.1
Gravel with Cobbles and Sand	28-49%fines	4.0E-04	1.13	4	1.0E-04	0.28
	18-27% fines	1.0E-03	2.8	4	2.5E-04	0.71
	13-17% fines	1.0E-02	28	10	1.0E-03	2.8
	8-12% fines	1.0E-01	284	10	1.0E-02	28
	0-7% fines	2.0E-01	570	10	2.0E-02	57
Drain Rock	Gravel	1.0E01	2835	1	1.0E01	2835

The resistance to penetration of the soils measured in blows per foot (field N-value) during the driving of Standard Penetration Test (SPT) samplers and Cone Penetrometer Test (CPT) tip resistance served as a site specific data source for the determination and verification of shear strength parameters for granular, cohesionless soils through empirical correlations. Empirical correlations with SPT N-values by Uchida (1996) and Peck (1974) were used for the estimation of the drained (effective stress) angle of internal friction Φ'. For cohesive soils (including clays and plastic silts), the empirical correlations by Mitchell (1976) and Bowles (1996) were used for estimation of Φ' using the Plasticity Index (PI) of the soil.

For both cohesive and cohesionless materials, shear strengths predicted by correlations were compared to typical published values and values used in previous analysis in similar materials, and then adjusted based on engineering judgment. Typical shear strengths by material classification used in steady state slope stability analysis are shown in Table 3-3.

Table 3-3: Shear Strength of Soils

Material Type	**Soil Description**	**Shear Strength**		
		c' (psf)	**Φ' (^{o})**	**γ(pcf)**
Cutoff Wall	SB	50	0	85
	SCB	500		
	CB	5000		
Clay	Clay Foundation	50-100	20-30	115
	Clay Engineered Embankment	50-200	28-30	115
	Clay Non-engineered Embankment	50-100	22-26	115
Silt		0	28-32	120
Clayey Sand and Silty Sand		0	28-33	125
Sand		0	30-35	130
Gravel and Drain Rock		0	35-40	135

3.3.4 Representative Cross Sections

Typically, cross-sections for geotechnical analysis are selected to represent critical surface and subsurface conditions of each reach. The topography of each reach is inherently variable. The existence of access ramps on both the landside and waterside of the levee, roadways and railroads running perpendicular and parallel to the levee, and/or pump stations or other structures built up adjacent to the levee section create difficulties to discern the typical versus critical cross-section.

For the LSJRFS area, the sections were selected based on subsurface data, laboratory test results, geomorphology, surface conditions, field reconnaissance, historical performance, and levee geometry. The ground surface elevations used in the cross-sections were based on the LiDAR and bathymetric surveys performed by URS, Kleinfelder, and Fugro for DWR from 2007 and 2008. The natural soil layers were delineated based on boring logs and laboratory test results. Typically one cross section per reach was selected for analysis and referred to as an index point.

In some cases, multiple cross sections were analyzed in each reach to verify the initial location. Table 3-4 and Figure 3-5 present the location of the cross-sections representing the LSJRFS index points. A total of fourteen (14) cross-sections were analyzed, 4 cross-sections were analyzed in the RD-17 Basin along the east bank of the LSJR, 2 cross-sections were analyzed in the French Camp Slough area (one section in the northern portion of RD-17, one section in the southern portion of RD-404); 2 cross-sections were analyzed along the west bank of the SDC; 4 cross-sections were analyzed along the Calaveras River (two sections along the right bank, two section along the left bank); and 2 cross-sections were analyzed in the Delta Front area (one section in the Brookside area, one section in the Lincoln Village area).

Table 3-4: Index Point Locations ([1]200-yr. WSE not given)

Index Point	Station	State Plane (ft) Northing	State Plane (ft) Easting	Crest Elev. (ft)	≈200-yr DWSE (ft) NAVD88	River
CL1/CL2	6757+00	2185288	6336628	31.4	25.5	Calaveras River
CR1/CR2	3306+00	2185583	6337043	29.7	26.9	Calaveras River
D4	3092+00	2180295	6319366	18.8	14.2	Calaveras River
D5	6535+00	2178738	6317908	17.5	13.2	Calaveras River
SL1	846+68	2183207	6340943	39.2	30.2	Diverting Canal
SL2	976+00	2176913	6352470	44.6	40.4	Diverting Canal
FR1	1164+20	2158156	6329042	21.8	15.9	French Camp Slough
FL1	1049+00	2156653	6329984	21.4	15.9	French Camp Slough
LR1	1292+00	2139808	6322846	25.0	19.8	San Joaquin River
LR2	1417+00	2131643	6324275	27.8	21.5	San Joaquin River
LR3	1685+00	2116984	6326321	31.0	26.9	San Joaquin River
LR4	1815+00	2105994	6330785	33.9	31.3	San Joaquin River
D-LV	162+50	2185939	6315555	13.6	11.0[1]	14-Mile Slough
D-BS	166+50	2183200	6311320	18.2	10.0[1]	LSJ/14-Mile Slough

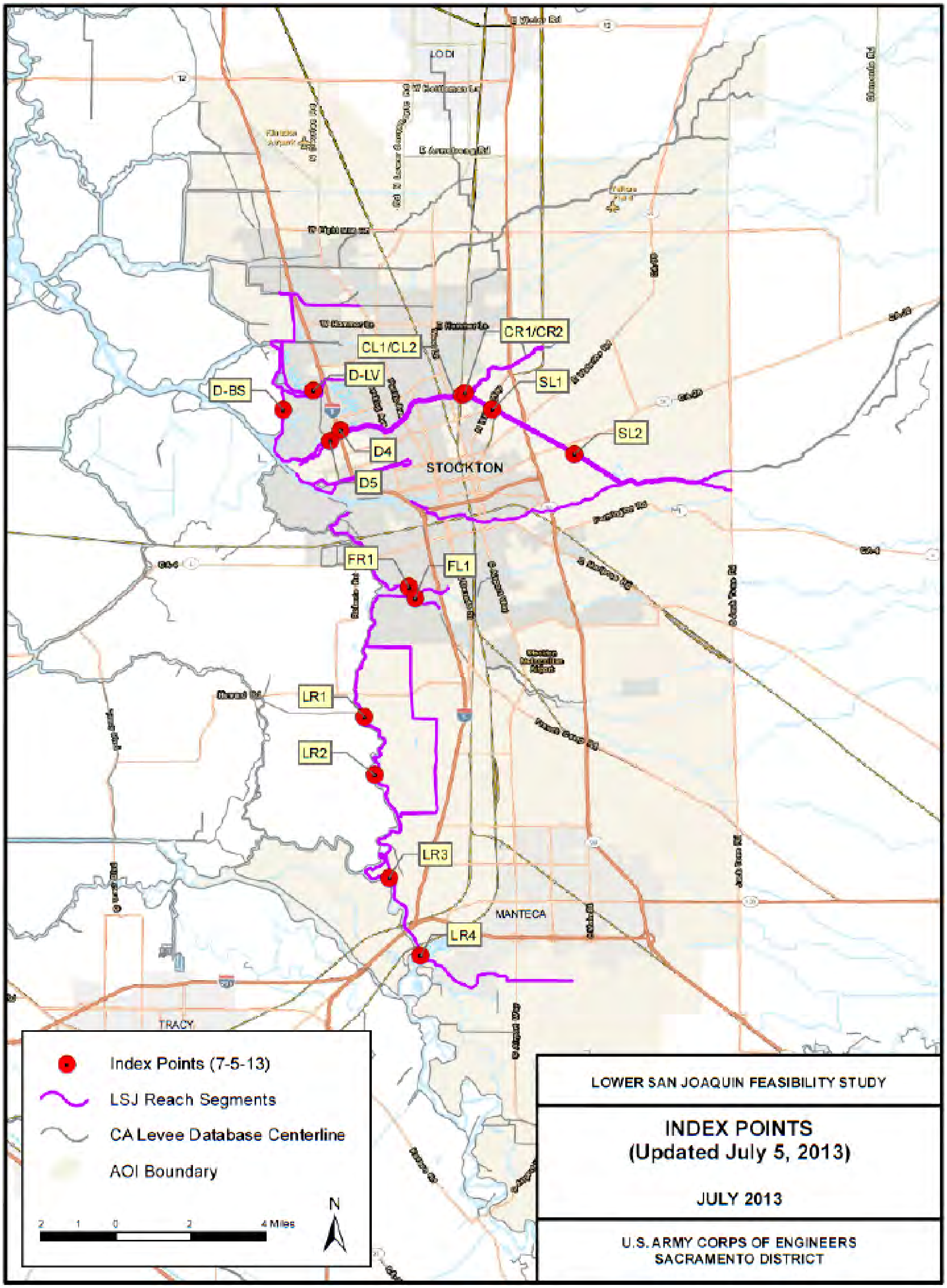


Figure 3-5: LSJRFS Index Point Location Map

3.4 SEEPAGE AND STABILITY ANALYSIS RESULTS

The following section presents the results of geotechnical steady state seepage and slope stability analyses performed in accordance with the methodology described in Section 3.3. The analyses cross-sections were evaluated in accordance with design criteria described in Section 4.3.2 for various water surface elevations, including top of levee, as indicated in Section 2.4. The analyses for each location were performed for the without-project condition as described in Section 3.3.

Enclosure 2 contains a tabulation of the analyses results including: the hydraulic conductivities and material strength parameters assigned for each cross-section used in analysis; seepage gradients and slope stability factors of safety for various WSE; plates of cross-section geometry, stratigraphy, total head contours (seepage analysis), and failure surfaces (slope stability analysis) for a crest water surface elevation are included.

The following subsections present the analyses results for without project conditions at each of the cross-section locations. Figures presented for each cross-section display underseepage average vertical exit gradient calculated at the landside levee toe and slope stability FOS for the analyzed water surface elevations.

3.4.1 South Stockton – Lower San Joaquin River East Bank RD-17

The without-project conditions analyses for south Stockton includes five (5) index points; four (4) along the right (east) bank of the Lower San Joaquin River, and one (1) index point along the left (south) bank of French Camp Slough; these five (5) index points represent RD-17. The index point locations, LR-1, LR-2, LR-3, LR-4, and FL-1, are shown in Figure 3.5. As the results show below, LR-1, LR-2 and LR-3 display exit gradients and slope stability factors of safety (FOS) that do not meet design criteria at various water surface elevations. Figure 3-6 to Figure 3-10 displays steady state seepage and landside slope stability results for the analyzed water surface elevations.

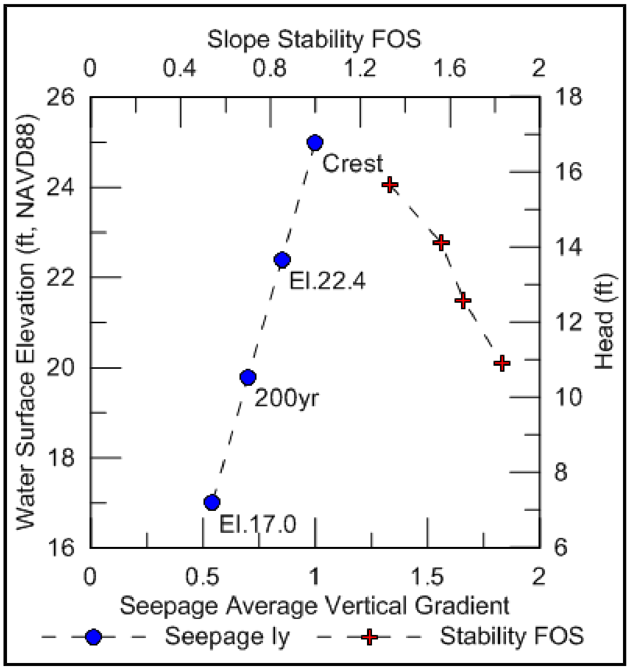


Figure 3-6: RD-17 Index Point LR-1 Without-Project Analyses Results

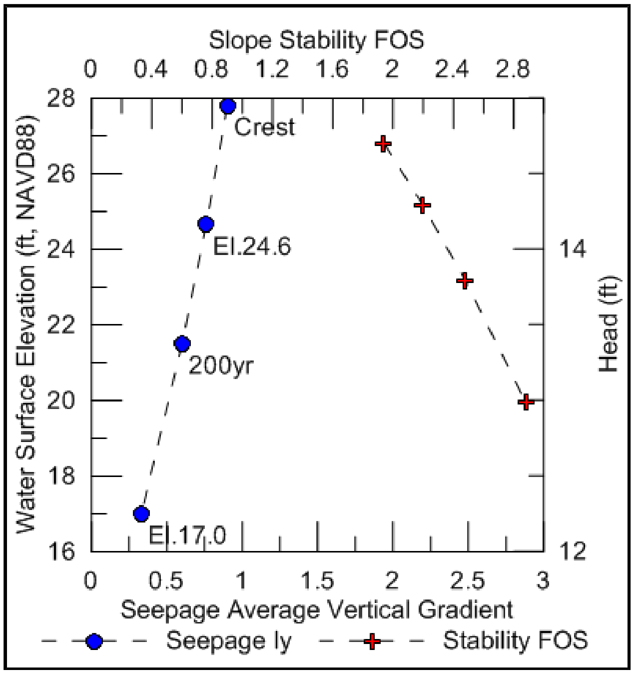


Figure 3-7: RD-17 Index Point LR-2 Without-Project Analyses Results

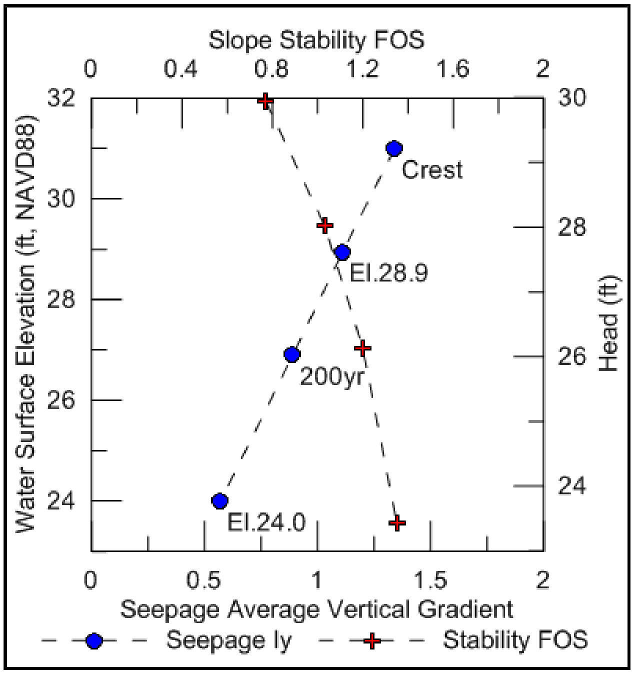


Figure 3-8: RD-17 Index Point LR-3 Without-Project Analyses Results

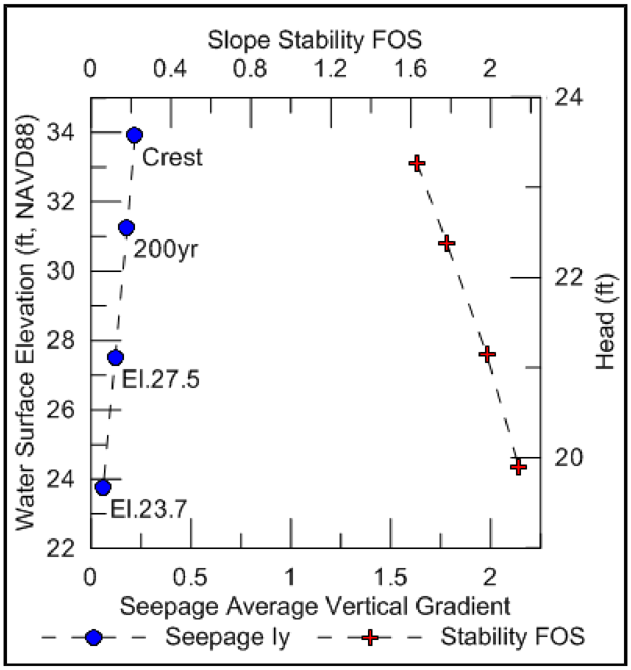


Figure 3-9: RD-17 Index Point LR-4 Without-Project Analyses Results

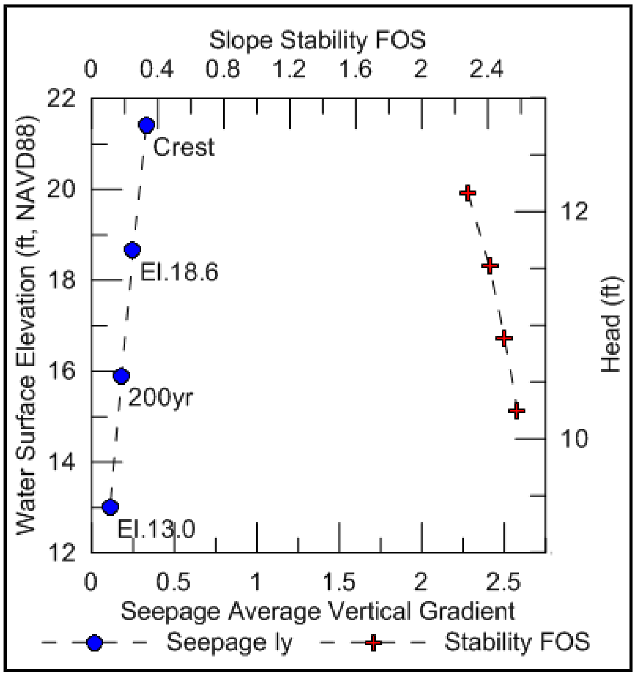


Figure 3-10: RD-17 Index Point FL-1 Without-Project Analyses Results

3.4.2 Central Stockton – RD-404 French Camp Slough/Stockton Diverting Canal, Left Bank Calaveras River

The without-project conditions analyses for central Stockton includes a total of five (5) index points; one (1) along the right (north) bank of French Camp Slough in RD-404, two (2) along the left (west) bank of the Stockton Diverting Canal, and tow (2) along the Left (south) bank of the Calaveras River. The index point locations for FR-1, SL-1, SL-2, CL-1/CL-2, and D-5, are shown in Figure 3.5. As the results show below, FR-1, SL-1 and SL-2 display exit gradients, and in some cases slope stability FOS, that do not meet design criteria at various water surface elevations. Figure 3-11 to Figure 3-15 displays steady state seepage and landside slope stability results for the analyzed water surface elevations.

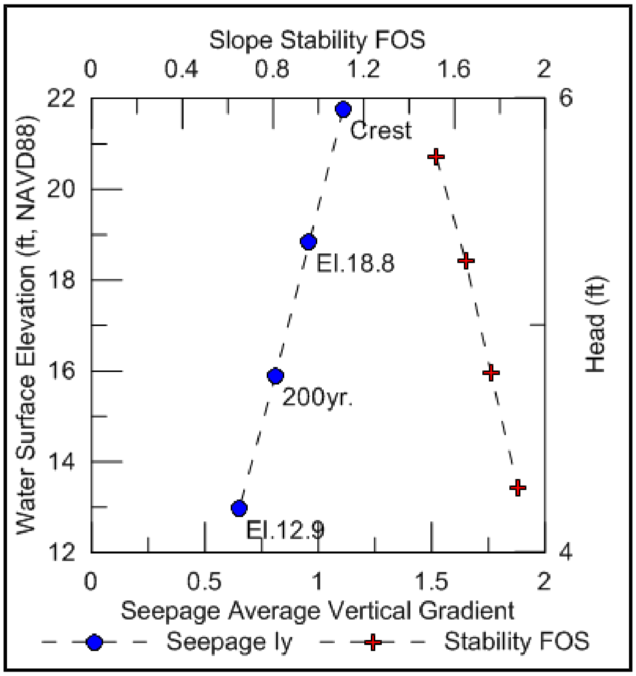


Figure 3-11: RD-404 Index Point FR-1 Without-Project Analyses Results

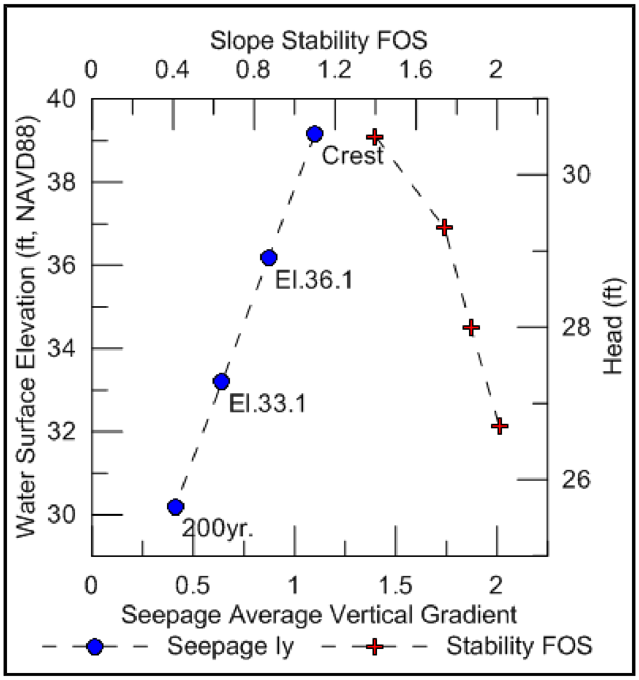


Figure 3-12: SDC Index Point SL-1 Without-Project Analyses Results

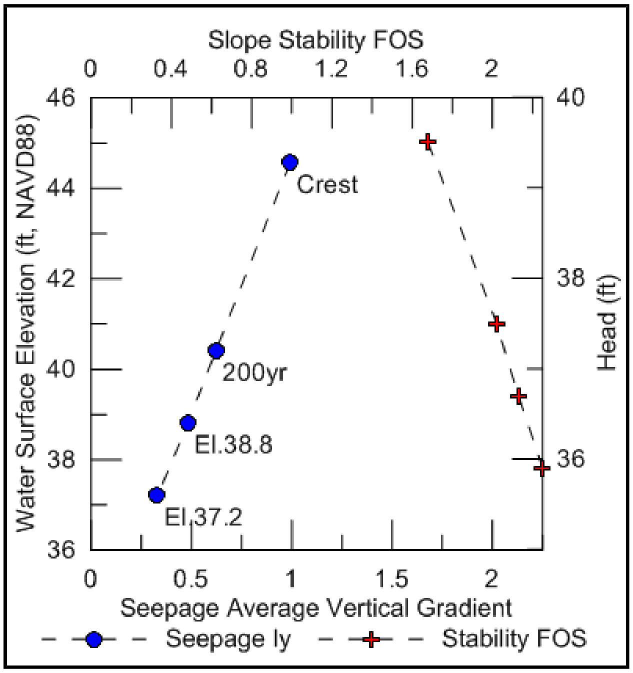


Figure 3-13: SDC Index Point SL-2 Without-Project Analyses Results

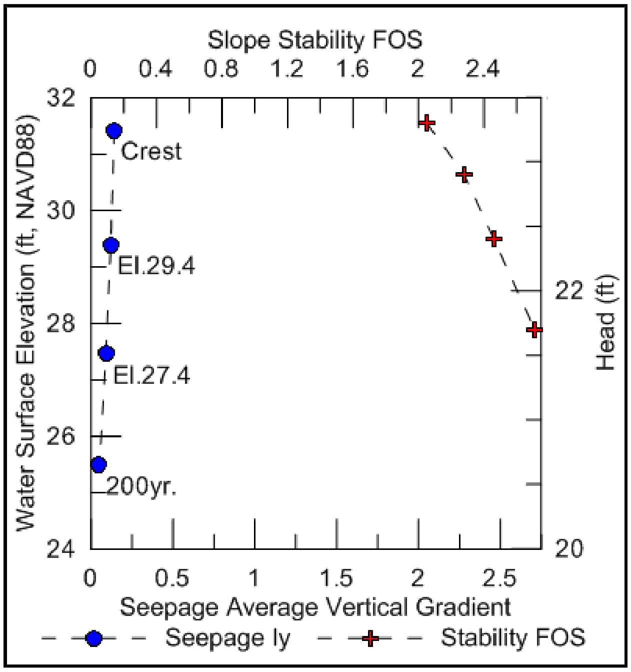


Figure 3-14: Calaveras River Index Point CL-1/CL-2 Without-Project Analyses Results

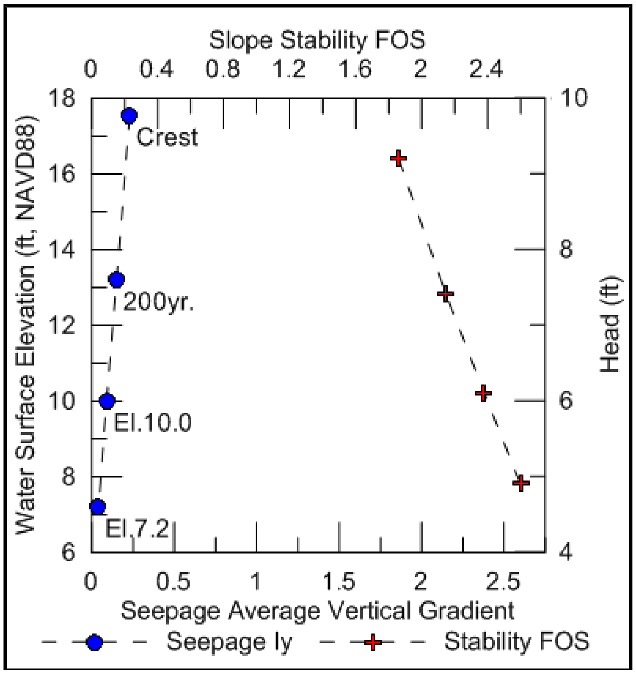


Figure 3-15: Calaveras River Index Point D-5 Without-Project Analyses Results

3.4.3 North Stockton – Right Bank Calaveras River, Delta Brookside, Delta Lincoln Village

The without-project conditions analyses for North Stockton includes a total of four (4) index points; two (2) along the right (north) bank of the Calaveras River, one (1) along the Delta Brookside Study Area, and one (1) along the Delta Lincoln Village Study Area. The index point locations for CR-1/CR-2, D-4, D-BS, and D-LV are shown in Figure 3.5. As the results show below, CR-1/CR-2, D-4, and D-BS display exit gradients, and in some cases slope stability FOS, that do not meet design criteria at various water surface elevations. Figure 3-16 to Figure 3-19 displays steady state seepage and landside slope stability results for the analyzed water surface elevations.

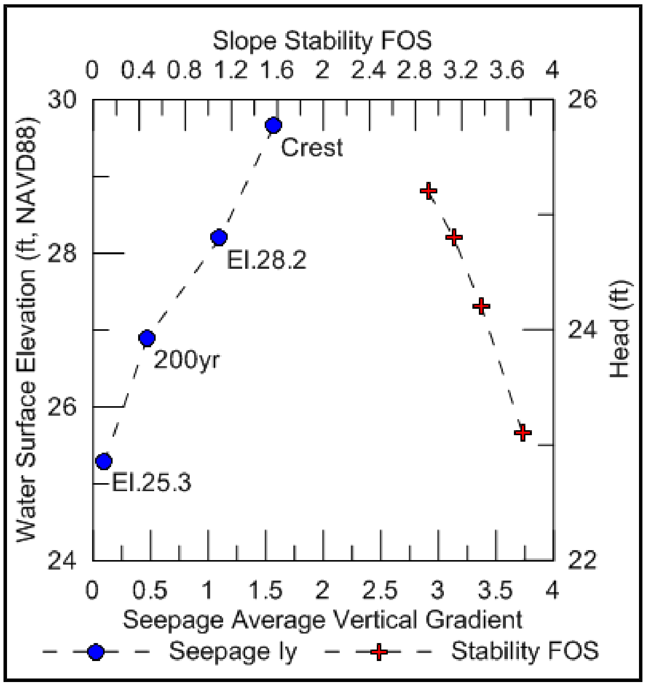


Figure 3-16: Calaveras River Index Point CR-1/CR-2 Without-Project Analyses Results

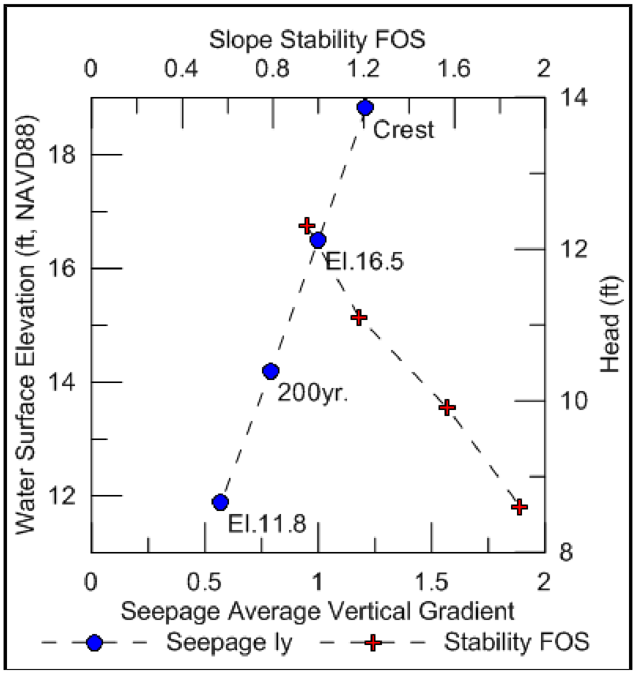


Figure 3-17: Calaveras River Index Point D-4 Without-Project Analyses Results

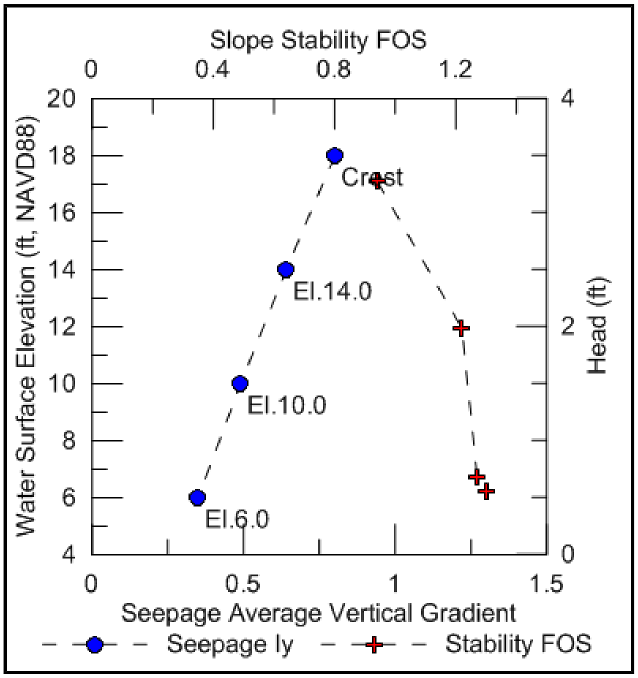


Figure 3-18: Delta Brookside Index Point D-BS Without-Project Analyses Results

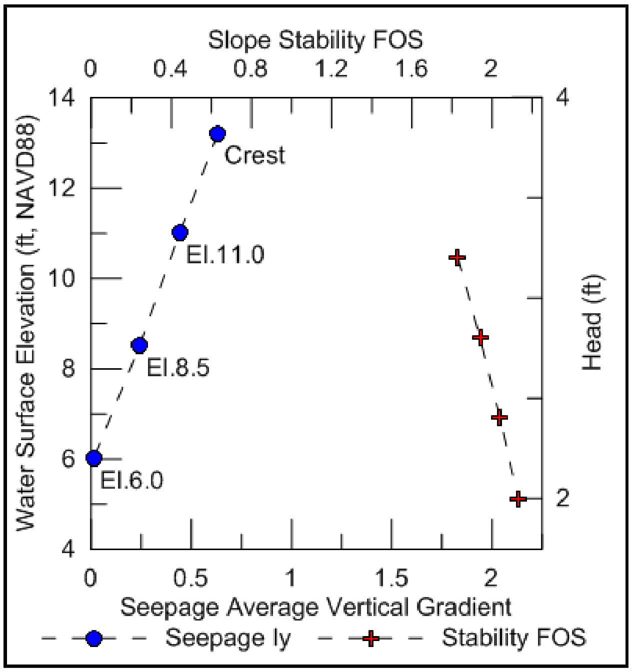


Figure 3-19: Delta Lincoln Village Index Point D-LV Without-Project Analyses Results

3.5 PROBABILISTIC ANALYSIS METHODOLOGY

Index points were selected for geotechnical analysis to represent the critical surface and subsurface conditions of each planning reach in order to identify the geotechnical deficiencies of the reach. The sections were selected based on previous geotechnical analysis, past levee performance, existing levee improvements, subsurface data, laboratory test results, surface conditions, field reconnaissance, and levee geometry. The ground surface elevations used in the cross-sections were based on the LiDAR and bathymetric surveys performed by URS, Kleinfelder, and Fugro for DWR from 2007 and 2008. The analysis model stratigraphy was interpreted based on existing boring logs near the index point.

The First-Order-Second-Moment (FOSM) method, as recommended in ETL 1110-2-556, “Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies,” dated 28 May 1999, was followed during the probabilistic evaluation of each index point. In this approach, the uncertainty in performance is taken to be a function of the uncertainty in model parameters. The standard deviations of a performance function were estimated based on the expected values (means) and the standard deviation of the random variable means. The performance functions considered were underseepage, through-seepage, and slope stability.

The final result of the FOSM method is a reliability index, Beta (β), representing the amount of standard deviation of the performance function by which the expected value exceeds the limit equilibrium state. The limit equilibrium state was defined using a FOS of 1.0. The standard deviation and variance of the performance function are calculated from the standard deviation and variance of the foundation and embankment parameters using the Taylor series method based on a Taylor series expansion of the performance function about the expected values. The partial derivatives were calculated numerically using an increment of plus and minus one standard deviation centered on the expected mean value. The variance of the performance function was obtained by summing the products of the partial derivatives of the performance function considering the variance of the corresponding parameters. The probability of poor performance, Pr(U), of the levee was expressed as a function of the river water elevation and the random variables of each performance function.

Potential sources of levee distress, or failure, considered in the analyses were underseepage through the levee foundation, through-seepage through the levee embankment, and instability of the landside levee slope under steady state conditions. The levees were evaluated against the above mentioned performance modes at five different water surface elevations. The loading conditions in most cases included the levee crest, levee crest minus three feet, half levee height, toe plus three feet, and landside levee toe where the probability of poor performance was considered to be zero. Using this method of selecting loading conditions, the levee performance curves would theoretically represent probability of poor performance at multiple flood frequencies.

Sudden drawdown conditions may result in levee slope failure; however, flooding is unlikely to occur when the water is at low elevation. Sudden drawdown was not considered in the analysis. Additionally, a judgment based conditional probability of poor performance curve is included in the risk and uncertainty analysis. This analysis considers: existing and past erosion history of the levee and riverbank, maintenance, encroachments, vegetation on the levee slopes and within the levee critical area, animal burrows and other external damaging conditions.

The probability of poor performance was evaluated by assessing the foundation and embankment materials and assigning values for the probability moments of the random variables considered in the analyses. Random variables for underseepage included the ratio of the horizontal permeability of the aquifer to the vertical permeability of the blanket, blanket thickness, and aquifer thickness. Random variables for through-seepage included critical tractive stress, porosity, and intrinsic permeability of the levee embankment material. Random variables for slope stability included effective friction angle, effective cohesion, and total unit weight of the levee embankment, and effective friction angle and cohesion of the foundation material.

3.5.1. Underseepage

Underseepage analysis was performed using blanket theory analysis (BTA) as described in ETL 1110-2-556, EM 1110-2-1913, and TM 3-424. Finite element analyses using the SEEP2D program, part of the GMS version 6.5 software package, were developed to independently check the blanket theory results. In general, the finite element and the empirical seepage calculations supported each other, predicting qualitatively similar results. Statistical analysis was used for each reach in determination of the coefficients of variation and standard deviation of the permeability ratios, blanket thickness, and thickness of the underlying aquifer. A critical gradient of 0.80 was used, considering 112 pounds per cubic foot (pcf) unit weight of the blanket. The unit weight of the blanket was considered the same at all index points. Values of vertical and horizontal permeability based on material classification and fines content are shown in Table 3-5 below (a reduced version of Table 3-2).

Table 3-5: Vertical and Horizontal Hydraulic Conductivity

Material Type	Soil Description	Hydraulic Conductivity				
		k_H (cm/sec)	k_H (ft/day)	k_H/k_V	k_V (cm/sec)	k_V (ft/day)
Clay	Blanket ≥10ft Thick	1.0E-05	0.028	4	2.5E-06	0.0071
	Blanket 5ft<>10ft Thick	1.0E-05	0.028	1	1.0E-05	0.028
	Blanket ≤5ft Thick	1.0E-05	0.028	0.1	1.0E-04	0.28
Silt	Elastic (plastic)	5.0E-05	0.14	4	1.3E-05	0.035
	Non-plastic	2.0E-04	0.57	4	5.0E-05	0.14
Clayey Sand to Sand	30-49% fines	5.0E-05	0.14	4	1.3E-05	0.035
	13-29% fines	1.0E-04	0.28	4	2.5E-05	0.071
	8-12% fines	1.0E-03	2.8	4	2.5E-04	0.71
	0-7% fines	5.0E-03	14	4	5.0E-04	3.5
Silty Sand to Sand	30-49% fines	5.0E-04	1.4	4	1.3E-04	0.35
	13-29% fines	1.0E-03	2.8	4	2.5E-04	0.71
	8-12% fines	5.0E-03	14	4	5.0E-04	3.5
	0-7% fines	1.0E-02	28	4	1.0E-03	7.1
Gravel	28-49% fines	4.0E-04	1.13	4	1.0E-04	0.28
	18-27% fines	1.0E-03	2.8	4	2.5E-04	0.71
	13-17% fines	6.0E-03	17	4	6.0E-04	4.3
	8-12% fines	1.2E-02	34	4	1.2E-03	8.5
	0-7% fines	2.5E-02	71	4	2.5E-3	17.8

3.5.2 Through-Seepage

Levees constructed either of fine grained clays, having stability berms with drainage layers that extend along the levee slope that captures seepage through the levee, or having cutoff walls constructed through the levee embankment are unlikely to be susceptible to through-seepage caused internal erosion. Levees of silt, silty sand, and sand were considered to be susceptible to internal erosion and were evaluated using the modified Khilar, Folger, and Gray erosion model as prescribed in ETL 1110-2-556. Using this method, the critical gradient through the levee embankment was calculated based on variations in the critical tractive stress, porosity, and intrinsic permeability of the levee material and compared with the predicted horizontal gradient through the levee embankment from the SEEP2D model. Table 3-6 shows the mean values of the random variables of the levee embankment material used to calculate the critical gradient were critical tractive stress (dynes/cm^2) which was taken as ten times the d_{50} (mm), the porosity based on material classification as proposed by Weight and Sonderegger in "Manual of Applied Field Hydrology", and intrinsic permeability was taken as approximately 1x10^{-5} times the horizontal permeability (cm/sec). Table 3-7 presents coefficients of variation for the through-seepage analysis random variables that were obtained using methodologies outlined in ETL 1110-2-556.

Table 3-6: Through-Seepage Random Variables

Material	Tractive Stress (dynes/cm^2)	Porosity (%)	Intrinsic Permeability (cm^2)
Clay	5 – 50	40 - 70	1.0E^{-10}
Silt	0.5 – 50	35 - 50	2.0E^{-9} – 5.0E^{-10}
Sand	1 – 20	25 - 50	1.0E^{-6} – 5.0E^{-9}
Gravel	15 – 250	20 - 40	2.5.0E^{-6} – 4.0E^{-9}
Sand and Gravel	15 – 250	15 - 35	

Table 3-7: Variation of Through-Seepage Random Variables

Random Variable	Coefficient of Variation (%)
Critical Tractive Stress (T_c dynes/cm^2)	10
Porosity (n)	10
Intrinsic Permeability (k_o cm^2)	30

3.5.3 Landside Slope Stability

The cases analyzed for stability risk analyses considered long-term conditions with steady state seepage along the landside slope of the levee. The phreatic surface and pore water pressures were developed for the steady state condition using the SEEP2D finite element computer program developed as part of GMS version 6.5. The limit equilibrium computer program UTEXAS4 was used to perform the stability analyses. Circular failure surfaces were assumed and the embankment was modeled as homogeneous. All analyses consisted of running a search routine to identify the critical failure surface using the Spencer's Method.

A sensitivity study was done to determine which parameters in the slope stability calculations were most influential. For this study, those variables are soil strength and unit weights of the soil in the levee embankment and soil strength in the foundation. Statistical descriptors for these variables were determined using available site-specific information and published statistical data. The piezometric lines or pore water pressures for each water elevation were determined using the finite element program SEEP2D for the levee embankment and its foundation.

The drained soil strength parameters used in the stability analyses are shown in Table 3-8; these values were based on a generalized conservative assumption of shear strength by soil type from previous studies in the project area. For each index point the generalized assumption was compared with available field and laboratory testing from nearby explorations. The coefficients of variation for soil strength parameters and unit weight of the fill material in the levee or the top impervious blanket are shown in Table 3-9 and were obtained using methodologies outlined in ETL 1110-2-556, and those proposed by Harr in the "Reliability-Based Design in Civil Engineering", and Duncan in the "Manual for Geotechnical Engineering Reliability Calculations".

Table 3-8: Drained Shear Strength of Soil

Material Type	Soil Description	Shear Strength		
		c' (psf)	Φ' (^{o})	γ (pcf)
Cutoff Wall	SCB, SB, CB	50	0	85
Clay	CH Levee Embankment	100	22	115
	CH Foundation	100	26	115
	CL Levee Embankment	50	24	115
	CL Foundation	50	28	115
Silt	ML Levee Embankment-	0	28	115
	ML Foundation	0	30	120
Clayey Sand and Silty Sand	-	0	33	125
Sand	-	0	35	130
Gravel and Drain Rock	-	0	35	135

Table 3-9: Variation of Drained Shear Strength Parameters

Random Variable	Coefficient of Variation (%)
Effective Friction Angle (Φ)	13
Effective Cohesion (c psf)	40
Total Unit Weight (γ pcf)	7

3.5.4 Judgment

A judgment based conditional probability function was based on the existing and past erosion history of the levee and riverbank, maintenance, encroachments, vegetation on the levee slopes and within the levee critical area, animal burrows and other external damaging conditions. Generally, past experience with poor performance at utility crossing and rodent activity indicates the risk of failure is somewhat significant in the analyzed areas. The judgment based curve is included for each analyzed levee cross section and in the combined curve of failure.

In June 2009, an expert elicitation was conducted for the purpose of developing the geotechnical judgment portion of the curves; the meeting minutes are included as Enclosure 6. This expert elicitation was conducted in accordance with ETL 1110-2-561, “Appendix E, Expert Elicitation in Geological and Geotechnical Applications” 31 January 2006. The members of the expert elicitation team were highly recognized professional specialists in erosion and geotechnical issues. The expert elicitation focused on the judgment part of the geotechnical risk and uncertainty curves for flood control structures; the team discussed and reached consensus on the impact of different factors of the judgment curve, such as:

- the vegetation on the levees and within the levee right of way
- penetrations through the levee and foundation

- encroachments into the levee and levee right-of-way
- erosion of the riverbank and waterside slopes of the levee
- animal burrows

The expert elicitation also concluded that up to a certain water elevation, the risk of poor performance as determined by analyses or considered in the judgment portion of the curves may not necessarily coincide with the risk of failure. Based on historical performances of levees, it appears the risk of failure as determined in the analyses may be conservative and the poor performance of a levee may not lead to a catastrophic levee failure; even if distresses of the levee embankment needed to be repaired after a flood to bring the levee to the pre-flood performance. Consequently, the risk of catastrophic failure may be reduced based on the historical past performance, and consequently the curves may be altered.

3.5.5 Combined Curves

The total conditional probability of poor performance as a function of floodwater elevation has been developed by combining the probability of poor performance functions for four failure modes; underseepage, through-seepage, slope instability, and judgment.

3.6 PERFORMANCE CURVES

The results of the geotechnical risk and uncertainty analyses are briefly discussed in the following sections. As previously discussed, underseepage, through-seepage, and slope stability probabilities of failure were calculated analytically based on site specific subsurface information used to select material parameters and coefficients of variation. Included as Enclosure 3 are the spreadsheet analyses used to calculate the probabilities of poor performance. These spreadsheets include data from borings used to select parameters, the selected parameters, and the calculated results including the combined performance curve. The judgment curve remains as the non analytical component to the curve; those probabilities of poor performance were based on site specific conditions regarding vegetation, penetrations, encroachments, erosion, and animal burrows. The Reach Description section (Section 3.2) of this report describes in general terms the levee conditions regarding vegetation, penetrations, encroachments, and animal burrows.

3.6.1 RD-17 – Lower San Joaquin River, East Bank

The subsurface explorations for LR-1 used in probabilistic analyses resulted in a mean blanket thickness of 13.0-ft with a coefficient of variation of 46, and a mean aquifer thickness of 28.0-ft with a coefficient of variation 0.57. The levee embankment was comprised of lean clay and silt. The blanket was comprised of predominantly lean clay. The aquifer was comprised of poorly graded sand with silt to silty sand. Past performance indicates the embankment has experienced seepage, sand boils, and cracking.

The underseepage and judgment component curves accounted for the majority of the combined without-project curve. The judgment curve contributed 24.3% and the underseepage curve contributed 49.0% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of encroachments, animal burrows, and utilities. Figure 3-20 presents the without-project conditions combined curve for LR-1.

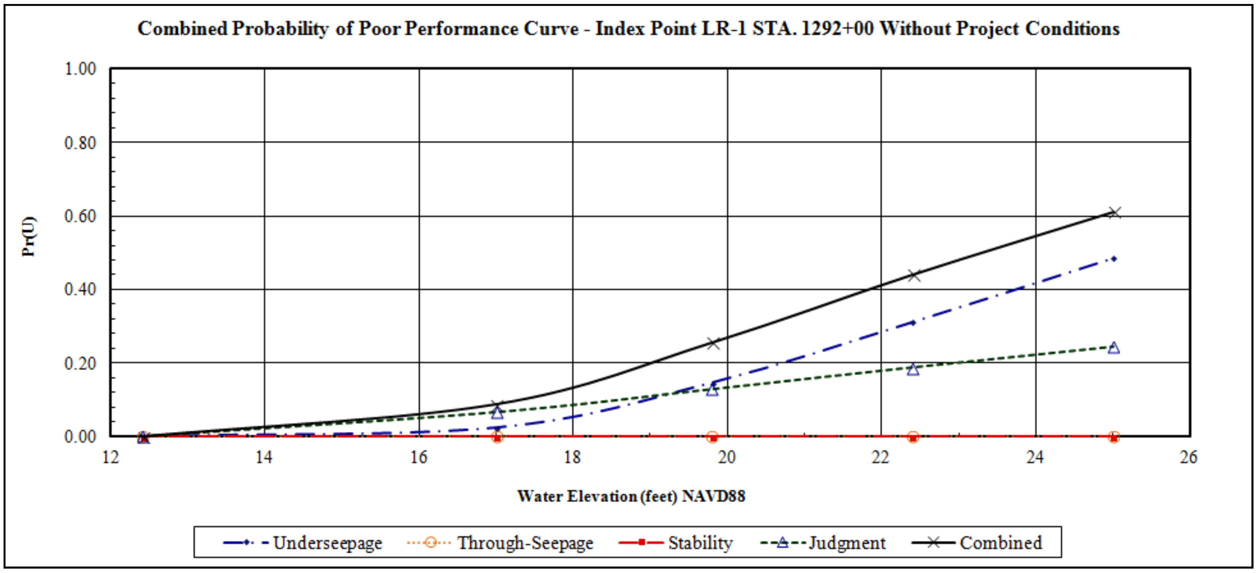


Figure 3-20: Index Point LR-1 Combined Probability of Poor Performance Curve for Without Project Conditions

The subsurface explorations for LR-2 used in probabilistic analyses resulted in a mean blanket thickness of 7.0-ft with a coefficient of variation of 57, and a mean aquifer thickness of 18.0-ft with a coefficient of variation 0.39. The levee embankment was comprised of lean clay to silty sand. The blanket was comprised of predominantly silty sand. The aquifer was comprised of poorly graded sand with silt to silty sand. Past performance indicates the embankment has experienced seepage, and sand boils.

The underseepage and judgment component curves accounted for the majority of the combined without-project curve. The judgment curve contributed 28.2% and the underseepage curve contributed 56.9% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of encroachments, vegetation, utilities, and animal burrows. Figure 3-21 presents the without-project conditions combined curve for LR-2.

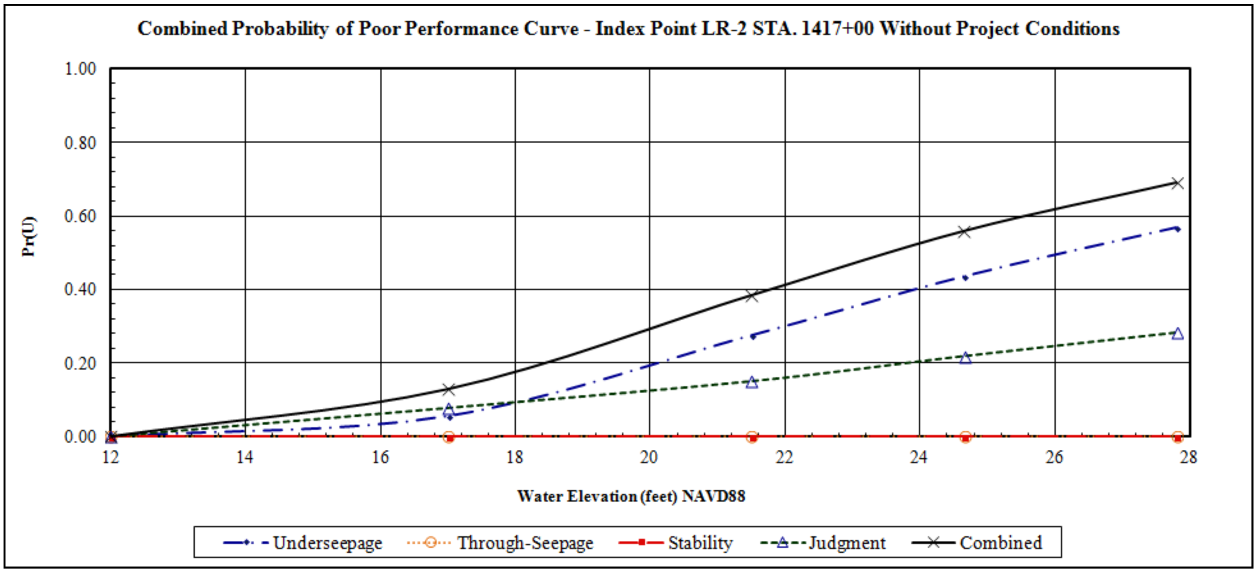


Figure 3-21: Index Point LR-2 Combined Probability of Poor Performance Curve for Without Project Conditions

The subsurface explorations for LR-3 used in probabilistic analyses resulted in a mean blanket thickness of 11.0-ft with a coefficient of variation of 55, and a mean aquifer thickness of 35.0-ft with a coefficient of variation 0.34. The levee embankment was comprised of poorly graded sand with silt to clayey sand. The thin blanket was comprised of predominantly lean clay to silty sand. The aquifer was comprised of poorly graded sand with silt. Past performance indicates the embankment has experienced seepage, sand boils, and breach conditions.

The underseepage, through-seepage, slope stability, and judgment component curves accounted for the majority of the combined without-project curve. The judgment curve contributed 20.2%, the underseepage curve contributed 48.6%, the through-seepage curve contributed 68.0%, and the slope stability curve contributed 99.9% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of vegetation, utilities, and animal burrows. Figure 3-22 presents the without-project conditions combined curve for LR-3.

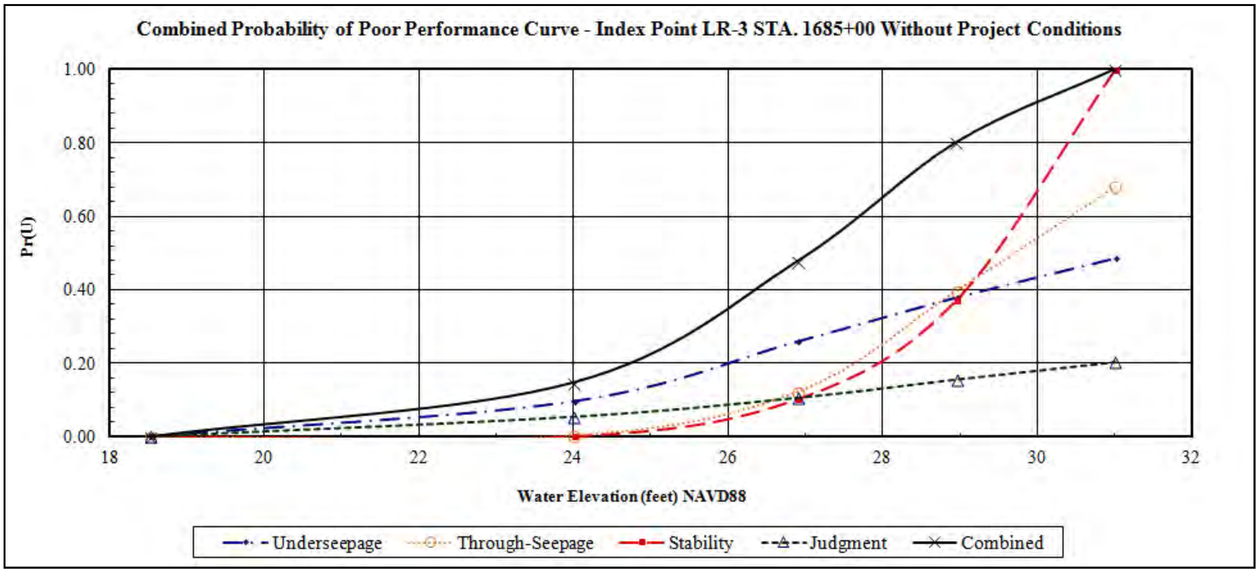


Figure 3-22: Index Point LR-3 Combined Probability of Poor Performance Curve for Without Project Conditions

The subsurface explorations for LR-4 used in probabilistic analyses resulted in a mean blanket thickness of 23.0-ft with a coefficient of variation of 13, and a mean aquifer thickness of 33.0-ft with a coefficient of variation 0.24. The levee embankment was comprised of clayey sand. The blanket was comprised of lean clay to sandy lean clay. The aquifer was comprised of poorly graded sand with silt. Past performance indicates the embankment has experienced seepage, and sand boils.

The judgment component curve accounted for the majority of the combined without-project curve. The judgment curve contributed 22.7% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of vegetation, encroachments, and animal burrows. Figure 3-23 presents the without-project conditions combined curve for LR-4.

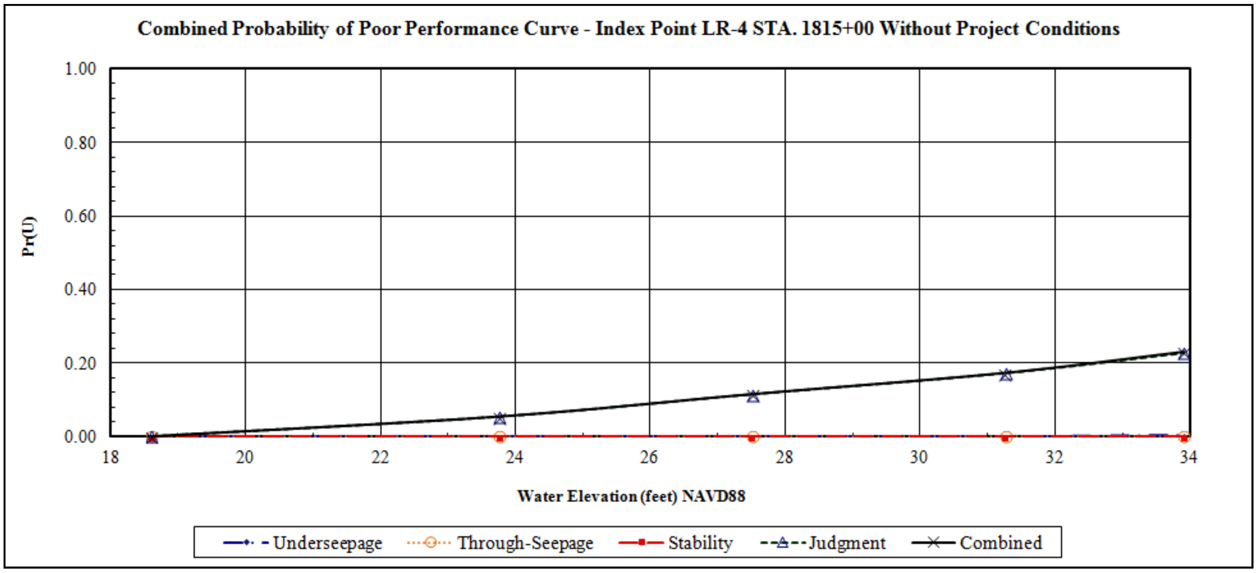


Figure 3-23: Index Point LR-4 Combined Probability of Poor Performance Curve for Without Project Conditions

3.6.2 French Camp Slough, North and South Bank

The subsurface explorations for FL-1 used in probabilistic analyses resulted in a mean blanket thickness of 10.0-ft with a coefficient of variation of 10, and a mean aquifer thickness of 9.0-ft with a coefficient of variation 0.67. The levee embankment was comprised of sandy clay. The blanket was comprised of lean clay to sandy lean clay. The aquifer was comprised of silty sand. Past performance indicates the embankment has experienced seepage, and sand boils.

The judgment component curve accounted for the majority of the combined without-project curve. The judgment curve contributed 23.5% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of vegetation, encroachments, and animal burrows. Figure 3-24 presents the without-project conditions combined curve for FL-1.

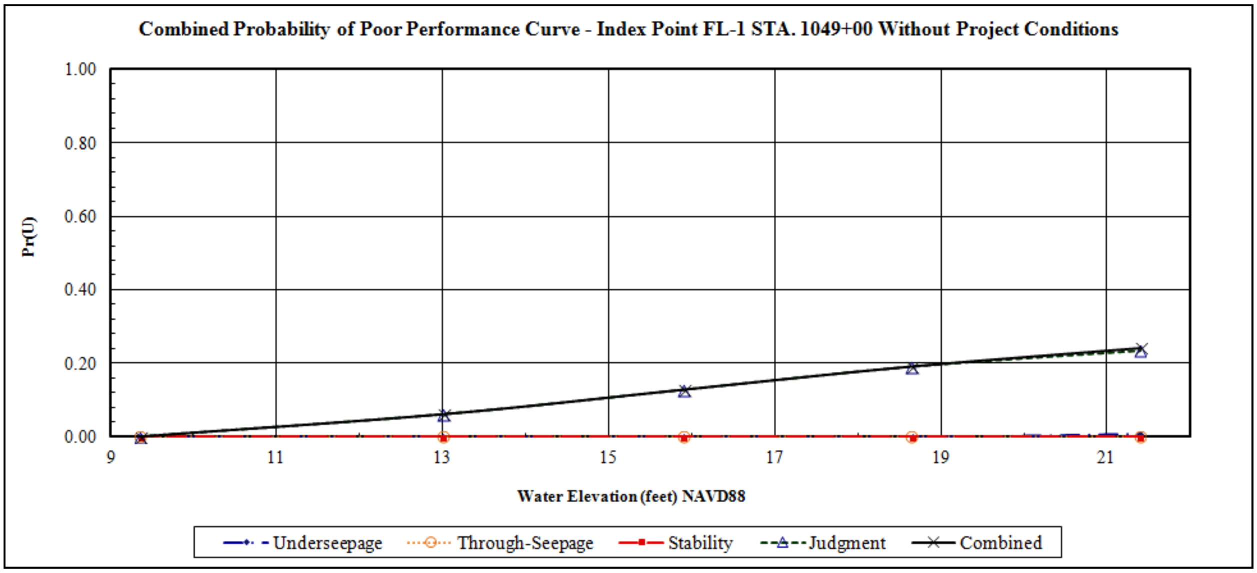


Figure 3-24: Index Point FL-1 Combined Probability of Poor Performance Curve for Without Project Conditions

The subsurface explorations for FR-1 used in probabilistic analyses resulted in a mean blanket thickness of 7.0-ft with a coefficient of variation of 29, and a mean aquifer thickness of 8.0-ft with a coefficient of variation 0.25. The levee embankment was comprised of lean clay and silt. The thin blanket was comprised of predominantly clayey sand. The aquifer was comprised of silty sand. Past performance indicates the embankment has experienced seepage, sand boils, and bank erosion.

The underseepage and judgment component curves accounted for the majority of the combined without-project curve. The judgment curve contributed 21.9% and the underseepage curve contributed 63.9% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of encroachments, vegetation, animal burrows, and erosion. Figure 3-25 presents the without-project conditions combined curve for FR-1.

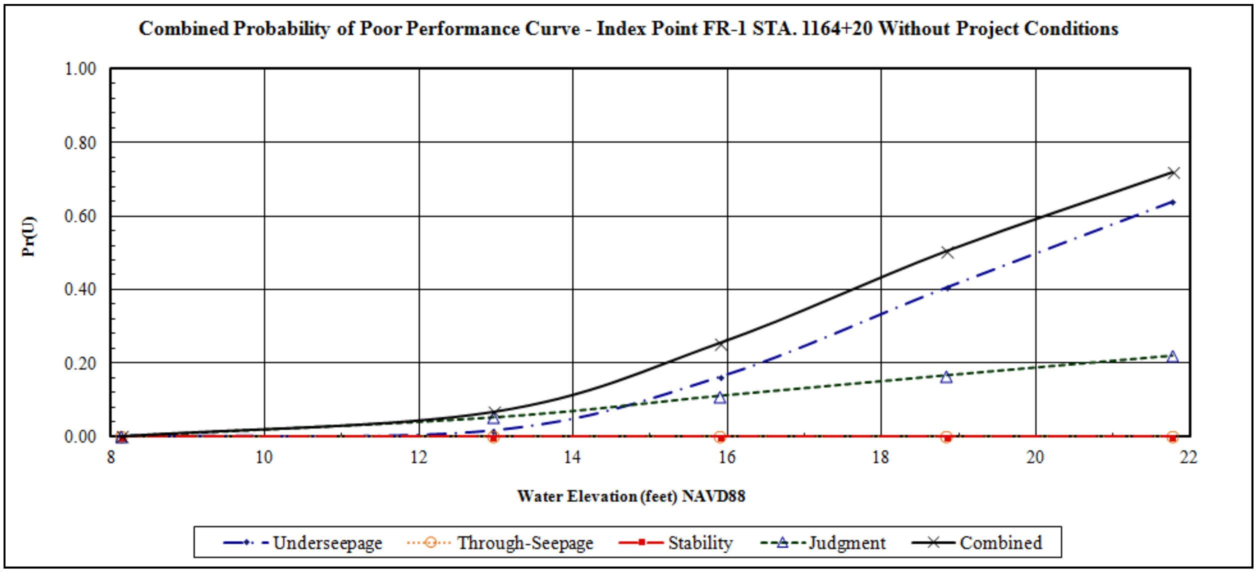


Figure 3-25: Index Point FR-1 Combined Probability of Poor Performance Curve for Without Project Conditions

3.6.3 Stockton Diverting Canal, Left Bank

The subsurface explorations for SL-1 used in probabilistic analyses resulted in a mean blanket thickness of 10.0-ft with a coefficient of variation of 50, and a mean aquifer thickness of 17.0-ft with a coefficient of variation 0.65. The levee embankment was comprised of sandy lean clay. The thin blanket was comprised of predominantly lean clay. The aquifer was comprised of silty sand. Past performance indicates the embankment has experienced no known issues with seepage or stability.

The underseepage and judgment component curves accounted for the majority of the combined without-project curve. The judgment curve contributed 19.3% and the underseepage curve contributed 30.9% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of animal burrows, encroachments, and utilities. Figure 3-26 presents the without-project conditions combined curve for SL-1.

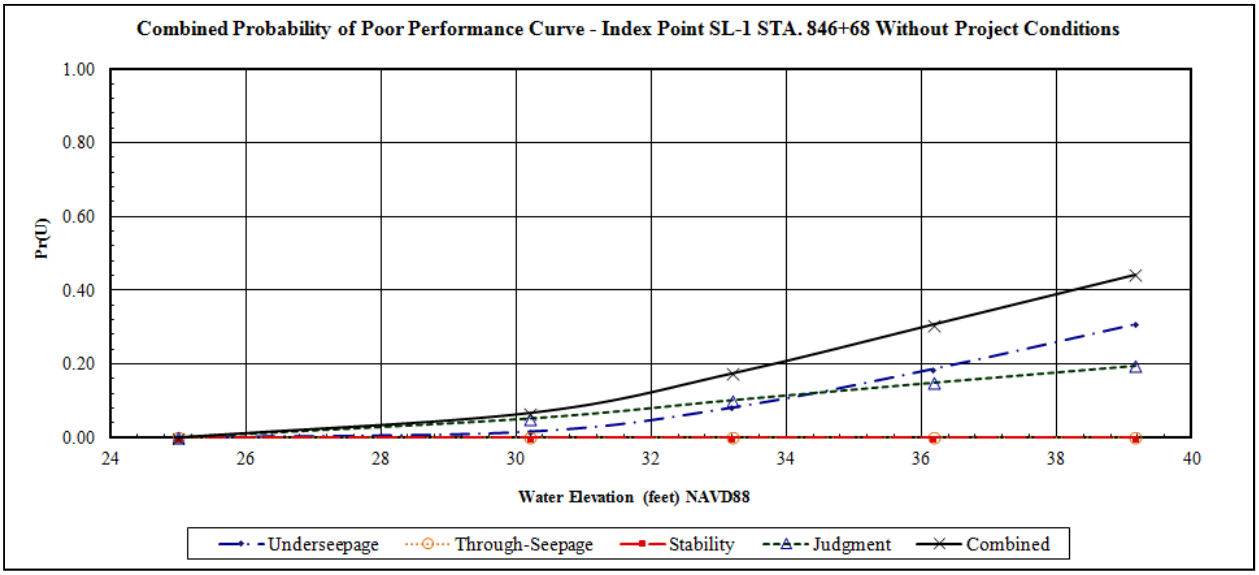


Figure 3-26: Index Point SL-1 Combined Probability of Poor Performance Curve for Without Project Conditions

The subsurface explorations for SL-2 used in probabilistic analyses resulted in a mean blanket thickness of 7.0-ft with a coefficient of variation of 29, and a mean aquifer thickness of 10.0-ft with a coefficient of variation 0.60. The levee embankment was comprised of sandy silt. The blanket was comprised of predominantly lean clay. The aquifer was comprised of silty sand. Past performance indicates the embankment has experienced no known issues with seepage or stability.

The underseepage and judgment component curves accounted for the majority of the combined without-project curve. The judgment curve contributed 19.3% and the underseepage curve contributed 22.4% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of animal burrows, encroachments, and utilities. Figure 3-27 presents the without-project conditions combined curve for SL-2.

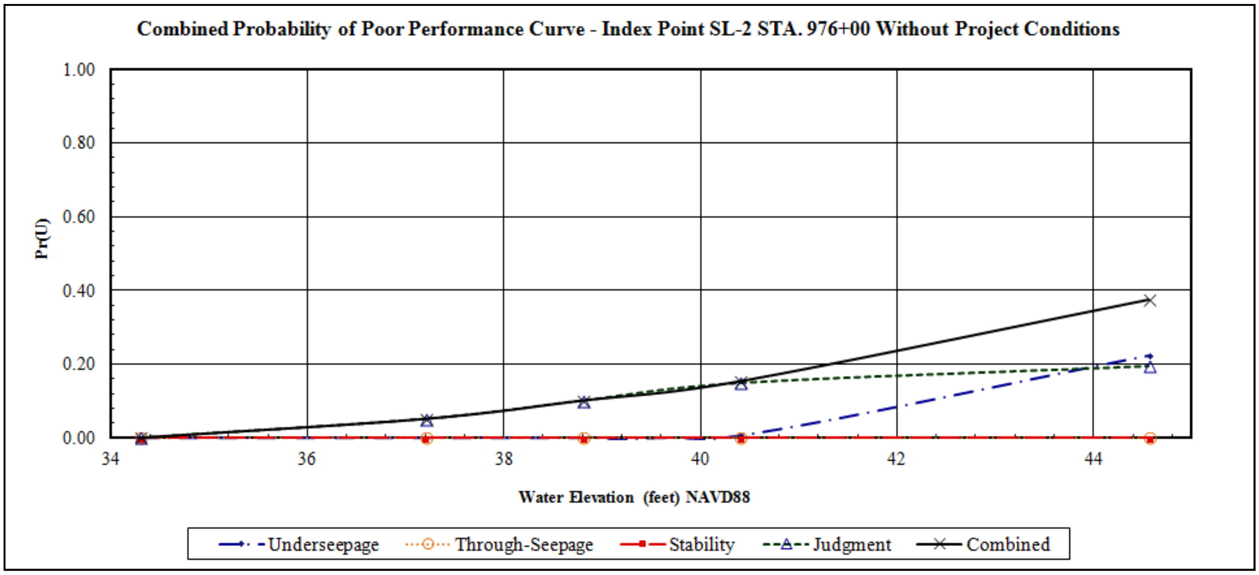


Figure 3-27: Index Point SL-2 Combined Probability of Poor Performance Curve for Without Project Conditions

3.6.4 Calaveras River, North and South Bank

The subsurface explorations for CL-1/CL-2 used in probabilistic analyses resulted in a mean blanket thickness of 19.0-ft with a coefficient of variation of 42, and a mean aquifer thickness of 15.0-ft with a coefficient of variation 0.73. The levee embankment was comprised of sandy silt. The blanket was comprised of predominantly elastic silt. The aquifer was deep and appeared in a few borings as poorly graded sand with silt. Past performance indicates the embankment has experienced seepage, settlement, and bank erosion.

The through-seepage and judgment component curves accounted for the majority of the combined without-project curve. The judgment curve contributed 32.7% and the through-seepage curve contributed 7.7% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of encroachments, erosion, animal burrows, utilities, and vegetation. Figure 3-28 presents the without-project conditions combined curve for CL-1/CL-2.

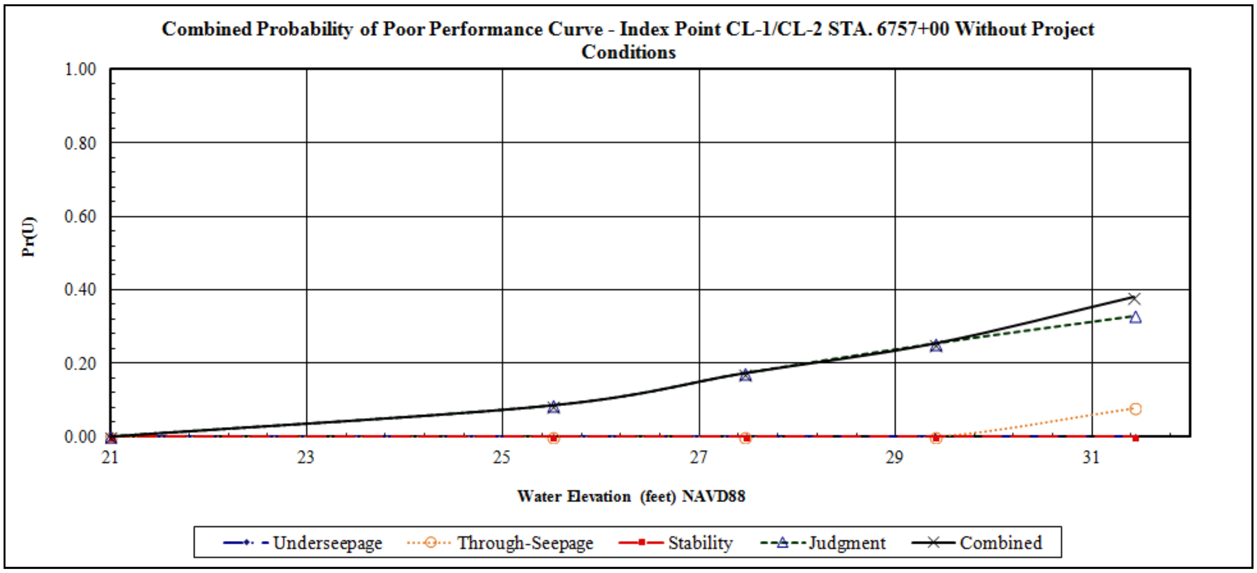


Figure 3-28: Index Point CL-1/CL-2 Combined Probability of Poor Performance Curve for Without Project Conditions

The subsurface explorations for CR-1/CR-2 used in probabilistic analyses resulted in a mean blanket thickness of 5.0-ft with a coefficient of variation of 40, and a mean aquifer thickness of 14.0-ft with a coefficient of variation 0.57. The levee embankment was comprised of sandy lean clay. The thin blanket was comprised of predominantly sandy lean clay. The aquifer was comprised of sandy silt. Past performance indicates the embankment has experienced seepage, settlement, and bank erosion.

The underseepage and judgment component curves accounted for the majority of the combined without-project curve. The judgment curve contributed 32.0% and the underseepage curve contributed 24.4% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of encroachments, utilities, erosion, and vegetation. Figure 3-29 presents the without-project conditions combined curve for CR-1/CR-2.

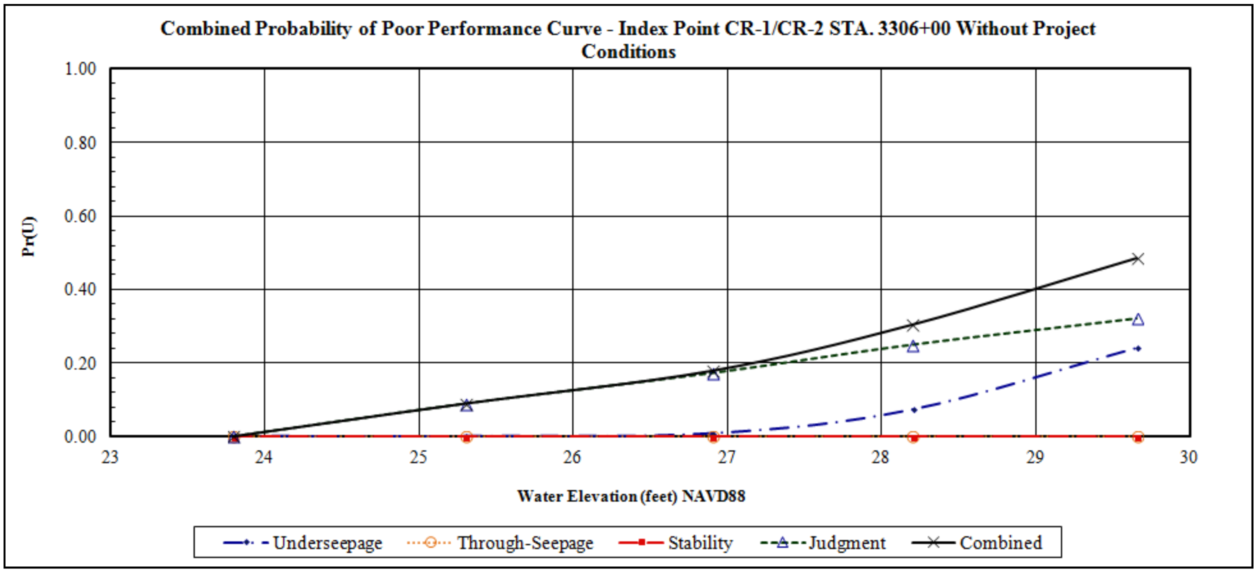


Figure 3-29: Index Point CR-1/CR-2 Combined Probability of Poor Performance Curve for Without Project Conditions

The subsurface explorations for D-4 used in probabilistic analyses resulted in a mean blanket thickness of 15.0-ft with a coefficient of variation of 47, and a mean aquifer thickness of 30.0-ft with a coefficient of variation 0.07. The levee embankment was comprised of sandy silt and sandy lean clay. The thin blanket was comprised of predominantly sandy fat clay to sandy silt. The aquifer was comprised of poorly graded sand with silt. Past performance indicates the embankment has experienced seepage, settlement, sand boils, and bank erosion.

The underseepage, through-seepage, slope stability, and judgment component curves accounted for the majority of the combined without-project curve. The judgment curve contributed 30.5%, the underseepage curve contributed 37.4%, the through-seepage curve contributed 8.5%, and the slope stability curve contributed 66.9% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of encroachments, vegetation, utilities, and animal burrows. Figure 3-30 presents the without-project conditions combined curve for D-4.

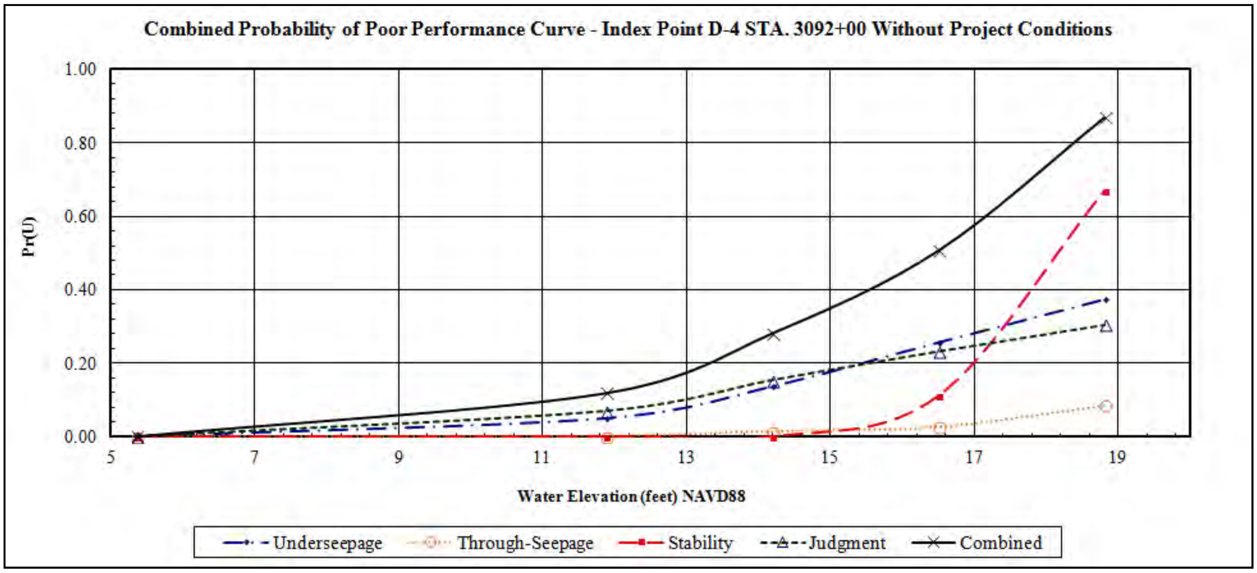


Figure 3-30: Index Point D-4 Combined Probability of Poor Performance Curve for Without Project Conditions

The subsurface explorations for D-5 used in probabilistic analyses resulted in a mean blanket thickness of 20.0-ft with a coefficient of variation of 45, and a mean aquifer thickness of 15.0-ft with a coefficient of variation 0.67. The levee embankment was comprised of silt. The blanket was comprised of predominantly lean clay. The aquifer was comprised of silty sand. Past performance indicates the embankment has experienced seepage, settlement, and bank erosion.

The through-seepage and judgment component curves accounted for the majority of the combined without-project curve. The judgment curve contributed 31.2% and the through-seepage curve contributed 12.8% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of encroachments, erosion, utilities, and vegetation. Figure 3-31 presents the without-project conditions combined curve for D-5.

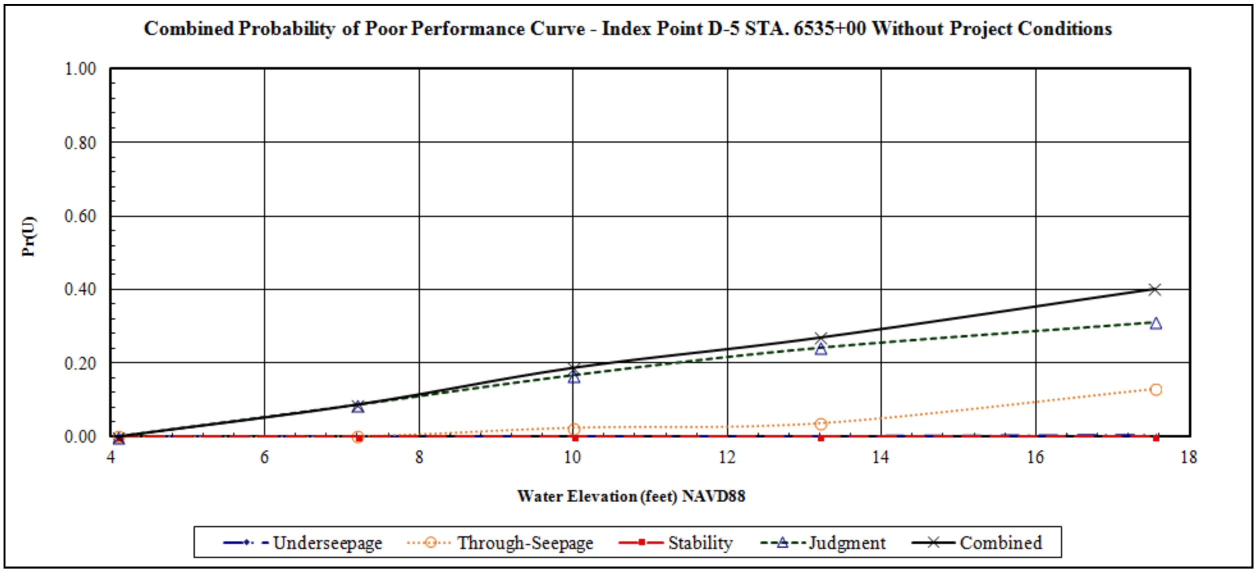


Figure 3-31: Index Point D-5 Combined Probability of Poor Performance Curve for Without Project Conditions

3.6.5 Delta Front Brookside / Delta Lincoln Village

The subsurface explorations for D-BS used in probabilistic analyses resulted in a mean blanket thickness of 18.0-ft with a coefficient of variation of 33, and a mean aquifer thickness of 20.0-ft with a coefficient of variation 0.45. The levee embankment was comprised of lean clay and portions of an older levee constructed of organic clay. The thin blanket was comprised of predominantly organic clay and lean clay. The aquifer was comprised of silty sand. Past performance indicates the embankment has experienced seepage, settlement, bank erosion, and animal burrows.

The underseepage, slope stability, and judgment component curves accounted for the majority of the combined without-project curve. The judgment curve contributed 25.9%, the underseepage curve contributed 41.8%, and the slope stability curve contributed 65.9% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of encroachments, erosion, vegetation, and utilities. Figure 3-32 presents the without-project conditions combined curve for D-BS.

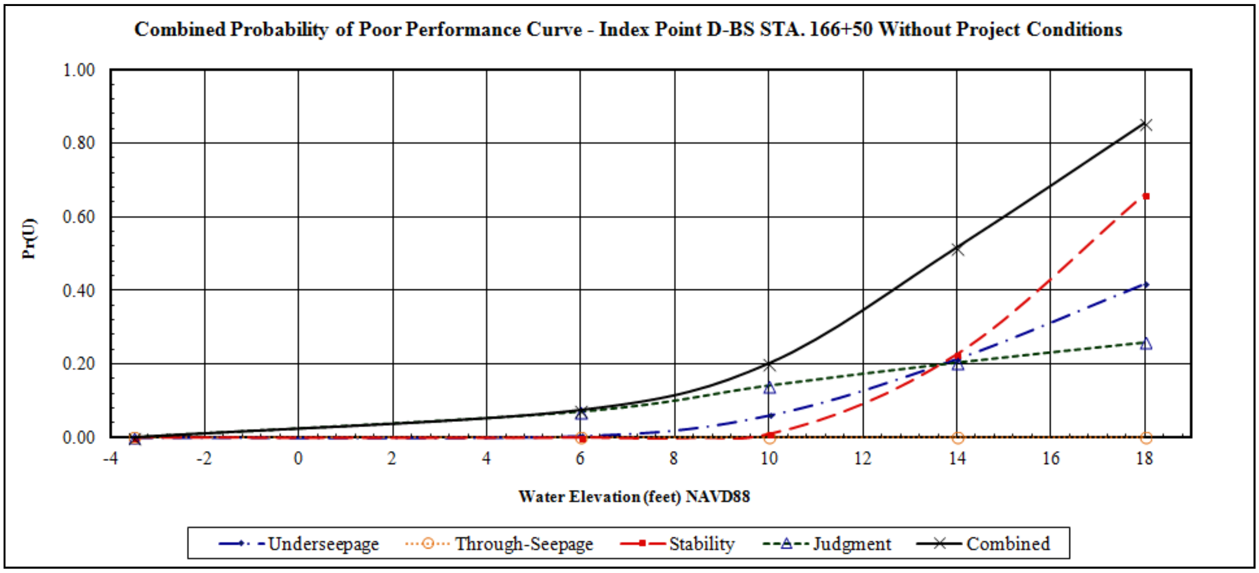


Figure 3-32: Index Point D-BS Combined Probability of Poor Performance Curve for Without Project Conditions

The subsurface explorations for D-LV used in probabilistic analyses resulted in a mean blanket thickness of 12.0-ft with a coefficient of variation of 58, and a mean aquifer thickness of 21.0-ft with a coefficient of variation 0.43. The levee embankment was comprised of lean clay. The thin blanket was comprised of predominantly lean clay. The deep aquifer was comprised of silty sand to poorly graded sand with silt. Past performance indicates the embankment has experienced seepage, and bank erosion.

The underseepage and judgment component curves accounted for the majority of the combined without-project curve. The judgment curve contributed 29.8% and the underseepage curve contributed 23.0% to the combined without-project curve at the levee crest WSE. The without-project judgment curve was primarily comprised of encroachments, vegetation, utilities, and animal burrows. Figure 3-33 presents the without-project conditions combined curve for D-LV.

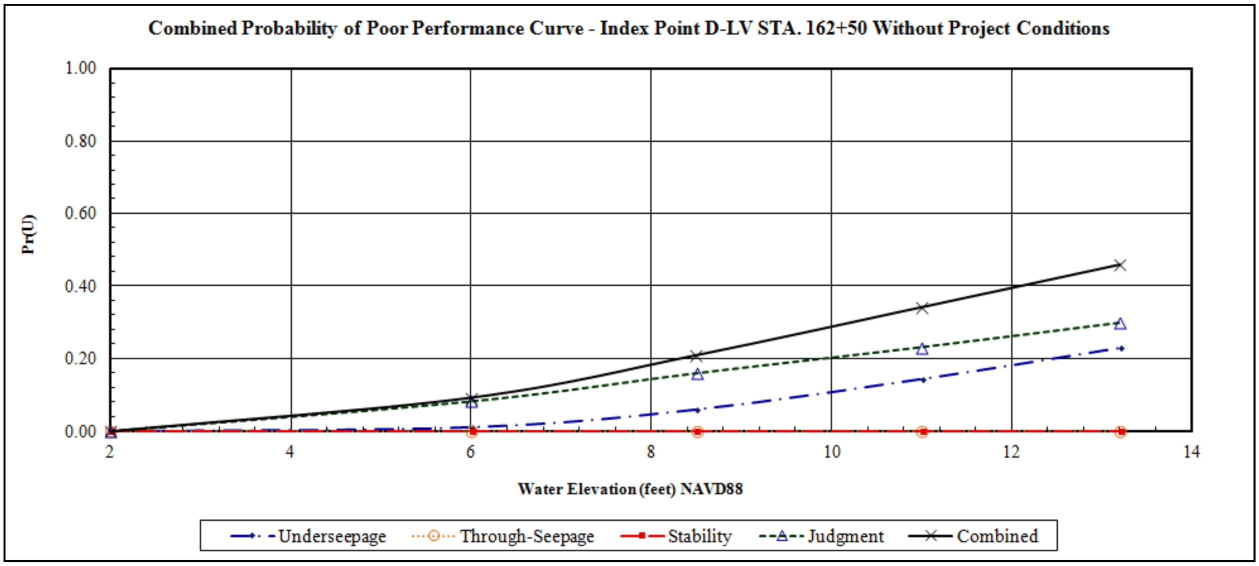


Figure 3-33: Index Point D-LV Combined Probability of Poor Performance Curve for Without Project Conditions

3.7 SEISMIC PERFORMANCE AND LIQUEFACTION ANALYSIS

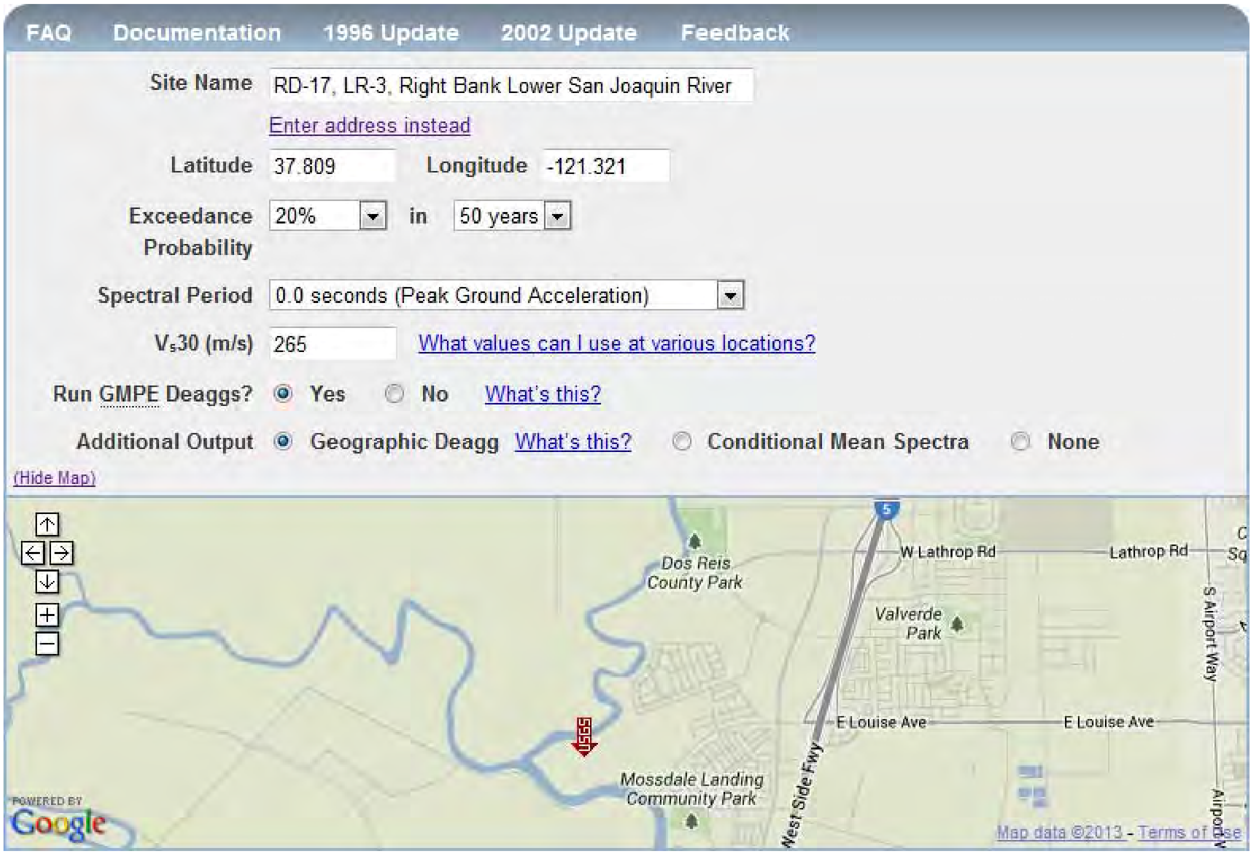
The main purpose of seismic vulnerability analyses was to identify the potential seismic performance of a levee. Major concerns during and after seismic events are liquefaction induced settlement and displacement, transverse cracks that may develop between liquefiable and non-liquefiable reaches, and at locations where liquefiable zones abut appurtenant structures with deep rigid foundations. Such zones should be identified and given special consideration.

3.7.1 Site Specific Seismic and Liquefaction Analysis

To evaluate the potential to liquefaction resistance of soils, liquefaction triggering analysis was performed based on a procedure from the summary report of the 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/National Science Foundation (NSF) Workshops on Evaluation of Liquefaction Resistance of Soils, published as part of the Journal of Geotechnical and Geoenvironmental Engineer, dated October 2001 (Youd, Idriss, Andrus, & Arango, October 2001).

Probabilistic Seismic Hazard Analysis (PSHA) based on the 2008 Next Generation Attenuation (NGA) relationships was used to develop seismic parameters for the LSJRFS area. The deaggregations are from the United States Geologic Society (USGS) developed 2008 Interactive Deaggregations web program. Figure 3-34 and Figure 3-35 presents an example of the interactive input screen and obtained results for index point LR-3 within the LSJRFS area. The following data were input:

- location, through latitude and longitude (up to three decimals each are considered)
- exceedance probability of the seismic event
- desired spectral period
- shear wave velocity of the upper 30 meters (V_S30) of the site



Site Name: RD-17, LR-3, Right Bank Lower San Joaquin River

Latitude: 37.809 Longitude: -121.321

Exceedance Probability: 20% in 50 years

Spectral Period: 0.0 seconds (Peak Ground Acceleration)

V_s30 (m/s): 265

Run GMPE Deaggs? ● Yes ○ No

Additional Output: ● Geographic Deagg ○ Conditional Mean Spectra ○ None

Figure 3-34: USGS 2008 Interactive Deaggregations (Beta) Input

The peak horizontal ground horizontal acceleration (PGA) for 20% exceedance in 50 years (224 -year average return period) at index point LR-3 was found to be 0.49g. The 20% probability of exceedance in 50 years (or 224 year average return period) was used in this study to be consistent with flood protection, per DWR. Seismic design is assumed to be based on ground motion probabilities that are equivalent to the high-water event exceedance probabilities that the project will be designed to withstand. For example, the project is expected to be designed for a 200-year high-water event, the expected seismic criteria is based on designing for the 200-year event. V_S30 was estimated as an average from several deep borings in the area through correlation with SPT blow counts. Figure 3-35 shows the peak horizontal ground acceleration and the contributions of various seismic sources based on USGS deaggregations.

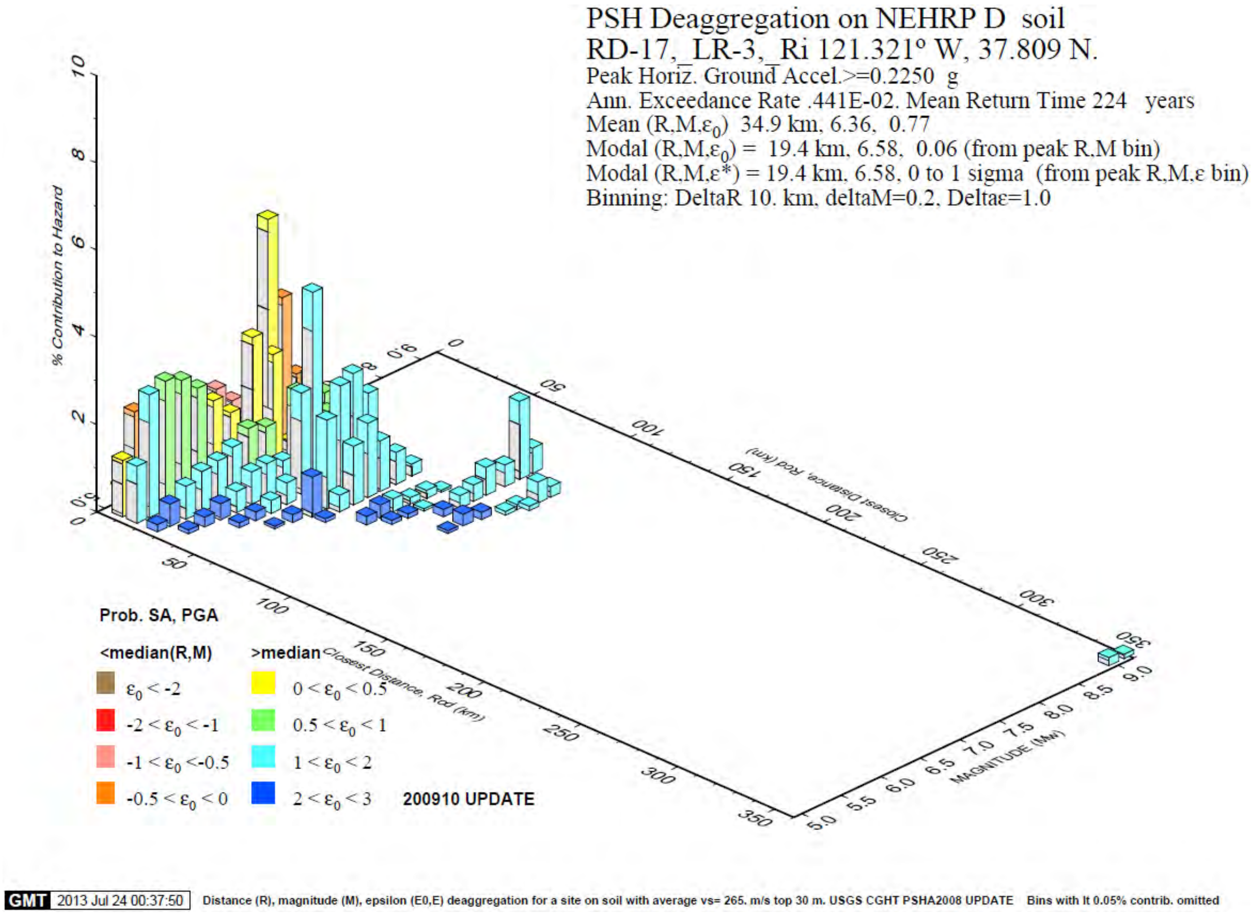


Figure 3-35: USGS 2008 Interactive Deaggregations (Beta) Output

The mean magnitude or the weighted average considering the percent contribution to the total hazard for the study levees is 6.4. The most significant contributions are induced by The Great Valley 7 Char Fault System and the Great Valley 7 GR Fault System. The Great Valley 7 Char Fault System is capable of M = 6.7 and located approximately 20 km from the site, while the Great Valley 7 GR Fault System is capable of M = 6.6 and located approximately 21 km from the site.

3.7.2 Liquefaction And Ground Deformation

Many of the levees within the LSJRFS area are constructed over alluvial deposits and may be susceptible to liquefaction or degradation due to a seismic event. Levees meeting static stability criteria likely have sufficient factors of safety to resist the additional loading from earthquakes unless the levee or foundation materials lose significant strength due to liquefaction. The LSJRFS area is unusual in that it contains infrequently loaded levees in Central and South Stockton, but also frequently loaded levees in North Stockton. Infrequently loaded levees are likely to be unsaturated at the time of a large seismic event; the material in the levee often can be considered non-liquefiable due to lack of saturation. Frequently loaded levees, as defined by Section 7.6 of the State of California Urban Levee Design Criteria (ULDC), experience a water surface elevation at least one foot above the landside toe at least once a day for greater than 36 days per year. Frequently loaded levees are likely to be sensitive to seepage leading to breach with seismic-event induced transverse cracking or displacement.

The seismic and liquefaction evaluation for the LSJRFS area primarily focused on examining potential layers that could experience liquefaction and their associated impact to global slope stability of the levee section. In most of the cases/Reaches it was determined that liquefaction was primarily isolated to the deeper foundation layers and that it had minimal effect on the global stability of the levee and foundation. In five (5) cases within RD-17 and RD-404, the liquefiable layers were shallow enough such that they could pose a significant effect on the stability of the levee.

Even though global instability resulting from liquefaction does not appear to be a primary concern when the liquefiable layers are located at greater depths, there could be other seismic performance concerns given the geologic nature of the area and the potential for differential settlement. The foundations for many of the segments, especially in the North Stockton areas of Delta Brookside and Delta Lincoln Village, consist of numerous geomorphologic braided channels that run orthogonal to the levee axis. As a result, there are variable foundation conditions along the axis of these levees. The variability of the foundation coupled with the potential for transverse cracking due to liquefaction, differential settlement, and areas that are frequently loaded that are protecting dense populations, are a concern and should be carefully considered in the alternatives. The results of the Seismic and liquefaction analysis for the LSJRFS are included as Enclosure 4.

4. WITH PROJECT CONDITIONS DESCRIPTION

The LSJRFS is evaluating Federal interest in alternatives to reduce flood risk in the study area. The geotechnical analyses performed have identified several technical deficiencies associated with the flood risk management system protecting the study area. There are various alternatives under consideration to address these deficiencies and the geotechnical components of those alternatives are discussed in the following sections of this report. Most of the alternatives consist of various structural measures to remediate existing levees for seepage, slope stability, and/or erosion, and some alternatives include measures to improve conveyance.

4.1 TYPICAL LEVEE IMPROVEMENT MEASURES

Where levee height, geometry, erosion, access, vegetation, seepage, and/or slope stability deficiencies were identified (criteria not met), improvement measures were assigned to the affected reaches of levees. Improvement measures for geotechnical deficiencies consisting primarily of cutoff walls, seepage berms, stability berms, and slope flattening were included in development of conceptual alternative cross-sections. This section of the report discusses the various improvement measures considered at a conceptual level, and not as applied to a specific reach.

4.1.1 Cutoff Walls

Seepage cutoff walls are vertical walls of low hydraulic conductivity material constructed through the embankment and foundation to cut off potential through-seepage and underseepage. In order to be effective for underseepage mitigation, cutoff walls usually tie into an impervious sub-layer. Cutoff walls generally require no additional permanent levee footprint. The levee typically is degraded by one half the levee height to provide a sufficient working surface (minimum about 30 feet) and prevent hydraulic fracture of the levee. Following construction of the cutoff wall, the levee is then rebuilt either with the existing levee material with an impervious cap above the cutoff wall, or with imported impervious levee fill material. Cutoff walls are typically constructed of either a soil bentonite (SB), soil cement bentonite (SCB), or cement bentonite (CB) mixture depending on in-situ soil conditions and desired construction method.

The conventional slurry method for SB or SCB walls is an open trench method that uses an excavator with a long-stick boom to excavate the slurry trench. A bentonite-water slurry is used to keep the trench open and stable prior to backfilling with the permanent wall material. Soil is mixed with bentonite (SB) or with bentonite and cement (SCB) then pushed into the trench, displacing the bentonite-water slurry. Alternatively, the open trench method can be used for CB walls, whereby the trench is backfilled with the self-hardening slurry mixture. The self-hardening slurry backfill is used to keep the trench open and stable, allowing excavation of a new section without waiting for the entire trench to be excavated. The conventional method using a long stick boom excavator has a maximum depth of about 70 to 80 feet.

Deeper cutoff walls can be constructed using the Deep Soil Mixing (DSM) or Deep Mix Method (DMM), jet grouting, and soil cutter mixing. These deeper cutoff walls use specialized construction equipment to mix the soil with low permeability materials, typically bentonite and/or cement, in-situ and are capable of depths of more than 100 feet. DSM and DMM use augers to mix low permeability materials into the subsurface soils, iteratively performed along a linear layout, to create overlapping columns of treated soil that form a wall within the subsurface soils. Jet grouting uses the injection of grout from vertical holes to create overlapping columns or panels that form a wall within the subsurface soils. Cutter soil mixing uses a cutter head equipped with cutter wheels that allow vertical penetration within the subsurface soils and mixing of bentonite and/or cement slurry that is injected during the penetration and withdrawal of the cutter head; iterative performance along a linear layout creates overlapping panels that form the cutoff wall.

4.1.2 Seepage Berms

Seepage berms are earth structures built along the levee landside toe that provide additional weight to prevent blanket layer heave, reduce exit gradients, and allow for safe exit of underseepage. Seepage berms can be pervious, semi-pervious, or impervious, and may require a significant amount of land. For some sites, due to adjacent property uses, there is not sufficient room along the landside toe for a seepage berm. The required dimensions of a seepage berm (width and thickness) depend on site specific conditions and may vary over the length of a levee. Seepage berm widths commonly range from a few tens of feet to a few hundred feet. Berm thickness typically ranges from a few feet to several feet. It was beyond the scope of the LSJRFS to perform site specific analyses to dimension seepage berms throughout the study area. Instead, typical berm dimensions were used, and levee height was used as a proxy for underseepage demand (indicating needed berm width). For the LSJRFS, the required seepage berm width was taken as ten times the levee height, with a maximum width of 300 feet. The thickness of the berm is 5 feet at the levee toe and 3 feet at the toe of the berm.

4.1.3 Slope flattening

Slope flattening is a mechanical method to repair a slope that may not have stable slopes by reducing the steepness of the slopes. Waterside and landside slopes can be graded using construction equipment to flatten slopes. In most cases, this process requires the removal of all vegetation and encroachments from the levee slope being flattened. Slopes are typically flattened to 3H:1V or flatter; for the LSJRFS, slope flattening was set at 3H:1V.

4.1.4 Stability Berms

Stability berms are earth structures built against the levee landside slope to stabilize unstable slopes, or in some cases to capture seepage through the levee. Stability berms may be constructed of a random fill material placed over blanket and chimney drainage features to capture seepage through the levee. A thin filter sand layer may be placed between the drainage layer and the levee embankment and native soils. Geotextile fabric may be placed between the free drainage layer and the levee fill. Typically, the height of the stability berm is on the order of two-thirds of the height of the levee. Drained stability berms have the benefit of also reducing susceptibility to through-seepage.

4.1.5 Floodwall/Retaining Wall

A floodwall is a structural wall that is constructed either in lieu of a levee or on top of a levee (to raise the elevation of the top) to separate the waterside from the protected side. Floodwalls are an efficient, space-conserving method for containing unusually high water surface elevations. They are often used in highly developed areas where space is limited. They are primarily constructed from pre-fabricated materials, although they may be cast or constructed in place, and are constructed almost completely upright. Floodwalls consist of relatively short elements (in plan view), making the connections very important to their stability. Floodwalls on top of levees are typically located along a levee hinge point to allow vehicular access along the crown. The drawback is that floodwalls prohibit access to or from the levee slopes, and may inhibit visual inspection of the slope and toe areas from the crown if the wall is of sufficient height.

At the time this report was authored, floodwalls were not part of the proposed alternatives; however, they still remain a topic of consideration.

4.1.6 Embankment Fill/Levee Raise

To address deficiencies found in the required levee height, various methods of raising the existing levee crown elevation may be implemented. Two options are forms of embankment fill placement: a crown-only levee raise, and a full levee raise. A crown-only levee raise is feasible where the levee crown is currently wide enough to support the placement of additional embankment material while maintaining the minimum allowable crown width upon the completion of the raise. A full levee raise includes an embankment raise from the waterside crown hinge point upward at an appropriate waterside slope angle, establishing a new crown width to meet criteria, and placement of fill against the landside slope such that the levee is widened to the landside and the new landside slope extends up to meet the newly established crown.

4.1.7 Bank Protection

In areas that have no or minimal waterside berm, on bank rip-rap is placed on the waterside levee slope to protect against erosion. This entails filling the eroded portion of the bank and installing stone protection along the levee slope from the base of the erosion area to the top of the erosion area. Vegetation would be limited to grass. If there is a natural bank distinct from the levee that requires erosion protection, it would be treated with stone protection. Existing woody vegetation would be removed within the vegetation-free zone. Grass would be allowed in this area.

Additionally, a rip-rap waterside berm may be constructed from the base of the erosion to above the mean summer water level (MSWL) and then placing stone protection on the levee or bank slope above the MSWL. The stone berm may support riparian vegetation and provide a place to anchor in-stream woody material (IWM). This design provides near-bank, shallow-water habitat for fish.

4.1.8 Anticipated Borrow Source

The Sponsors have provided preliminary locations of expected borrow site sources. The material is expected to be sourced from several sites within approximately 25 miles of the study area including:

- storm-water detention basins North of Bear Creek between I-5 and SR-99, south of Lodi Waste Water Treatment Plant (WWTP)

- storm-water detention basins in undeveloped areas of RD17
- groundwater recharge basins in the row crop parcels of the area (between Old Calaveras River, Diverting Canal, and Mormon)
- storm-water detention basins in the French Camp Slough/Airport area between I-5 and SR-99
- groundwater recharge basins in the row crop parcels of the French Camp Slough watershed just east of SR-99

No USACE investigation or laboratory testing has been performed in these areas to verify that the materials meet the requirements for borrow materials as stated in Section 4.3.2. Depending on the selected improvements, it is possible that existing levee material may be used as a source of borrow material. Typically, the existing levee is composed of poorly graded sands, silty sands, and sandy silts on the rivers and streams, while bypass levees were usually constructed of lean to fat clays. This material can be considered suitable for use in the construction of some stability berms, seepage berms, and for reconstructing the levee embankment where a cutoff wall is proposed; however, existing levee material is subject to the material requirements given in Section 4.3.2. Significant quantities of engineered fill of various specifications will be required to construct the proposed project. Refer to other Appendices for the estimated quantities needed for construction.

4.2 OTHER STRUCTURAL MEASURES

Other structural measures proposed for the LSJRFS area include closure structures, weirs, and proposed channel improvements.

4.2.1 Closure Structures/Gates

Some of the current project alternatives utilize closure structures at various locations within the LSJRFS area.

Fourteen Mile Slough would require an operable closure gate with the western-alignment levee configuration (refer to other appendices for description of the western alignment configuration). The closure structure would be operable to passing vessels and rising water surface elevations. With the western alignment configuration, the levees protecting Delta Lincoln Village on its western and southern sides, as well as the levee north of Delta Brookside, would remain both geotechnically and seismically vulnerable if the closure structure were not constructed and appropriately operated.

Excessive encroachments throughout the north and south banks of Smith Canal may necessitate a closure gate for controlling a high water event that may otherwise jeopardize existing levee performance. The gate for Smith Canal would be operable to passing vessels and rising water surface elevations.

The Mormon Slough Bypass would require a closure gate to convey an additional 2000 cfs of flow diverted from the Mormon Channel into the Bypass.

During this feasibility study, no geotechnical investigation or analysis was performed in these areas in support of evaluating or developing designs for closure structures. During the Pre-Construction Engineering and Design phase (PED) of this project, subsurface investigations would need to be performed in the areas of the proposed closure structures to determine foundation conditions for design, the need to mitigate any potential seepage, and further define constraints and requirements.

4.2.2 Channel Improvements/Weirs

Channel improvements are being considered as part of the project alternatives for Mormon Slough Bypass and Paradise Cut.

Currently, Mormon Slough Bypass receives no flow from Mormon Channel as it turns north-west into the Stockton Diverting Canal. The current flows in Mormon Slough Bypass are due to interior drainage with a maximum flow of approximately 1,000 cfs. The current project alternative would propose channel improvements to convey an additional 2,000 cfs of flow diverted from Mormon Channel (instead of that flow entering the Stockton Diverting Canal). Channel improvements would consist principally of channel widening and modification of potential obstructions (e.g., bridges, utilities). A gate would be constructed to divert flows greater than roughly the 5 or 10 year event that flow down the SDC to the Calaveras River.

Channel improvements to Paradise Cut would include dredging and widening to the area to increase flows and reduce stage downstream on the LSJR. Levees along the left bank of Paradise Cut would be set back further from the existing channel location. This process would also include improvements to widen the uncontrolled weir on the LSJR to allow for increased flows into Paradise Cut.

During this feasibility study, no geotechnical investigation or analysis was performed in these areas in support of evaluating or developing designs for weirs/diversion structures. During the Pre-Construction Engineering and Design phase (PED) of this project, subsurface investigations would need to be performed in the areas of the proposed structures to determine foundation conditions for design, the need to mitigate any potential seepage, and further define constraints and requirements.

4.3 LEVEE IMPROVEMENT MEASURES

Levee improvement measures constitute the vast majority of measures that comprise most alternatives for the LSJRFS. The following sections of this report describe the methodology, criteria, and resulting levee improvement templates developed to mitigate for levee performance issues within the LSJRFS area.

4.3.1 Methodology

The without-project conditions were initially characterized by roughly 40 miles of existing levees within the study area. As part of the Planning process of generating Management Measures and Alternatives, additional lengths of existing levees and also potential new levee alignments were added, expanding the project study area to roughly 90 miles. For all of the existing and proposed levee with-project conditions, the original 14 reaches were expanded to capture the added lengths of levees and then were further divided into 124 smaller reaches, the further division into smaller reaches was done to allow for flexibility in assigning mitigation measures and estimating project costs.

For each of the 124 reaches, the reach was assigned mitigation considering two primary factors: (1) the intent of the Management Measure for the reach, and (2) the geotechnical potential failure modes that need to be mitigated for the reach. For the LSJRFS alternatives, there are four different Management Measure intents for levees:

- Raise existing levee
- Strengthen existing levee
- Raise and Strengthen Existing Levee
- Construct New Levee

For any particular reach of existing levee, different Management Measure intents maybe needed for different alternatives. The geotechnical potential failure modes are the modes discussed in Section 3.1 of this report, mainly: underseepage, through-seepage, slope instability, erosion, and seismicity/liquefaction. The type of mitigation assigned to the reach depended on which potential failure mode(s) had been identified as present at the reach.

Flexibility was designed into the assigned mitigation measures by providing two different template options (per reach) to mitigate performance issues. For example, the option for a cutoff wall vs. the option for a seepage berm would each mitigate underseepage; the flexibility to choose how a performance issue is mitigated allowed for selection of an option that would minimize costs and/or impacts.

Eleven different template options were developed to address a variety of levee performance issues for this project. Discussion of design criteria used to develop the template options follows in Section 4.3.2. The eleven template options assigned as mitigation measures are described in detail in Section 4.3.3 and are included as Enclosure 5. The templates were created following USACE levee design criteria for the purpose of establishing project costs only, the templates are not intended for design.

With-project analyses were not completed on the templates shown in Section 4.3.3. Each of the templates was developed using standard levee criteria, constituents, and configurations. Similar projects with site conditions analogous to this area have used comparable mitigation measures yielding with-project analyses satisfying design requirements and criteria.

4.3.2 Criteria

The following paragraphs present USACE standard levee design and construction criteria as established in both national (HQ) and local (District and Division) policy documents and a discussion of how the PDT has made assumptions in applying those criteria to the LSJRFS area. As stated earlier, it is anticipated that significant quantities of material will be required for construction of the proposed project. Several different performance improvement measures, such as seepage berms, cutoff walls, embankment construction/reconstruction, and erosion protection are proposed. This section describes the proposed minimum material requirements and design criteria for the LSJRFS area.

TYPE I LEVEE FILL (SELECT LEVEE FILL)

The Sacramento District Geotechnical Engineering Branch SOP-03 established the requirements of engineered fill to be used for the construction of levee embankments. This is referred to as either Type I Levee Fill or Select Levee Fill and meets the following requirements:

- 100% passing the 2-inch sieve
- minimum 20% fines content (material passing the #200 sieve, i.e., silt and clay size particles)
- fines must have a liquid limit less than 45 and a plasticity index between 8 and 40
- no organic material or debris may be present

RANDOM FILL

It is acknowledge that not all improvement features will require Type I Levee Fill and that a less stringent material specification is required for some seepage berms, some stability berms, and in some cases reconstructed embankment slopes. The actual specification of this material will be based on the type of material available at project borrow sites, but in general would conform to the following requirements:

- 100% passing the 2-inch sieve
- minimum 12% fines content (material passing the #200 sieve, i.e., silt and clay size particles)
- no organic material or debris may be present

RIP-RAP

Since 1936, the Sacramento District has placed rock erosion protection on the banks and levees and associated tributaries. The Sacramento River Bank Protection Project (SRBPP) uses a standard rip-rap and filter gradation for repair sites which may be appropriate within the LSJRFS area. However, preliminary calculations of rip-rap requirements for a typical channel section with an average channel velocity of 7.0 fps and for 12.0 fps result in a D100 of 18.0 and 36.0 inches with a D15 of 7.1 and 14.3 inches, respectively. If erosion protection is to be part of the LSJRFS area mitigation alternatives, the actual gradations will need to be determined during design. Rip-rap erosion protection would adhere to the following: the rip-rap should be angular in shape, sound, durable, and hard; the rip-rap should also be free from laminations, weak cleavages, undesirable weather, blasting or handling induced fractures; the rip-rap stone should be of such character that it will not disintegrate from the action of air, water, or conditions of handling and placing and should be free from earth, clay, refuse, or adherent coatings.

GEOMETRY

The typical USACE levee section established by the USACE guidance document EM 1110-2-1913 is nationally considered to have a minimum 10-foot crest width with waterside and landside slopes no steeper than 2H:1V. The Sacramento District guidance document, SOP-03 (Standard Operating Procedure), suggests a minimum crest width of 20 feet for mainline and major tributary levees and 12 feet for minor tributary levees; the levee section should have 3H:1V waterside and landside slopes, except existing levees with good past performance where existing 2H:1V slopes are acceptable. The use of Sacramento District standard sections is generally limited to levees of moderate height, less than 25 feet, in reaches where there are no serious underseepage problems, weak foundation soils, or constructed of unsuitable materials. The standard levee section may have more than the minimum allowable FOS relative to slope stability and seepage, its slopes being established primarily on the basis of construction and maintenance considerations.

For the LSJRFS area, the minimum crest width for mainline or major tributary levees is 20 feet; the minimum crest width for minor tributaries levees is 12 feet. Existing levees with landside and waterside slopes as steep as 2H:1V may be used in rehabilitation projects if slope performance has been good and if the slope stability analyses determined the factors of safety to be adequate. Newly constructed levees should have 3H:1V waterside and landside slopes.

VEGETATION AND ACCESS

Vegetation, encroachment, and access policy includes EM 1110-2-1913, SOP-03, and ETL 1110-2-571 *Guidelines for Landscaping and Vegetation Management at Levees, Floodwalls, Embankments Dams, and Appurtenant Structures*. The vegetation-free zone, as established by ETL 1110-2-571, is a three-dimensional corridor surrounding all levees, floodwalls, and critical appurtenant structures in a flood damage reduction system. The vegetation-free zone applies to all vegetation except grass. The minimum height of the corridor is 8 feet, measured vertically from any point on the ground. The minimum width of the corridor is the width of the flood-control structure (Levee toes or floodwall stem), plus 15 feet on each side, measured from the outer edge of the outermost critical structure. Figure 4-1 taken from Section 6-1 of ETL 1110-2-571 shows a two dimensional representation of the vegetation-free zone of a basic levee cross-section.

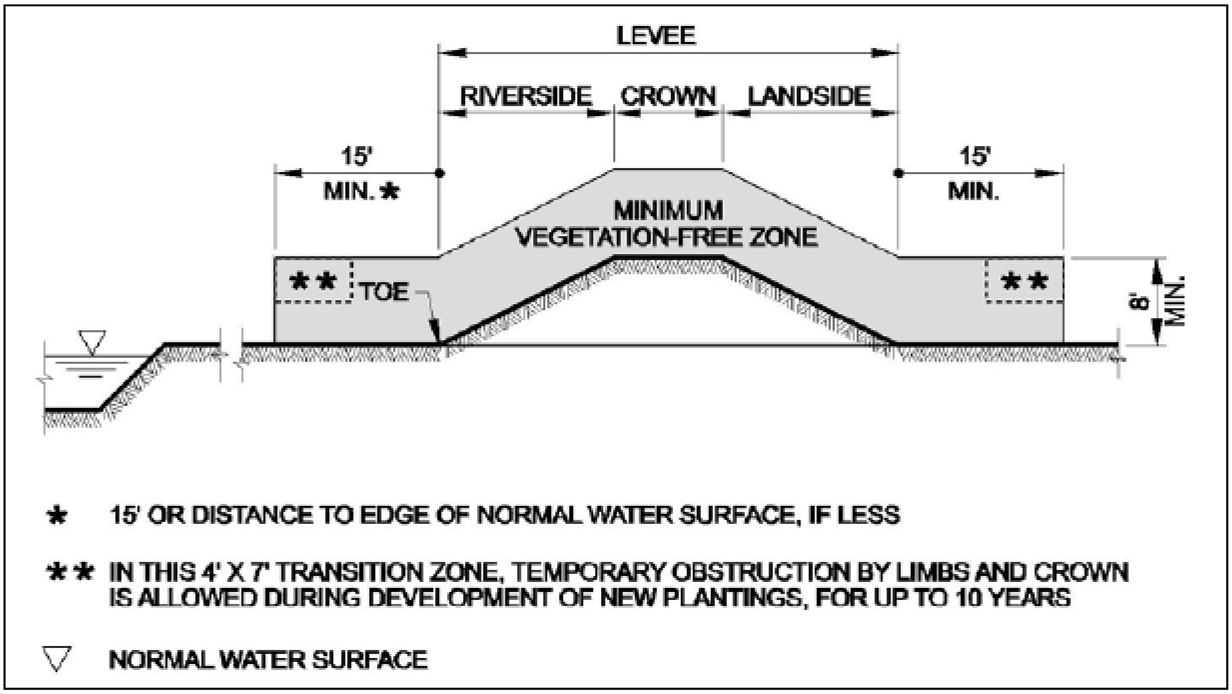


Figure 4-1: Vegetation-Free Zone of Basic Levee

The primary purpose of the vegetation-free zone is to prevent any damages of the levee embankment due to vegetation (including seepage along the woody vegetation root system, additional scouring of the waterside slope due to trees uprooting, and attraction of rodents) and to provide a reliable corridor of access to and along the flood-control structure for flood fighting, inspection, and maintenance of the flood control structures. The access corridor must be an all weather corridor free of obstructions to assure adequate access by personnel and equipment for surveillance, inspection, maintenance, monitoring, and flood-fighting. In the case of flood-fighting, this access corridor must also provide the unobstructed space needed for the construction of temporary flood-control structures. Access is typically by four-wheel-drive vehicles, but for some purposes, such as maintenance and flood-fighting, access is required for larger equipment, such as tractors, bulldozers, dump trucks, and helicopters. Accessibility is essential to the reliability of flood damage reduction systems. The Sacramento District guidance document, SOP-3, suggest easements consist of a minimum 20 foot landside clear access easement and a minimum 15 foot waterside easement.

For new levees constructed in the LSJRFS area, a minimum landside toe clear access easement of 20 feet is required; for existing levees within the LSJRFS area, a minimum landside toe clear access easement of 10 feet is required. For both new and existing levees in the LSJRFS a minimum waterside toe vegetation-free zone of 15 feet is required.

For a levee section to be considered compliant with USACE vegetation policy it must either have been cleared of vegetation within the vegetation-free zone or eligible for a variance from USACE policy on vegetation in ETL 1110-2-571. The variance must be necessary, and the only feasible means to preserve, protect, and enhance natural resources, and/or protect the rights of

Native Americans, pursuant to treaty, statute, or executive order. The variance must assure that safety, structural integrity, and functionality are retained, and accessibility for maintenance, inspection, monitoring, and flood-fighting are retained. The variance may require structural measures to mitigate vegetation, such as overbuilt sections, to improve levee system reliability, redundancy, or resiliency with respect to the detrimental impacts of the vegetation.

SEEPAGE AND STABILITY

Seepage and slope stability criteria for geotechnical analysis were established based on ETL 1110-2-569 Design Guidance for Levee Underseepage, EM 1110-2-1913 Design and Construction of Levees, Sacramento District's SOP-03, and the State of California ULDC. Steady state seepage analysis for a design water surface elevation considered a maximum allowable vertical exit gradient at the toe of the levee to be 0.5. In general, this provides a FOS against uplift failure of about 1.6, considering an impervious blanket saturated unit weight of 112 pcf. Steady state seepage analysis for a top-of-levee water elevation considered a maximum allowable vertical exit gradient at the toe of the levee to be 0.8. In general, this provides a FOS against uplift failure of about 1.0, considering the impervious blanket saturated unit weight of 112 pcf. The minimum required FOS for the design water surface elevation for the landside steady state slope stability analysis is 1.4. The minimum required FOS for the top-of-levee water surface elevation for the landside steady state slope stability analysis is 1.2. For landside seepage berms, a maximum allowable vertical exit gradient at the toe of the berm is considered to be 0.8. The analysis cases of during construction, post construction, rapid drawdown, and waterside partial pool were considered to be design level analyses and were not performed for this feasibility study.

As discussed in Section 4.3.1, geotechnical seepage and stability analyses were not performed in this study for the with-project template configurations. The template configurations were developed using standard levee criteria, constituents, and configurations. Configurations similar to the templates have been used in many previous projects and been shown to meet the seepage and stability criteria listed here. Some refinements to the configurations may be needed and should be expected; such refinements are design-level analysis and are beyond the scope of this feasibility study.

SEISMICITY AND LIQUEFACTION

As stated in Section 3.7.2, the LSJRFS area is unlike most levee system locations in that the study area contains both infrequently loaded levees (Central and South Stockton) and frequently loaded levees (North Stockton) as defined by the ULDC. The presence of frequently loaded levees in the study area creates special concern with respect to seismic events. In particular, the presence of frequently loaded levees means that it is not especially unlikely that a seismic event will occur concurrently with a high-water event. For most other study areas, it is very unlikely to have a concurrent seismic event and a high-water event. For such areas, a seismic event may damage levees, but since there is no water high on the levees when the damage occurs, flooding due to breach of the levees is very unlikely. For areas like North Stockton, water is often high on the levees. Therefore, it is not particularly unlikely to have a concurrent seismic event and a high-water event. During such an event, if the seismic event damages the levees, the damage may indeed cause flooding due to breaching of the levees.

For the LSJRFS levees, the most likely damage inducing mechanism during a seismic event is liquefaction. The consequences of triggering liquefaction may include flow slide or post-earthquake instability and lateral spreading. Where static driving shear stress is greater than the resisting strengths after liquefaction (residual strength), a global or structural failure can occur leading to loss of freeboard, cracking, and increased vulnerability to piping. Lateral deformation can also develop as a consequence of instability due to partial loss of shear strength or accumulation of shear strains throughout the soil profile. Lateral spreading towards any open channel or face can occur in mildly sloping ground and extend to very large distances away from the open face. Vertical displacement can develop as a consequence of reconsolidation of the liquefied soil.

Areas of concern during and after a seismic event would include those where transverse cracking might develop between liquefiable and non-liquefiable reaches and where these zones may abut infrastructure.

For the most critical category of levee (e.g., urban levees that are frequently loaded) the following displacements may be considered acceptable:

- For frequently loaded levees with less than 5 feet of freeboard, seismic deformations should be less than 3 feet of total deformation and limited to 1 foot of vertical displacement.
- Frequently loaded levees with larger cross-sections and freeboard may be allowed larger deformations with supporting analyses.
- Frequently loaded levees may need to consider design ground motions higher than the 200-year-return-period level.

As stated in Section 3.7.1, seismic loading parameters are developed using the USGS 2008 PSHA Interactive Deaggregation web program, and analyses to determine liquefaction potential are based on a procedure from the summary report of the 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/National Science Foundation (NSF) Workshops on Evaluation of Liquefaction Resistance of Soils; published as part of the Journal of Geotechnical and Geoenvironmental Engineer, dated October 2001 (Youd, Idriss, Andrus, & Arango, October 2001).

For the LSJRFS study, global or structural stability was evaluated where liquefiable layers with factors of safety less than 1.0 were found. Lateral spreading and post-liquefaction reconsolidation settlement were considered only when structural stability had a FOS greater than 1.0 but not greater than 1.2. Where liquefiable layers were found to have a FOS less than 1.0, static limit equilibrium stability analysis using UTEXAS4 based on Spencer’s method was performed; if an adjacent zone had a FOS less than 1.4, it was included with the zone containing liquefiable layers. Automatic circular shear surface search and non-circular or wedge shear surface search were performed for both the landside and waterside in UTEXAS4. Post-earthquake residual shear strength was used for the liquefiable layers. The residual strength was estimated per Seed and Harder, 1990.

A more detailed description of the design criteria used for the LSJRFS area is displayed in the graphics in Section 4.3.3 and included as Enclosure 5.

4.3.3 Mitigation Measure Templates

The eleven (11) templates described below were developed to address a variety of levee performance issues while following current USACE levee design criteria as described in the preceding sections of this report. For the LSJRFS, the purpose of the assigned template was to develop quantities for establishing project costs. The templates are not intended for design or construction.

Template Option 1, Landside Slope Reconstruction, has a reconstructed landside slope and includes an internal drainage layer to mitigate for through-seepage of the levee embankment and/or seepage-related landside slope instability. This template has the flexibility to accommodate varying levee heights and crest widths. The variables shown in Figure 4-2 were assigned values when submitted as a mitigation measure based on location (e.g., geotechnical conditions within the reach, geometry of existing levees within and adjacent to the reach, etc.) and USACE levee design criteria. This template would be assigned in areas where the landside of the embankment was identified as having potential deficiencies of landside slope instability and/or through-seepage, but without an underseepage deficiency.

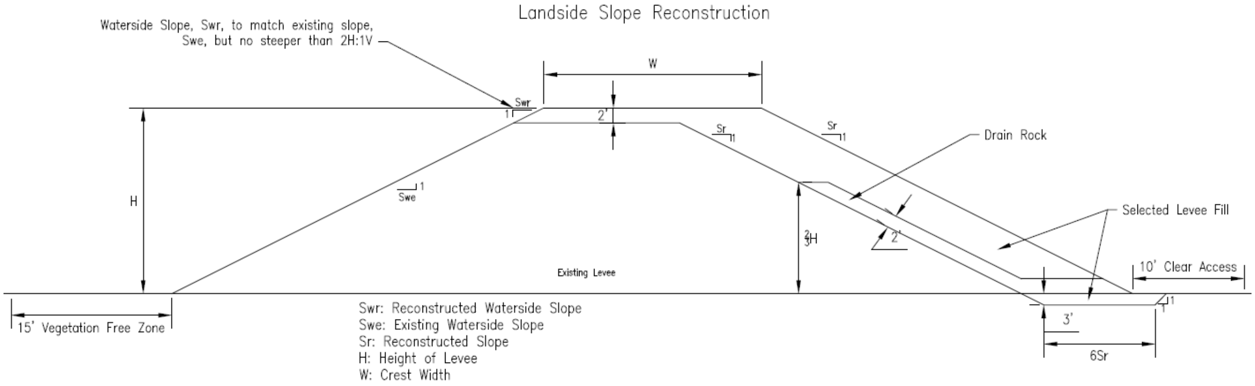


Figure 4-2: Template Option 1 – Landside Slope Reconstruction

Template Option 2, Centerline Cutoff Wall, contains a cutoff wall (usually SB or SCB) to mitigate for through-seepage and underseepage. This template provides secondary benefits by reducing pore pressures that could lead to internal erosion, and improved landside slope stability. This template has the flexibility to accommodate varying levee heights, and depth of cutoff wall. Traditional methods of cutoff wall excavation involve a long-arm excavator with maximum depths of excavation between 75 to 80 feet below ground surface (BGS) of the working platform; depths beyond 75 to 80 feet BGS would require a DSM method with increased associated costs. The variables shown in Figure 4-3 were assigned values when submitted as a mitigation measure based on location and USACE levee design criteria. This template would be assigned in areas that were identified as having an underseepage and/or through-seepage deficiency. If crest width (W) does not meet USACE levee design criteria, Template Option 3, Cutoff Wall with Landside Slope Reconstruction, would supersede this template option.

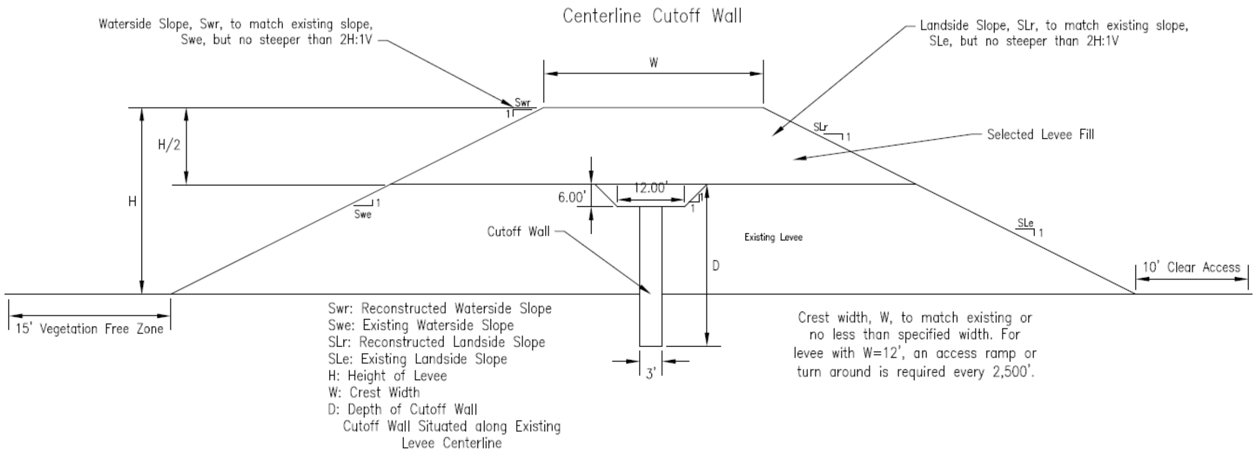


Figure 4-3: Template Option 2 – Centerline Cutoff Wall

Template Option 3, Cutoff Wall with Landside Slope Reconstruction, has a reconstructed landside slope and contains a cutoff wall (usually SB or SCB) to mitigate for through-seepage and underseepage. This template provides secondary benefits by reducing pore pressures that could lead to internal erosion, and improved landside slope stability. The presence of the cutoff wall negates the need for the internal drainage layer at the reconstructed landside slope. The template includes a half-levee degrade/reconstruction, as described in Section 4.1.1. This template has the flexibility to accommodate varying levee heights, crest widths, and depth of cutoff wall. Traditional methods of cutoff wall excavation involve a long-arm excavator with maximum depths of excavation between 75 to 80 feet below ground surface (BGS) of the working platform; depths beyond 75 to 80 feet BGS would require a DSM method with increased associated costs. The variables shown in Figure 4-4 were assigned values when submitted as a mitigation measure based on location and USACE levee design criteria. This template would be assigned in areas that were identified as having an underseepage and/or through-seepage deficiency along with a levee crest that is narrow (i.e., that needs to be widened).

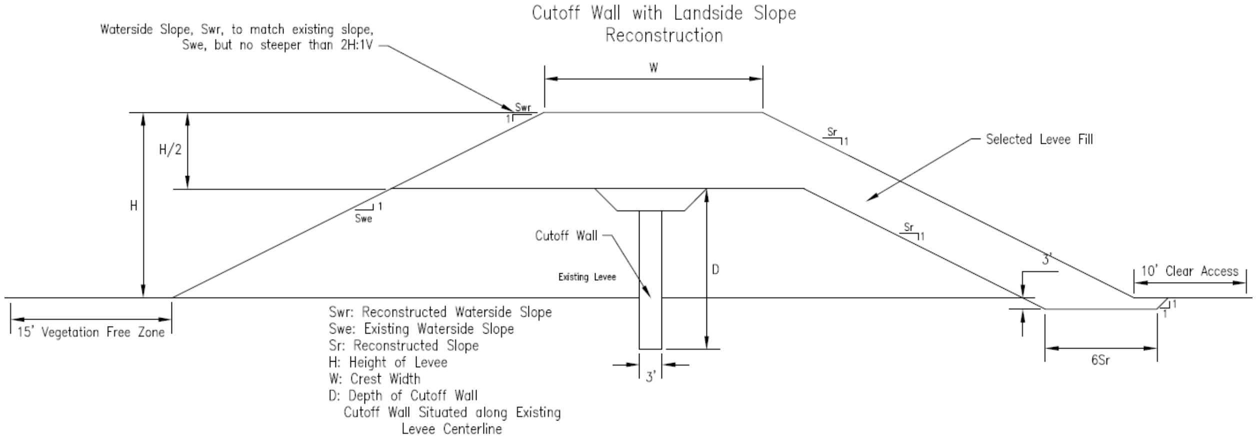


Figure 4-4: Template Option 3 – Cutoff Wall with Landside Slope Reconstruction

Template Option 4, Levee Raise with Cutoff Wall, is similar to Template Option 3 (Cutoff Wall with Landside Slope Reconstruction) but also includes components to raise the height of the levee to address height deficiency. The variables shown in Figure 4-5 were assigned values when submitted as a mitigation measure based on location and USACE levee design criteria. This template would only be assigned in an area with a height deficiency where there was also an underseepage and/or through-seepage deficiency. Template Option 3 would supersede this option if no height deficiency were present.

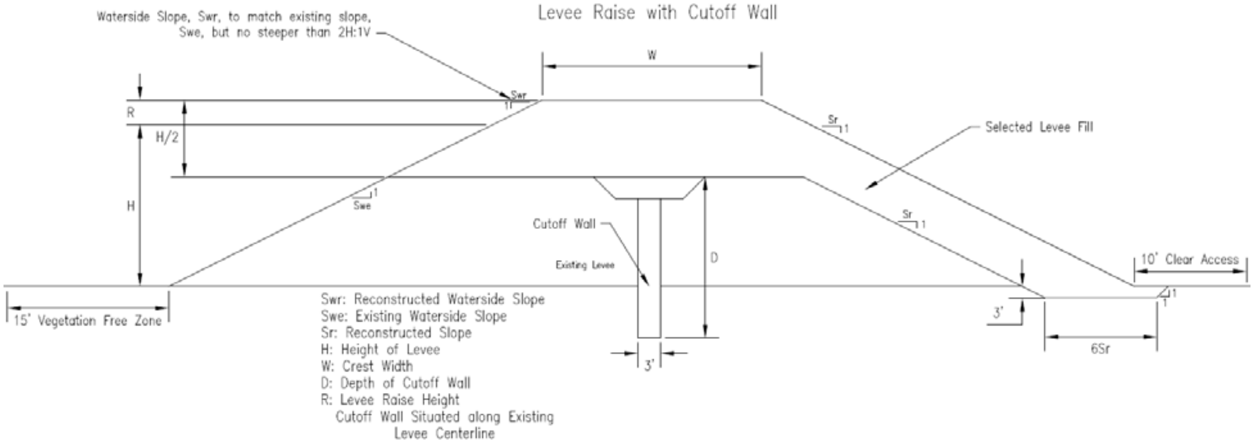


Figure 4-5: Template Option 4 – Levee Raise with Cutoff Wall

Template Option 5, Seepage Berm, includes a landside seepage berm to mitigate for underseepage. This template would be for existing levees with an underseepage deficiency but not through-seepage or landside slope instability. Even though this template has the flexibility to accommodate varying levee heights and crest widths, the width of the seepage berm, W_b, shown in Figure 4-6 follows USACE levee design criteria and adjusts to varying levee heights per site conditions. The seepage berm width, W_b, was set at 10H (where H is the levee height) for cost estimating purposes. Actual seepage berm widths depend largely on site specific geotechnical conditions; calculation of actual widths that would be needed was beyond the scope of this study.

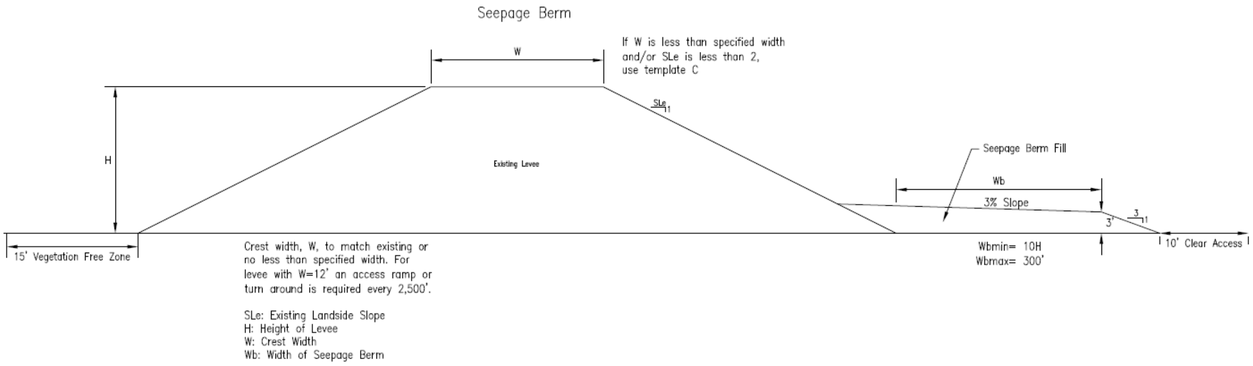


Figure 4-6: Template Option 5 – Seepage Berm

Template Option 6, Combination Berm, has a reconstructed landside slope and includes a landside seepage berm to mitigate for underseepage and also through-seepage and/or landside slope instability and/or crest widening. This template was included as an alternative option to the Cutoff Wall with Landside Slope Reconstruction option, Template Option 3. This template has the flexibility to accommodate varying levee heights, crest widths, and seepage berm widths. The variables shown in Figure 4-7 were assigned values when submitted as a mitigation measure based on location and USACE levee design criteria. The seepage berm width, W_b, was set at 10H (where H is the levee height) for cost estimating purposes. Actual seepage berm widths depend largely on site specific geotechnical conditions; calculation of actual widths that would be needed was beyond the scope of this study. This template would be assigned in areas that were identified as having an underseepage deficiency along with the need for levee crest widening.

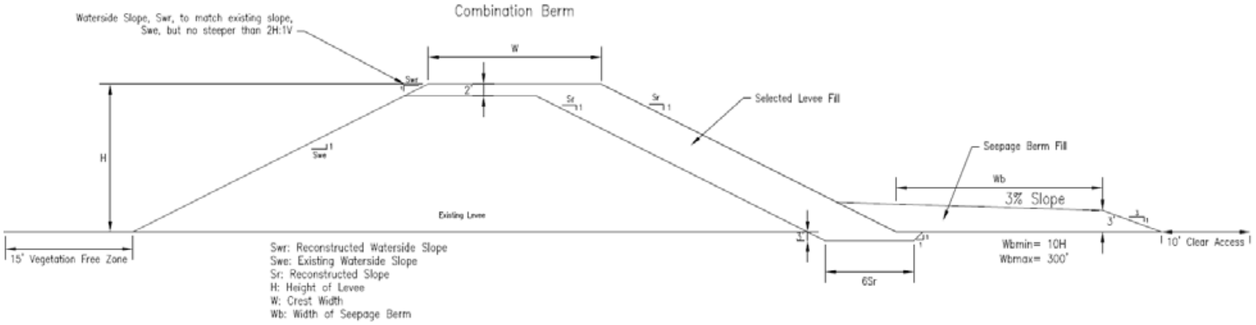


Figure 4-7: Template Option 6 – Combination Berm

Template Option 7, Levee Raise with Combination Berm, is similar to Template Option 6 (Combination Berm) but also includes components to raise the height of the levee to address height deficiency. This template was included as an alternative option to the Levee Raise with Cutoff Wall option, Template Option 4. The variables shown in Figure 4-8 were assigned values when submitted as a mitigation measure based on location and USACE levee design criteria. The seepage berm width, W_b, was set at 10H (where H is the levee height) for cost estimating purposes. Actual seepage berm widths depend largely on site specific geotechnical conditions; calculation of actual widths that would be needed was beyond the scope of this study. This template would only be assigned in areas with a height deficiency where there was also an underseepage deficiency. Template Option 6 would supersede this option if no height deficiency were present.

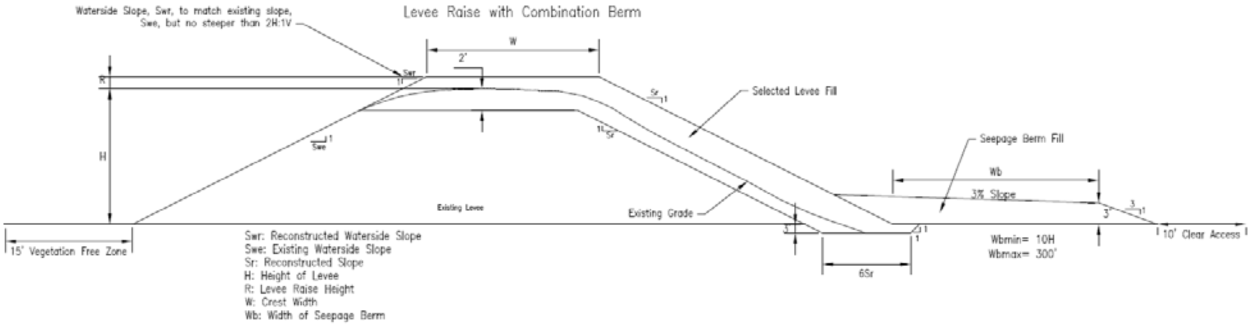


Figure 4-8: Template Option 7 – Levee Raise with Combination Berm

Template Option 8, New Levee, would be for areas where a new levee is proposed and no additional measures are needed to mitigate for underseepage. This template has the flexibility to accommodate varying levee heights and crest widths. The variables shown in Figure 4-9 were assigned values when submitted as a mitigation measure based on location and USACE levee design criteria.

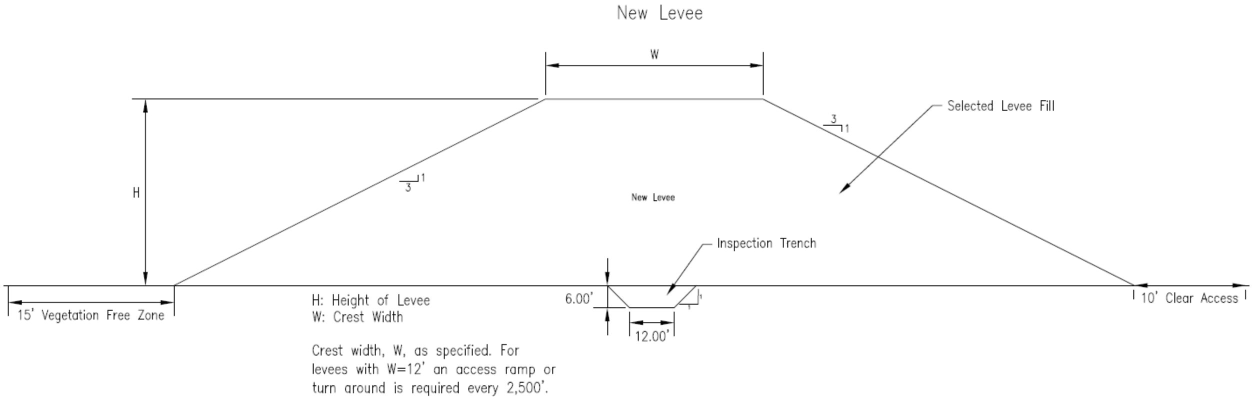


Figure 4-9: Template Option 8 – New Levee

Template Option 9, New Levee with Cutoff Wall, is a template for a new levee (i.e., at a location where no levee currently exists) but that also includes a cutoff wall to mitigate for underseepage. This template was included as an alternative option to the New Levee with Seepage Berm option, Template Option 10. This template has the flexibility to accommodate varying levee heights, crest widths, and depth of cutoff wall. The variables shown in Figure 4-10 were assigned values when submitted as a mitigation measure based on location and USACE levee design criteria.

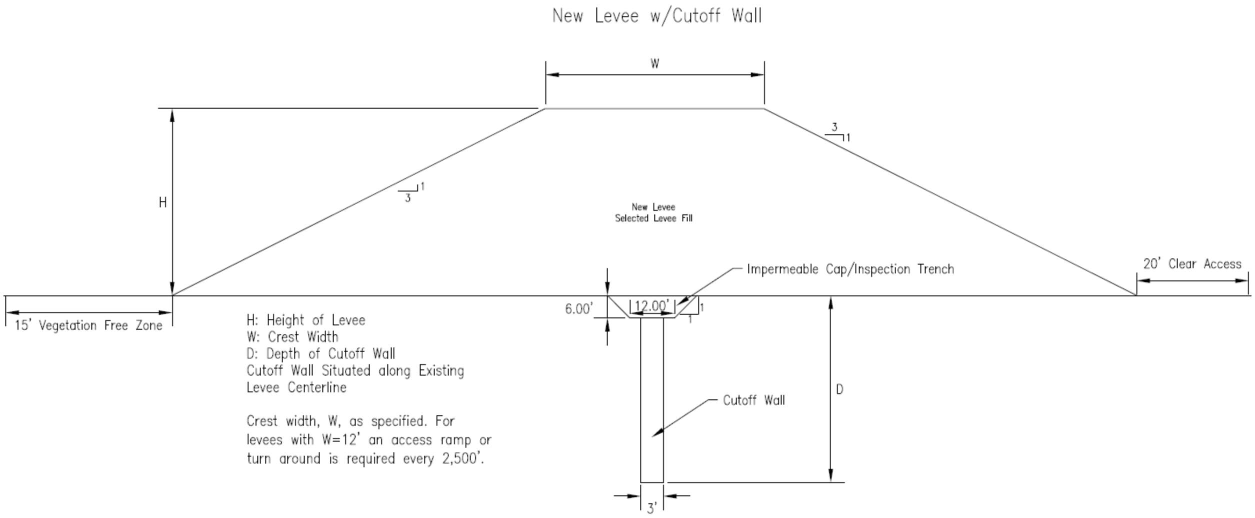


Figure 4-10: Template Option 9 – New Levee with Cutoff Wall

Template Option 10, New Levee with Seepage Berm, is a template for a new levee (i.e., at a location where no levee currently exists), but also includes a landside seepage berm to mitigate for underseepage. This template would be for new levee construction in areas with the potential for underseepage, where the underseepage potential would not be adequately mitigated by the standard levee width. This template has the flexibility to accommodate varying levee heights, crest widths, and seepage berm widths. The variables shown in Figure 4-11 were assigned values when submitted as a mitigation measure based on location and USACE levee design criteria. The seepage berm width, W_b, was set at 10H (where H is the levee height) for cost estimating purposes. Actual seepage berm widths depend largely on site specific geotechnical conditions; calculation of actual widths that would be needed was beyond the scope of this study.

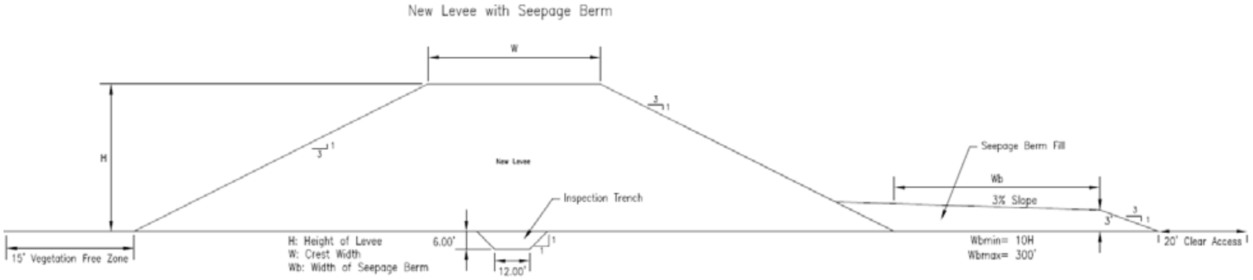


Figure 4-11: Template Option 10 – New Levee with Seepage Berm

Template Option 11, Seismic DSM (Levee Degrade) Seismic Remediation, is an option to remediate areas of special seismic concern, i.e., areas of levee within North Stockton that are frequently loaded (due to slough water surface elevations that are tidally influenced) and that are also subject to potentially significant deformations due to a seismic event. This template incorporates:

- a levee degrade to half its height
- a series of overlapping deep-soil-mixing columns installed at specified longitudinal and transverse spacing that extend just beyond the extents of the levee prism
- reconstructed levee using select levee fill

This template has the flexibility to accommodate varying levee heights, crest widths, and depth of ground improvement. This template provides secondary benefits by reducing pore pressures that could lead to internal erosion, and improved landside slope stability. The variables shown in Figure 4-12 were assigned values when submitted as a mitigation measure based on preliminary seismic analyses, engineering judgment, and USACE levee design criteria. This template would be assigned only in an area of special seismic concern, i.e., areas where the levees are frequently loaded (tidally) and also subject to potentially significant deformations due to a seismic event.

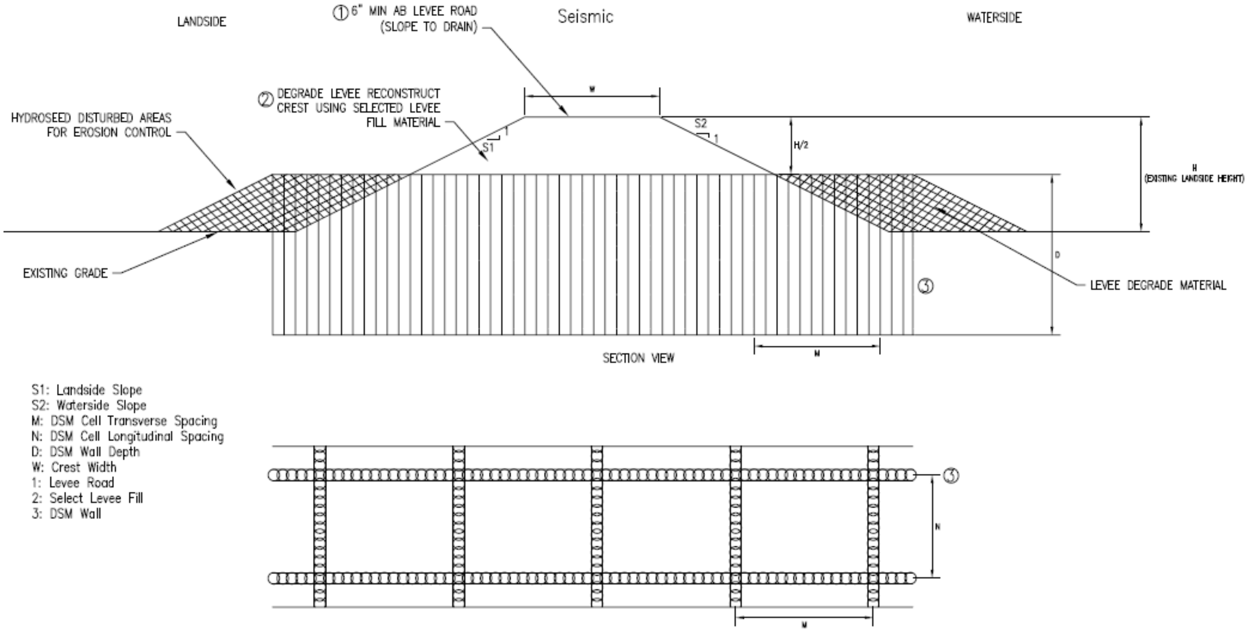


Figure 4-12: Template Option 11 – Seismic DSM (Degrade Levee) Seismic Remediation

4.3.4 Selection of Template Options for Mitigation Measures

As discussed in Section 4.3.1, template options were assigned to each reach considering two primary factors: (1) the intent of the Management Measure for the reach (e.g., strengthen existing levee, raise existing levee); and (2) the geotechnical potential failure modes that need to be mitigated for the reach. Also, to the extent possible, for each reach, two different options were assigned (typically a cutoff wall option and a seepage berm option) to allow for some optimization with respect to costs and impacts. Through this process, Management Measures and their mitigation options were assigned to more than 120 reaches.

Working through the Planning process to the Final Array of Alternatives has yielded six different alternatives. The approximate distribution of the selected template options within the Final Array of Alternatives ranges as follows:

- Template options for cutoff walls and seepage berms were chosen as mitigation between 70-80 percent and 8-10 percent of the time, respectively, to address through-seepage and underseepage.
- The template option for seismic was chosen to represent a smaller percentage of the reaches, roughly 6-8 percent of the time, to address areas with special seismic concerns.
- This template option for new levees was chosen to represent a smaller percentage of the reaches, roughly 4-6 percent of the time, to address areas where a levee did not currently exist.

4.4 WITH PROJECT PERFORMANCE CURVES

Consistent with the evolving Planning process and the implementation of Planning Modernization initiatives, with-project fragility curves were not developed for the LSJRFS. The PDT decided that a with-project condition would be approximated sufficiently by assuming zero fragility for fully remediated levees for the prior-to-overtopping condition, i.e., up to the point where the water surface elevation exceeds the crest elevation (overtopping). This assumption would therefore flat-line the through-seepage, underseepage, slope stability, and judgment curves for water surface elevations below the levee crest elevation. Experience of performing analyses on with-project conditions in similar project areas and design configurations for seepage and stability mitigation has shown it to be reasonable to assume the mitigation measures assigned would successfully mitigate poor performing levees to produce such results. The judgment curve component of the fragility curve would be the only curve that likely would not completely flatten with implementation of the template options, due to remaining potential for vegetation, encroachments, animal burrows, and/or erosion associated with many of the templates (in particular at the lower potions of unaltered levee slopes). Therefore, the assumption of the zero-fragility (i.e., flat-line) fragility curve may potentially overestimate with-project benefits and underestimate residual risk. This was recognized by the PDT and included as a Risk Register item. For further explanation of developing with-project fragility for the LSJRFS, refer to the Economics appendix.

5. CONCLUSIONS

This report presented the results of geotechnical analyses and feasibility level geotechnical recommendations to address technical deficiencies in the flood risk management system protecting the LSJRFS area. The recommended measures consist of a combination of structural measures to mitigate deficiencies in levee height, geometry, erosion, access, vegetation, seepage, and slope stability.

The results of the without project seepage and slope stability analyses for South Stockton indicated that the levees represented by index points LR-1, LR-2, and LR-3 in RD-17 did not meet minimum levee design criteria at various flood frequencies. Historical documentation indicates performance-related issues with seepage, slope instability, and erosion. The measures identified in this study to mitigate these performance issues, to create with-project conditions, typically included a cutoff wall and/or seepage berm.

The results of the without project seepage and slope stability analyses for Central Stockton indicated that the levees represented by index points FR-1 in RD-404, and SL-1 and SL-2 along Stockton Diverting Canal did not meet minimum levee design criteria at various flood frequencies. Historical documentation indicates performance-related issues with seepage and erosion along RD-404, erosion along the left bank of the Calaveras River with isolated areas of seepage, and erosion along the left bank of Stockton Diverting Canal. The measures identified in this study to mitigate these performance issues, to create with-project conditions, typically included a cutoff wall and/or seepage berm.

The results of the without project seepage and slope stability analyses for North Stockton indicated that the levees represented by index points CR-1/CR-2 and D-4 along the right bank of the Calaveras River, and index point D-BS along Delta Brookside, did not meet minimum levee design criteria at various flood frequencies. Historical documentation indicates performance-related issues with settlement, seepage, erosion, and animal burrowing activity along the Delta Brookside study area, and seepage and erosion along Delta Lincoln Village study area. The measures identified in this study to mitigate these performance issues, to create with-project conditions, typically included a cutoff wall and/or seepage berm.

The results of seismic and liquefaction evaluation indicated isolated areas throughout the study area that are capable of inducing significant deformation of the levees. Additionally, liquefaction analyses showed two areas within RD-17, and one area within RD-404, that contained zones of material that are susceptible to liquefaction when subjected to a 200-year seismic event. Most of these areas are unlikely to be capable of inducing flow failures and significant deformation of the levees. However, the Delta Lincoln Village levees and the levees north of Delta Brookside may also be susceptible to significant deformation due to a seismic event. Importantly, these levees are frequently loaded levees. As a result, seismically induced deformation may occur concurrently with a high water condition, which poses a greater risk than is typically the case for levees subject to possible seismic damage. Consequently, a special seismic mitigation measure was identified in this study to mitigate this performance issue to create a with-project condition in these areas.

6. REFERENCES

Atwater, Brian, F. “Attempts to Correlate Late Quaternary Climatic Records Between San Francisco Bay, the Sacramento-San Joaquin Delta, and the Mokelumne River, California.” University of Delaware. May 1980. Print.

Atwater, Brian, F. “Geologic maps of the Sacramento-San Joaquin Delta.” California: U. S. Geological Survey Miscellaneous Field Studies MF-1401, scale 1:24,000, 20-21 sheets. 1981. Print.

Bowles, Joseph E. “Foundation Analysis and Design.” McGraw Hill, 5th Ed. 1996. Print.

Cosby, S.W., and Carpenter, E.J. “Soil Survey of the Lodi Area.” California, U.S. Dept. of Agriculture: Bureau of Chemistry and Soils, Series 1932, no. 14, p.52. 1937. Print.

Duncan, James M. “Manual for Geotechnical Engineering Reliability Calculations.” Virginia Polytechnic. 1999. Print.

State of California, The Natural Resources Agency, Department of Water Resources. “Urban Levee Design Criteria.” May 2012.

State of California, Regional Flood Management Planning, Department of Water Resources. “Mid-San Joaquin River Region Regional Flood Atlas-Draft.” http://www.water.ca.gov/cvfmp/regionalplan/docs/05-Mid-SJ-River-Atlas-May2013.pdf. Web.

Fugro William Lettis & Associates, Inc. “Surficial Geologic Map and Geomorphic Assessment, San Joaquin Area Flood Control Agency (SJAFCA) area, San Joaquin County, California.” Prepared for URS Corporation. March 2012. Print.

Fugro William Lettis & Associates, Inc. “Surficial Geologic Map and Geomorphic Assessment, California Department of Water Resources Urban Levees, Reclamation District 404 (RD-404) Study Area, San Joaquin County, California.” Prepared for URS Corporation. August 2010. Print.

Fugro William Lettis & Associates, Inc. “Surficial Geologic Map and Geomorphic Assessment, California Department of Water Resources Urban Levees, Reclamation District 17 (RD-17) Study Area, San near Stockton, California.” Prepared for URS Corporation. June 2010. Print.

Harr, Milton E. “Reliability-Based Design in Civil Engineering.” Dover Publishing. 1997. Print.

Hosford Scheirer, A., and Magoon, L.B. “Age, Distribution, and Stratigraphy Relationship of Rock Units in the San Joaquin Basin Province, California, in Hosford Scheirer, A., ed., Petroleum systems and geologic assessment of oil and gas in the San Joaquin Basin

Province, California." U.S. Geological Survey Professional Paper 1713, chapter 5, p, 1-107. 2008a. Print.

Kleinfelder. "Draft Geotechnical Data Report, Lincoln Village Study Area." Urban Levee Geotechnical Evaluations Program Contract 4600008102, Lower San Joaquin River. Prepared for Department of Water Resources, Division of Flood Management. June 2012. Print.

Kleinfelder. "Geotechnical Assessment Report, South NULE Study Area." Volume 2 of 4, Levee Segment Assessment Summaries South NULE Area 1-San Joaquin County – Non-Urban Levee Evaluations Project Contract 4600008102, Task Order K104. Prepared for Department of Water Resources, Division of Flood Management. April 2011. Print.

Kleinfelder. "Geotechnical Assessment Report, South NULE Study Area." Volume 1 of 4 – Non-Urban Levee Evaluations Project Contract 4600008102, Task Order K104. Prepared for Department of Water Resources, Division of Flood Management. May 2011. Print.

Kleinfelder. "Draft Geotechnical Data Report, Brookside Study Area." Urban Levee Geotechnical Evaluations Program Contract 4600008102, Lower San Joaquin River. Prepared for Department of Water Resources, Division of Flood Management. August 2012. Print.

Kleinfelder. "Technical Memorandum, Surficial Quaternary Geologic Map and Geomorphic Assessment, Brookside Study Area, San Joaquin County, California." Non-Project Urban Levee Geotechnical Evaluations (ULE) Project Contract 4600008102, Take Order K08. Prepared for Department of Water Resources, Division of Flood Management. June 2012. Print.

Kleinfelder. "Technical Memorandum, Surficial Quaternary Geologic Map and Geomorphic Assessment, Lincoln Village Study Area, San Joaquin County, California." Non-Project Urban Levee Geotechnical Evaluations (ULE) Project Contract 4600008102, Take Order K08. Prepared for Department of Water Resources, Division of Flood Management. June 2012. Print.

Marchand, D.E., and Atwater, B.F. "Late Cenozoic Stratigraphic Units, Northeastern San Joaquin Valley, California." U.S. Geological Survey Bulletin, v. 1470. 1981. Print.

Marchand, D.E., and Atwater, B.F. "Preliminary Geologic Map Showing Quaternary Deposits of the Lodi Quadrangle, California." U.S. Geological Survey Open File Report OFR 79-993, scale 1:62,500. 1979. Print.

Mualchin, L. "A Technical Report to Accompany the CALTRANS California Seismic Hazard Map. Prepared for CALTRANS." Office of Earthquake Engineering, 70 pp. 1996. Print.

Seed, H.B. and Harder, L.F. "SPT-Based Analysis of Cycle Pore Pressure Generation and Undrained Residual Strength." Bi-Tech Publishing Ltd., Vol. 2. 1990. Print.

Unruh, J., and Krug, K. “Assessment and documentation of transpressional structures, northeastern Diablo Range, for the Quaternary fault map database: collaborative research with William Lettis & Associates, Inc. and the U.S. Geological Survey.” June 2007. Print.

URS Corporation. "Phase 1 Geotechnical Evaluation Report (P1GER), San Joaquin Area Flood Control Agency Study Area Calaveras River Drainage.” Urban Levee Geotechnical Evaluations Program Contract 4600007418, Prepared for Department of Water Resources, Division of Flood Management. July 2011. Print.

URS Corporation. "Phase 1 Geotechnical Data Report (P1GDR), San Joaquin Area Flood Control Agency (SJAFCA) Study Area Calaveras River Drainage.” Urban Levee Geotechnical Evaluations Program Contract 4600007417, Task Order 36, Prepared for Department of Water Resources, Division of Flood Management. January 2010. Print.

URS Corporation. "Draft Supplemental Geotechnical Data Report (SGDR), San Joaquin Area Flood Control Agency Calaveras River Study Area.” Urban Levee Geotechnical Evaluations Program Contract 4600008108, Prepared for Department of Water Resources, Division of Flood Management. March 2013. Print.

URS Corporation. "Draft Supplemental Geotechnical Data Report (SGDR), Reclamation District 404 Study Area.” Urban Levee Geotechnical Evaluations Program Contract 4600007418, Prepared for Department of Water Resources, Division of Flood Management. April 2011. Print.

URS Corporation. "Phase 1 Geotechnical Evaluation Report (P1GER), Reclamation District 17 (RD 17).” Urban Levee Geotechnical Evaluations Program Contract 4600007418, Task Order 21, Prepared for Department of Water Resources, Division of Flood Management. December 2007. Print.

URS Corporation. "Phase 1 Geotechnical Data Report (P1GDR), Reclamation District 17 (RD 17) Study Area.” Urban Levee Geotechnical Evaluations Program Contract 4600007418, Prepared for Department of Water Resources, Division of Flood Management. September 2008. Print.

URS Corporation. "Draft Supplemental Geotechnical Data Report (SGDR), Reclamation District 17.” Urban Levee Geotechnical Evaluations Program Contract 4600007418, Prepared for Department of Water Resources, Division of Flood Management. December 2010. Print.

U.S. Army Corps of Engineers, Engineer Manual (EM) 1110-2-1902, “Slope Stability.” October 2003. Print.

U.S. Army Corps of Engineers, Engineer Manual (EM) 1110-2-1913, “Design and Construction of Levees.” April 2000. Print.

U.S. Army Corps of Engineers, Engineer Manual (ER) 1110-2-1150, “Engineering and Design for Civil Works Projects.’ August 1999. Print.

U.S. Army Corps of Engineers, Technical Memorandum (TM) No. 3-424, "Investigation of Underseepage and its Control, Lower Mississippi River Levees." October 1956. Print.

U.S. Army Corps of Engineers, Engineer Technical Letter (ETL) 1110-2-556, "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies." May 1999. Print.

U.S. Army Corps of Engineers, Engineer Technical Letter (ETL) 1110-2-561, "Reliability Analysis and Risk Assessment for Seepage and Slope Stability Failure Modes for Embankment Dams." January 2006. Print.

U.S. Army Corps of Engineers, Engineer Technical Letter (ETL) 1110-2-569, "Design Guidance for Levee Underseepage." May 2005. Print.

U.S. Army Corps of Engineers, Engineer Technical Letter (ETL) 1110-2-571, "Guidelines for Landscaping Planting and Vegetation Management at Levees, Floodwalls, Embankments Dams, and Appurtenant Structures." April 2009. Print.

U.S. Army Corps of Engineers, Sacramento District, "Design Manual for Levee Construction", October 1969.

U.S. Army Corps of Engineers, Sacramento District, "Geotechnical Levee Practice REFP10L0." April 2008. Print.

U.S. Army Corps of Engineers, Sacramento District, "Geotechnical Levee Practice SOP-EDG-03." July 2004. Print.

Weight, Willis D. and Sonderegger, John L. "Manual of Applied Field Hydrology." McGraw-Will Professional. 2001. Print.

Wright, Stephen G. "UTEXAS4, A Computer Program for Slope Stability Calculations." Austin, Texas. May 1999. Print.

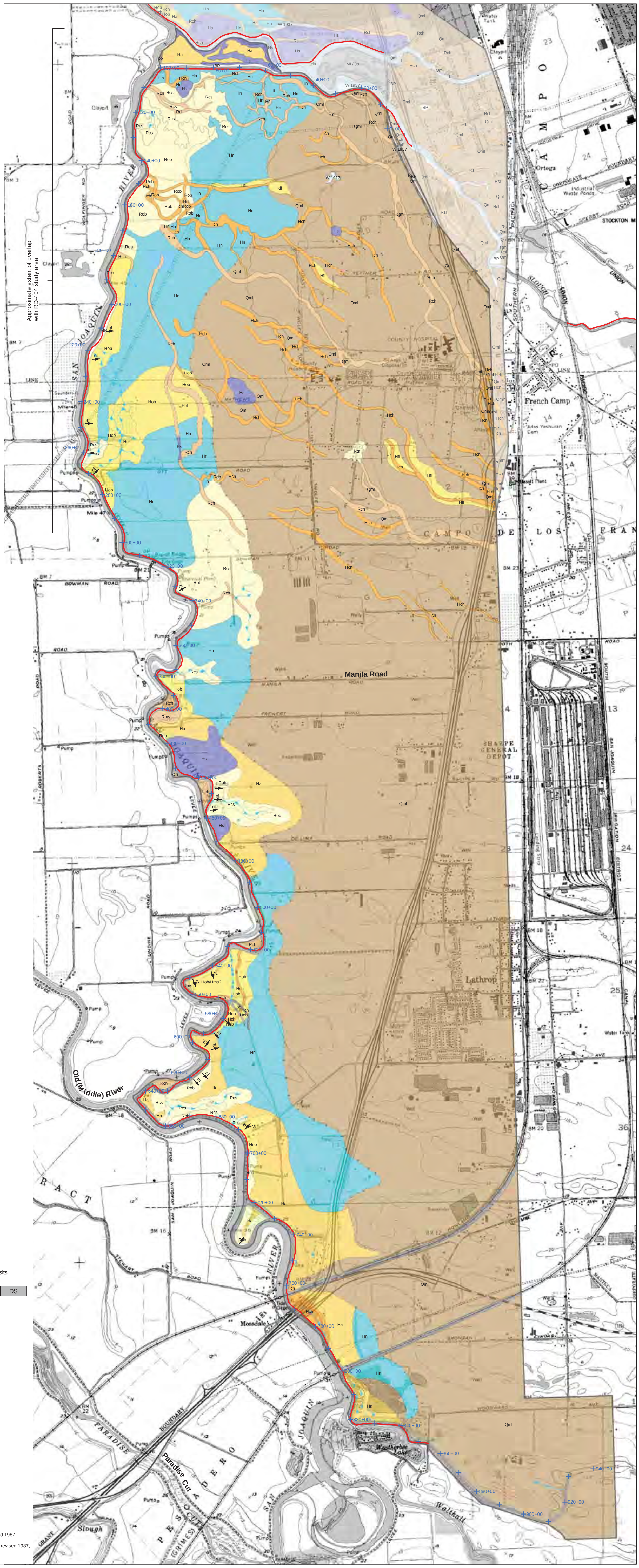
U.S. Department of the Interior, Bureau of Reclamation. Central Valley Project. http://www.usbr.gov/projects/Project.jsp?proj_Name=Central+Valley+Project. Web.

San Joaquin County Resource Conservation District. http://sjcrcd.org/articles/MokP07.pdf. Web.

LOWER SAN JOAQUIN RIVER
FEASIBILITY STUDY

GEOTECHNICAL REPORT

ENCLOSURE E1
GEOMORPHOLOGY MAPS



This map shows surficial geologic deposits and levees as they existed in 1937. Map units and boundaries are drawn by interpretation of historical aerial photography supplemented by data from historical maps and surveys. For reference, the mapping is superimposed on a modern U.S. Geological Survey 7.5' topographic base map.
Screened back semi-transparent mapping shown on this plate is from RD-404 study area, which is not assessed in this investigation.
For clarity, only the surficial geologic map units of this study appear in the explanation.
See accompanying report for complete descriptions of map units, process descriptions and methodology.

Explanation

—— – – ?······	Geologic contact; dashed where approximate, dotted where concealed, queried where uncertain. Solid contacts accurate to within 100' of line shown on map, dashed contacts accurate to within about 250' on either side of the line.
►—►—►	Erosional channel, generally <100 ft in width; likely contains unsorted soil.
–·–·–·–	Swale; topographic lineament associated with meander scroll topography low.
——	Urban Project Levee
↘ nl	Natural levee; arrow indicates slope direction away from channel.
+ 860+00	Approximate RD-17 Levee stationing distance in feet.

W 1937	Water, circa 1937.
W 1913	Water, circa 1913.
BP	Borrow pit present in 1937.

Geologic Units

	L	Levee (made of artificial fill), circa 1937.
	R	Road embankment (made of artificial fill), circa 1937.
	DS	Dredge spoils; material from dredging operations within channels.
HISTORICAL	Rob	Overbank deposits; sand, silt, and clay; deposited during high-stage water flow, overtopping channel banks.
	Rcs	Crevasse splay deposits; fine to coarse sand, with minor lenses of clay deposited from breaching of natural or artifical levees.
	Rdf	Distributary fan deposits; sand, silt and clay.
	Rch	Channel deposits; well sorted sands and fine gravels.
	Rms	Channel meander scroll deposits; sand, silt and clay from lateral channel migration.
	Rsl	Slough deposits; silt, clay, and sand, fining upward facies, low-energy channel deposits.
HOLOCENE	Hob/Hms?	Thin veneer of overbank deposits overlying probable channel meander scroll deposits.
	Hob	Overbank deposits; sand, silt, and clay; deposited during high-stage water flow, overtopping channel banks.
	Hcs	Crevasse splay deposits; fine sand and silt with clay deposited from breaching of natural levees.
	Hdf	Distributary fan deposits; sand, silt and clay.
	Hfl	Fan channel levee deposits; relatively coarser (sandier and siltier) deposits accumulating next to alluvial fan channels.
	Hch	Channel deposits; sorted sands and silts; fining upward.
	Hms	Channel meander scroll deposits; sand, silt and clay from lateral channel migration.
	Ha	Alluvial deposits; undifferentiated; sand, silt, and minor lenses of gravel; under cultivation in 1937.
	Hn	Basin deposits; fine sand, silt and clay, under cultivation in 1937.
	Hs	Marsh deposits; silt and clay, likely organic-rich; perennially or seasonally submerged on 1937 photography.
PLEISTOCENE	Qml	Modesto Formation; lower member; unconsolidated gravel, sand, silt, and clay; Alluvial fan deposits of the Stanislaus River.

Stratigraphic Correlation Chart

Time	Depositional Environment			
Epoch	Channel deposits	Floodplain and alluvial-fan deposits	Flood basin deposits	Cultural deposits
Historical	Rch Rms Rsl	Rcs Rdf Rob	Hn Hs	L R DS
Holocene	Hch Hms	Ha Hob Hcs Hfl Hdf		
Latest Pleistocene		Qml		

DWR URBAN LEVEE PROJECT
WLA WORK ORDER W02 STOCKTON

Surficial Geologic Map of the Eastern Side of the San Joaquin River, along RD-17 Levee System near Stockton and Lathrop, California

Fugro William Lettis & Associates, Inc.

Plate 1

2083_DWRLevees_RD17_Stockton_Plate.mxd MGT 06/16/2010

N

1:20,000

0 2,000 ft

0 600 m

Map projection: NAD83 UTM Zone 10N

Topographic base USGS quadrangles:
Lathrop topographic quadrangle, published 1952, revised 1987;
map scale 1:24,000, five foot contour interval.
West Stockton topographic quadrangle published 1968, revised 1987;
map scale 1:24,000, five foot contour interval.

Plate 1 - Surficial Geologic Map of French Camp Slough and the Lower San Joaquin River along RD-404 Levee System, near Stockton, California

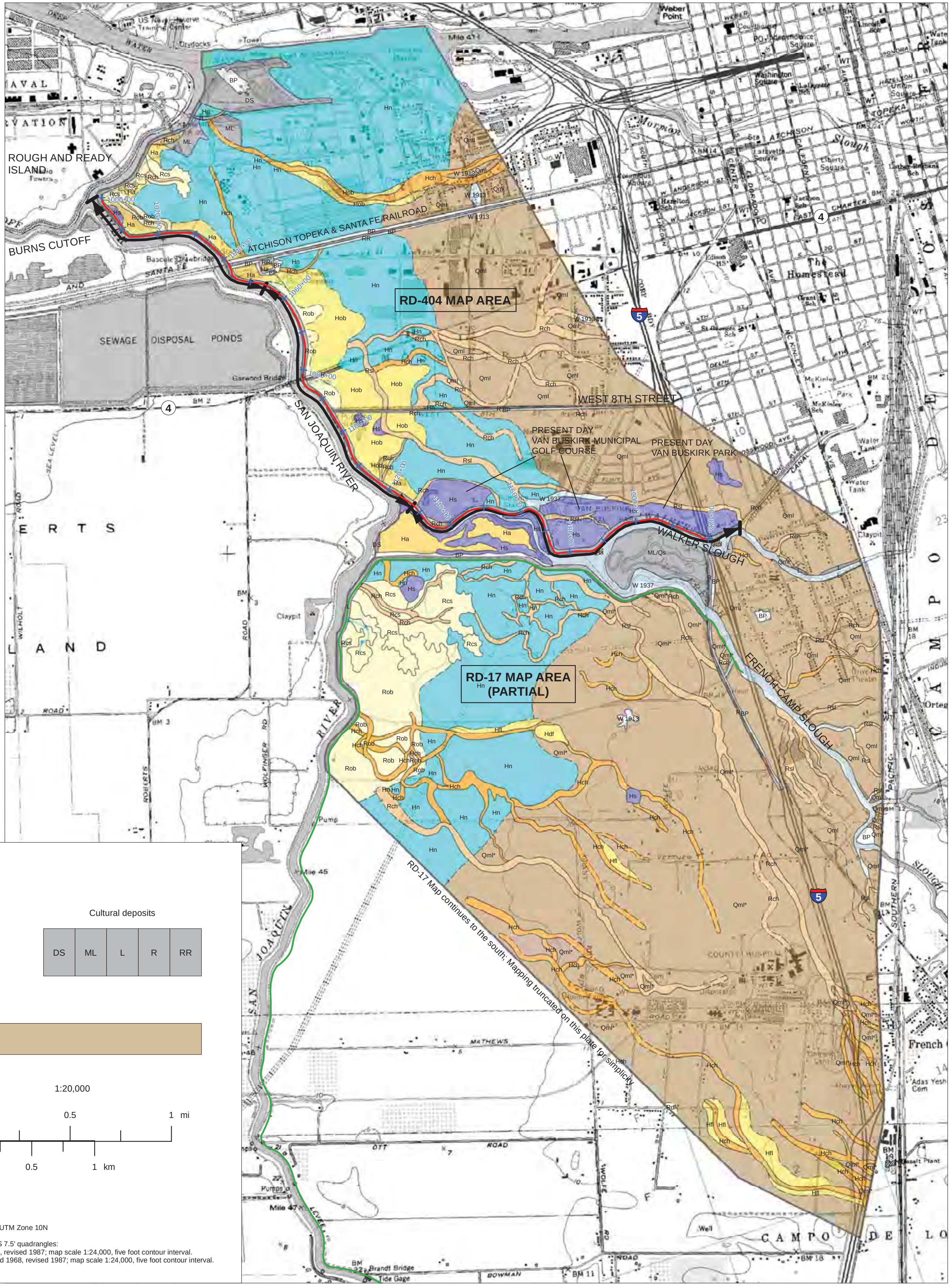
This map shows surficial geologic deposits and levees as they existed in 1937. Map units and boundaries are drawn by interpretation of historical aerial photography supplemented by data from historical maps and surveys. For reference, the mapping is superimposed on modern U.S. Geological Survey 7.5' topographic base maps (individual maps referenced below). See accompanying report for complete descriptions of map units, process descriptions and methodology.

Explanation

Symbol	Description
—— – – –? ·······	Geologic contact; dashed where approximate, dotted where concealed, queried where uncertain. Solid contacts are accurate to within about 100 feet on either side of the line shown on the map, dashed contacts are accurate to within about 200 feet
►—►—►	Centerline of channel <30ft in width. Likely contains sorted sands and silts. Arrows indicate direction of flow.
├◄SJR-II►┤	Geomorphic Reach described in text.
🟥	Urban Project Levee (RD-404 area)
🟩	Urban Project Levee (RD-17 area)
+ 1180+00	Approximate RD404 Levee stationing distance in feet (draft, 03/11/08 version).
W 1913	Water, circa 1913.
W 1937	Water, circa 1937.
BP	Borrow pit present in 1937.

Geologic Units

Age	Unit	Description
HISTORICAL	L	Levee (made of artificial fill), circa 1937.
	R	Road embankment (made of artificial fill), circa 1937.
	RR	Railroad embankment (made of artificial fill), circa 1937.
	DS	Dredge spoils; material from dredging operations within channels.
	ML	Made land; fill material of local and non-local sources: sand, silt, organics, garbage, etc.
	ML/Qs	Made land overlying marsh deposits.
	Rob	Overbank deposits; sand, silt, and clay; deposited during high-stage water flow, overtopping channel banks.
	Rcs	Crevasse splay deposits; fine to coarse sand, with minor lenses of clay deposited from breaching of natural or artifical levees.
	Rdf	Distributary fan deposits; sand, silt and clay.
	Rch	Channel deposits; well sorted sand and trace gravel.
	Rsl	Slough deposits; sand, silt and clay, fining upward facies, low-energy channel deposits; tidally-influenced in RD-404.
HOLOCENE	Hob	Overbank deposits; sand, silt, and clay; deposited during high-stage water flow, overtopping channel banks.
	Hdf	Distributary fan deposits; sand, silt and clay.
	Hch	Channel deposits; sorted sand and silt; fining upward.
	Ha	Alluvial deposits; undifferentiated; sand, silt, and minor lenses of gravel; under cultivation in 1937.
	Hn	Basin deposits; sand, silt and clay, dark yellow to dark yellowish brown, under cultivation in 1937.
	Hs	Marsh deposits; silt and clay, likely organic-rich; perennially or seasonally submerged on 1937 photography.
PLEISTOCENE	Qml	Modesto Formation; lower member; unconsolidated gravel, sand, silt, and clay. (Qml*) Asterisk denotes Modesto Formation of Stanislaus Alluvial Fan.



Stratigraphic Correlation Chart

Time	Depositional Environment													
Epoch	Channel deposits		Floodplain and alluvial-fan deposits				Basin deposits		Cultural deposits					
Historical	Rch	Rsl		Rob	Rcs	Rdf			DS	ML	L	R	RR
Holocene	Hch		Ha	Hob	Hdf		Hn	Hs					
Pleistocene	Qml												

1:20,000

0 0.5 1 mi

0 0.5 1 km

N

Map projection: NAD83 UTM Zone 10N

Topographic base USGS 7.5' quadrangles:
Lathrop, published 1952, revised 1987; map scale 1:24,000, five foot contour interval.
West Stockton, published 1968, revised 1987; map scale 1:24,000, five foot contour interval.

Surficial Geologic Map of French Camp Slough and the Lower San Joaquin River along RD-404 Levee System, near Stockton, California

Fugro William Lettis & Associates, Inc.

Plate 1

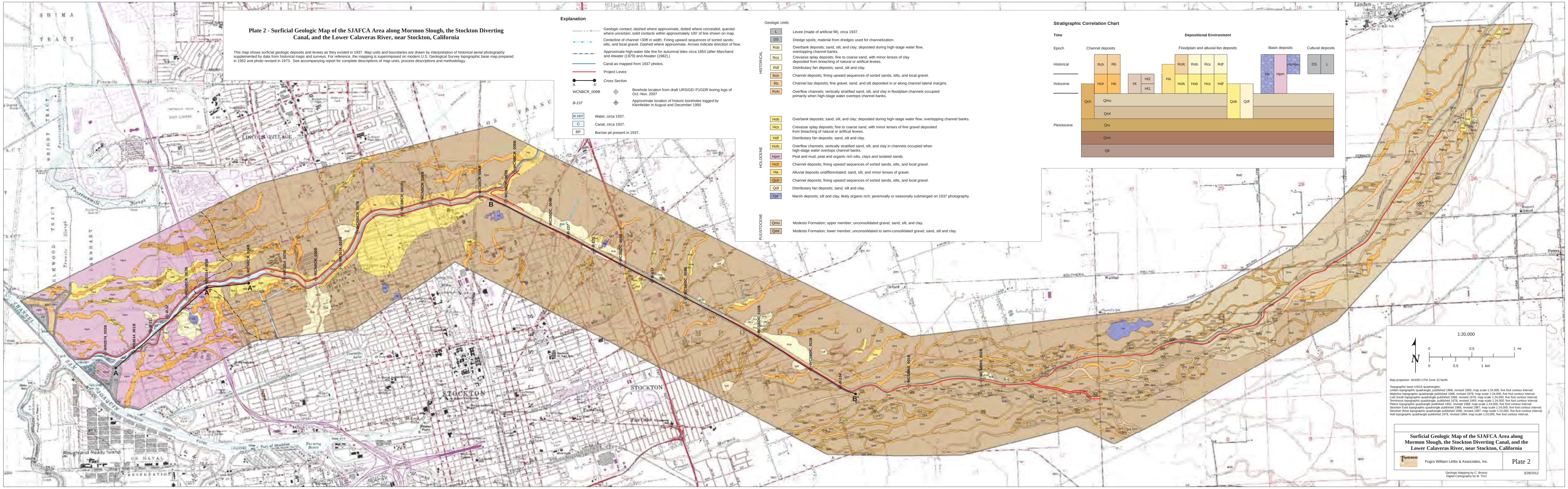
1965_RD404_Plate.mxd

Geologic Mapping by C. Brossy and J. Pearce
Digital Cartography by M. Ticci

12/14/2010

Plate 2 - Surficial Geologic Map of the SJAFCA Area along Mormon Slough, the Stockton Diverting Canal, and the Lower Calaveras River, near Stockton, California

This map shows surficial geologic deposits and levees as they existed in 1937. Map units and boundaries are drawn by interpretation of historical aerial photography supplemented by data from historical maps and surveys. For reference, the mapping is superimposed on modern U.S. Geological Survey topographic base map prepared in 1952 and photo revised in 1973. See accompanying report for complete descriptions of map units, process descriptions and methodology.



Explanation

——— – – ? ········	Geologic contact; dashed where approximate, dotted where concealed, queried where uncertain; solid contacts within approximately 100' of line shown on map.
▸— ▸ - ▸	Centerline of channel <30ft in width. Fining upward sequences of sorted sands, silts, and local gravel. Dashed where approximate. Arrows indicate direction of flow.
-·—·—·—	Approximate high-water tide line for autumnal tides circa 1850 (after Marchand and Atwater (1979) and Atwater (1982).)
———	Canal as mapped from 1937 photos.
———	Project Levee
A ●——● A'	Cross Section
WCNBCR_009B ⌖	Borehole location from draft URS/GEI P1GDR boring logs of Oct.-Nov. 2007
B-137 ⌖	Approximate location of historic boreholes logged by Kleinfelder in August and December 1990
W 1937	Water, circa 1937.
C	Canal, circa 1937.
BP	Borrow pit present in 1937.

Geologic Units

HISTORICAL	L	Levee (made of artificial fill), circa 1937.
	DS	Dredge spoils; material from dredges used for channelization.
	Rob	Overbank deposits; sand, silt, and clay; deposited during high-stage water flow, overtopping channel banks.
	Rcs	Crevasse splay deposits; fine to coarse sand, with minor lenses of clay deposited from breaching of natural or artifical levees.
	Rdf	Distributary fan deposits; sand, silt and clay.
	Rch	Channel deposits; fining upward sequences of sorted sands, silts, and local gravel.
	Rb	Channel bar deposits; fine gravel, sand, and silt deposited in or along channel lateral margins.
	Rofc	Overflow channels; vertically stratified sand, silt, and clay in floodplain channels occupied primarily when high-stage water overtops channel banks.
HOLOCENE	Hob	Overbank deposits; sand, silt, and clay; deposited during high-stage water flow, overtopping channel banks.
	Hcs	Crevasse splay deposits; fine to coarse sand, with minor lenses of fine gravel deposited from breaching of natural or artifical levees.
	Hdf	Distributary fan deposits; sand, silt and clay.
	Hofc	Overflow channels; vertically stratified sand, silt, and clay in channels occupied when high-stage water overtops channel banks.
	Hpm	Peat and mud; peat and organic rich silts, clays and isolated sands.
	Hch	Channel deposits; fining upward sequences of sorted sands, silts, and local gravel.
	Ha	Alluvial deposits undifferentiated; sand, silt, and minor lenses of gravel.
	Qch	Channel deposits; fining upward sequences of sorted sands, silts, and local gravel.
	Qdf	Distributary fan deposits; sand, silt and clay.
	Qs	Marsh deposits; silt and clay, likely organic-rich; perennially or seasonally submerged on 1937 photography.
PLEISTOCENE	Qmu	Modesto Formation; upper member; unconsolidated gravel, sand, silt, and clay.
	Qml	Modesto Formation; lower member; unconsolidated to semi-consolidated gravel, sand, silt and clay.

Map projection: NAD83 UTM Zone 10 North

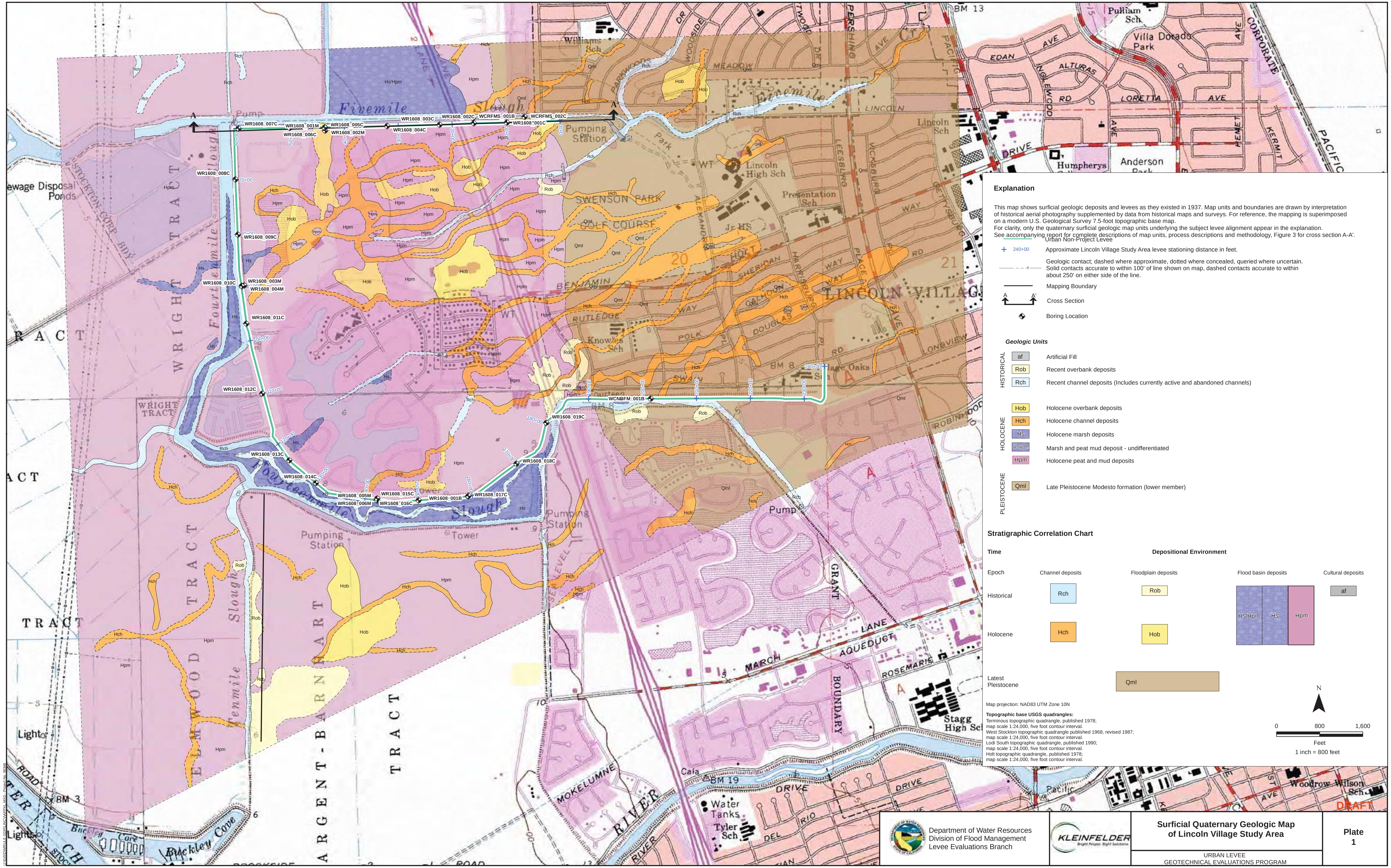
Topographic base USGS quadrangles:
Linden topographic quadrangle, published 1968, revised 1993; map scale 1:24,000, five foot contour interval.
Waterloo topographic quadrangle published 1968, revised 1978; map scale 1:24,000, five foot contour interval.
Lodi South topographic quadrangle published 1968, revised 1976; map scale 1:24,000, five foot contour interval.
Terminous topographic quadrangle, published 1978, revised 1993; map scale 1:24,000, five foot contour interval.
Peters topographic quadrangle published 1952, revised 1968; map scale 1:24,000, five foot contour interval.
Stockton East topographic quadrangle published 1968, revised 1987; map scale 1:24,000, five foot contour interval.
Stockton West topographic quadrangle published 1968, revised 1987; map scale 1:24,000, five foot contour interval.
Holt topographic quadrangle published 1978, revised 1994; map scale 1:24,000, five foot contour interval.

Surficial Geologic Map of the SJAFCA Area along Mormon Slough, the Stockton Diverting Canal, and the Lower Calaveras River, near Stockton, California	
Fugro William Lettis & Associates, Inc.	Plate 2

Geologic Mapping by C. Brossy
Digital Cartography by M. Ticci

3/28/2012

L:\Projects\1881_URS\GIS\MXD\SJAFCA_Mormon_Slough_Rev2012.mxd



Explanation

This map shows surficial geologic deposits and levees as they existed in 1937. Map units and boundaries are drawn by interpretation of historical aerial photography supplemented by data from historical maps and surveys. For reference, the mapping is superimposed on a modern U.S. Geological Survey 7.5-foot topographic base map.
For clarity, only the quaternary surficial geologic map units underlying the subject levee alignment appear in the explanation.
See accompanying report for complete descriptions of map units, process descriptions and methodology, Figure 3 for cross section A-A'.

―	Urban Non-Project Levee
+ 240+00	Approximate Lincoln Village Study Area levee stationing distance in feet.
――― – – -?········	Geologic contact; dashed where approximate, dotted where concealed, queried where uncertain. Solid contacts accurate to within 100' of line shown on map, dashed contacts accurate to within about 250' on either side of the line.
―	Mapping Boundary
A ↑___↑ A'	Cross Section
⊕	Boring Location

Geologic Units

HISTORICAL	af	Artificial Fill
	Rob	Recent overbank deposits
	Rch	Recent channel deposits (Includes currently active and abandoned channels)
HOLOCENE	Hob	Holocene overbank deposits
	Hch	Holocene channel deposits
	Hs	Holocene marsh deposits
	Hs/Hpm	Marsh and peat mud deposit - undifferentiated
	Hpm	Holocene peat and mud deposits
PLEISTOCENE	Qml	Late Pleistocene Modesto formation (lower member)

Stratigraphic Correlation Chart

Time	Depositional Environment			
Epoch	Channel deposits	Floodplain deposits	Flood basin deposits	Cultural deposits
Historical	Rch	Rob	Hs/Hpm Hs Hpm	af
Holocene	Hch	Hob		
Latest Pleistocene		Qml		

Map projection: NAD83 UTM Zone 10N

Topographic base USGS quadrangles:
Terminous topographic quadrangle, published 1978;
map scale 1:24,000, five foot contour interval.
West Stockton topographic quadrangle published 1968, revised 1987;
map scale 1:24,000, five foot contour interval.
Lodi South topographic quadrangle, published 1990;
map scale 1:24,000, five foot contour interval.
Holt topographic quadrangle, published 1978;
map scale 1:24,000, five foot contour interval.

N

0 800 1,600
Feet
1 inch = 800 feet

Department of Water Resources
Division of Flood Management
Levee Evaluations Branch

KLEINFELDER
Bright People. Right Solutions.

Surficial Quaternary Geologic Map of Lincoln Village Study Area

URBAN LEVEE
GEOTECHNICAL EVALUATIONS PROGRAM

Plate
1

Draft Geotechnical Data Report

LOWER SAN JOAQUIN RIVER
FEASIBILITY STUDY

GEOTECHNICAL REPORT

ENCLOSURE E2
CALCULATION PACKAGE

LOWER SAN JOAQUIN RIVER
FEASIBILITY STUDY

WITHOUT PROJECT
STRENGTH PARAMETERS

LOWER SAN JOAQUIN RIVER
GEOTECHNICAL ANALYSIS
SJR - REACH LR-1
ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR0017_036B	1	Clay Levee	1.00E-06	4	2.50E-07	0.00284	0.00071	28	50	120
	2	Silt Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	28	0	120
	3	Clay Blanket	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120
	4	Silty Sand	1.00E-03	4	2.50E-04	2.83500	0.70875	32	0	125
	5	Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120
	6	Sand	5.00E-03	4	1.25E-03	14.17500	3.54375	32	0	125

LOWER SAN JOAQUIN RIVER
GEOTECHNICAL ANALYSIS
SJR - REACH LR-2
ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR0017_052B	1	Levee Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	28	100	120
	2	Silty Sand Blanket	1.00E-05	4	2.50E-06	0.02835	0.00709	30	0	125
	3	Poorly Graded Sand wSilt	1.00E-04	4	2.50E-05	0.28350	0.07088	32	0	130
	4	Foundation Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	28	100	120

LOWER SAN JOAQUIN RIVER
GEOTECHNICAL ANALYSIS
SJR - REACH LR-3
ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR0017_085B	1	Poorly Graded Sand wSilt	5.00E-03	4	1.25E-03	14.17500	3.54375	32	0	125
	2	Clayey Sand	5.00E-05	4	1.25E-05	0.14175	0.03544	30	50	125
	3	Poorly Graded Sand	5.00E-03	4	1.25E-03	14.17500	3.54375	32	0	125
	4	Sandy Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	28	140	120
	5	Silty Sand	1.00E-04	4	2.50E-05	0.28350	0.07088	30	0	120
	6	Poorly Graded Sand wSilt	1.00E-02	4	2.50E-03	28.35000	7.08750	32	0	125
	7	Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120

LOWER SAN JOAQUIN RIVER
GEOTECHNICAL ANALYSIS
SJR - REACH LR-4
ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR0017_100C	1	Clayey Sand	1.00E-04	4	2.50E-05	0.28350	0.07088	30	50	125
	2	Lean Clay Blanket	1.00E-06	4	2.50E-07	0.00284	0.00071	28	100	120
	3	Poorly Graded Sand w/Silt	1.00E-03	4	2.50E-04	2.83500	0.70875	32	0	130
	4	Lean Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120

FRENCH CAMP SLOUGH
GEOTECHNICAL ANALYSIS
FRENCH CAMP - REACH FL-1
ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR0017_007B	1	Levee Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	28	100	120
	2	Lean Clay Blanket	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120
	3	Foundation Silty Sand	5.00E-03	4	1.25E-03	14.17500	3.54375	32	0	125
	4	Foundation Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120

FRENCH CAMP SLOUGH
GEOTECHNICAL ANALYSIS
FRENCH CAMP - REACH FR-1
ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR0404_042B	1	Clay Levee	1.00E-06	4	2.50E-07	0.00284	0.00071	28	100	120
	2	Silt Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	30	0	120
	3	Clayey Sand Blanket	1.00E-05	4	2.50E-06	0.02835	0.00709	28	50	125
	4	Silty Sand	1.00E-04	4	2.50E-05	0.28350	0.07088	32	0	125
	5	Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120
	6	Silty Sand	1.00E-03	4	2.50E-04	2.83500	0.70875	32	0	125
	7	Silt and Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120

STOCKTON DIVERTING CANAL
GEOTECHNICAL ANALYSIS
DIVERTING CANAL - REACH SL-1
ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WCSBDC_004B	1	Sandy Lean Clay Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	34	100	115
	2	Lean Clay Blanket	1.00E-05	4	2.50E-06	0.02835	0.00709	31	150	120
	3	Silty Sand	1.00E-04	4	2.50E-05	0.28350	0.07088	35	0	120
	4	Sandy Silt	1.00E-04	4	2.50E-05	0.28350	0.07088	35	0	120
	5	Silty Sand (more permeable)	1.00E-03	4	2.50E-04	2.83500	0.70875	35	0	120
	6	Lean Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	120

STOCKTON DIVERTING CANAL
GEOTECHNICAL ANALYSIS
DIVERTING CANAL - REACH SL-2
ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WCSBDC_025C	1	Sandy Silt Levee	1.00E-04	4	2.50E-05	0.28350	0.07088	34	0	115
	2	Lean Clay/Silty Lean Clay Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	31	150	115
	3	Lean Clay/Silty Lean Clay Blanket	1.00E-05	4	2.50E-06	0.02835	0.00709	31	150	115
	4	Sand to Silty Sand	5.00E-04	4	1.25E-04	1.41750	0.35438	35	0	125
	5	Lean Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	115
	6	Sandy Silt	1.00E-04	4	2.50E-05	0.28350	0.07088	35	0	120

CALAVERAS RIVER

GEOTECHNICAL ANALYSIS

CALAVERAS RIVER - REACH CL-1/CL-2

ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WCSBCR_004B	1	Silt Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	34	100	115
	2	Silt Blanket	1.00E-05	4	2.50E-06	0.02835	0.00709	31	150	115
	3	Lean Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	120
	4	Sandy Silt Foundation	1.00E-04	4	2.50E-05	0.28350	0.07088	35	0	120

CALAVERAS RIVER

GEOTECHNICAL ANALYSIS

CALAVERAS RIVER - REACH CR-1/CR-2

ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WCNBCR_010A	1	Lean Clay wSand Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	34	100	120
	2	Sandy Silt	1.00E-05	4	2.50E-06	0.02835	0.00709	31	150	115
	3	Lean Clay wSand	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	120
	4	Pooly Graded Sand wSilt	2.10E-03	4	5.25E-04	5.95350	1.48838	35	0	120
	5	Silt	1.00E-05	4	2.50E-06	0.02835	0.00709	31	150	115

CALAVERAS RIVER
GEOTECHNICAL ANALYSIS
CALAVERAS RIVER - REACH D-4
ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WCNBCR_003B	1	Sandy Silt Levee	1.00E-04	4	2.50E-05	0.28350	0.07088	31	0	110
	2	Lean Clay wSand to CH Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	34	100	110
	3	FAT Clay wSand Blanket	1.00E-05	4	2.50E-06	0.02835	0.00709	27	50	110
	4	Sandy Silt	1.00E-04	4	2.50E-05	0.28350	0.07088	31	0	115
	5	Pooly Graded Sand wSilt	6.40E-04	4	1.60E-04	1.81440	0.45360	32	0	120
	6	Lean Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	120

CALAVERAS RIVER
GEOTECHNICAL ANALYSIS
CALAVERAS RIVER - REACH D-5
ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR1614_004B	1	Silt to Sandy Silt Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	31	0	110
	2	Lean Clay Levee	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	115
	3	Lean Clay Blanket	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	115
	4	Silty Sand	1.00E-04	4	2.50E-05	0.28350	0.07088	35	0	120
	5	Lean Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	115

SAN JOAQUIN RIVER
GEOTECHNICAL ANALYSIS
LSJ RIVER - DELTA FRONT BROOKSIDE REACH D-BS
ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR2074_015C	1	Clay Levee	4.00E-06	4	1.00E-06	0.01134	0.00284	30	50	120
	2	Farm Levee	4.00E-06	4	1.00E-06	0.01134	0.00284	30	50	110
	3	Organic Soil	4.00E-06	4	1.00E-06	0.01134	0.00284	26	50	80
	4	Blanket	4.00E-06	4	1.00E-06	0.01134	0.00284	30	100	120
	5	Silty Sand	4.00E-04	4	1.00E-04	1.13400	0.28350	32	0	125
	6	Clay	4.00E-06	4	1.00E-06	0.01134	0.00284	30	100	120
	7	Poorly graded Sand w/silt	1.00E-03	4	2.50E-04	2.83500	0.70875	34	0	125
	8	Silt	4.00E-06	4	1.00E-06	0.01134	0.00284	32	0	120
	9	Silty Sand	4.00E-04	4	1.00E-04	1.13400	0.28350	32	0	125

SAN JOAQUIN RIVER
GEOTECHNICAL ANALYSIS
LSJ RIVER - DELTA FRONT LINCOLN VILLAGE REACH D-LV
ANALYSIS PARAMETERS SUMMARY

Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR1608_001B	1	Clay Levee	4.00E-06	4	1.00E-06	0.01134	0.00284	27	50	120
	2	Organic Soil	1.00E-04	10	1.00E-05	0.28350	0.02835	28	25	80
	3	Blanket	4.00E-06	4	1.00E-06	0.01134	0.00284	28	50	120
	4	Silty Sand	4.00E-04	4	1.00E-04	1.13400	0.28350	32	0	125
	5	Poorly graded Sand w/silt	1.00E-03	4	2.50E-04	2.83500	0.70875	34	0	125
	6	Foundation Clay	4.00E-06	4	1.00E-06	0.01134	0.00284	30	100	120
	7	Deep Clay Layer	4.00E-06	4	1.00E-06	0.01134	0.00284	30	100	120

LOWER SAN JOAQUIN RIVER
FEASIBILITY STUDY

RESULTS OF WITHOUT PROJECT
SEEPAGE AND STABILITY ANALYSES

SEEPAGE/STABILITY ANALYSES LOWER SAN JOAQUIN RIVER REACH LR-1

STA. 1292+00

Water Level		USACE Pre-Project Conditions						Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe of Berm	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Circular Failure Surface FOS UTexas4	
Crest	25.0	0.33	1.00	0.30	0.44	8.02	1.33	Seepage Complete 12/18/12 Stability Completed 12/18/12
Elev.	22.4	0.33	0.85	0.20	0.43	7.22	1.56	
200 yr	19.8	0.32	0.70	0.10	0.41	6.42	1.66	
Elev.	17.0	0.29	0.54	<0.1	0.37	1.10	1.83	

URS Results P1GER RD 17 December 2007

Water Level		Pre-Project Conditions						Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe of Berm		Horizontal Gradient	Breakout Above Landside Toe (ft)	Circular Failure Surface FOS UTexas4	
200 yr +3	22.80		0.90				1.90	URS data differs in material properties and absence of waterside Bathymetry and landside LIDAR data.
200 yr	19.80		0.80				2.10	
100 yr	18.90		0.80				2.00	

SEEPAGE/STABILITY ANALYSES LOWER SAN JOAQUIN RIVER REACH LR-2

STA. 1417+00

Water Level		USACE Pre-Project Conditions						Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe of Berm	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Circular Failure Surface FOS UTexas4	
Crest	27.8	0.83	0.90	0.11	0.24	5.06	1.94	Seepage Complete 11/26/12 Stability Completed 12/06/12
Elev.	24.6	0.70	0.76	<0.1	0.19	4.44	2.20	
200 year	21.5	0.54	0.60	<0.1	0.14	2.12	2.48	
Elev.	17.0	0.28	0.33	<0.1	0.09	1.00	2.88	

URS Results P1GER RD 17 December 2007

Water Level		Pre-Project Conditions						Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe of Berm		Horizontal Gradient	Breakout Above Landside Toe (ft)	Circular Failure Surface FOS UTexas4	
200 yr +3	24.50		0.80				2.60	2007 URS report used method not used by Corps, Corps uses different range of WSE to create curve.
200 yr	21.50		0.60				2.90	
100 yr	20.30		0.50				2.90	

SEEPAGE/STABILITY ANALYSES LOWER SAN JOAQUIN RIVER REACH LR-3

STA. 1685+00

Water Level		USACE Pre-Project Conditions						Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe of Berm	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Circular Failure Surface FOS UTexas4	
Crest	31.0	3.18	3.36	1.34	1.37	7.83	0.77	Seepage Complete 12/20/12 Stability Completed 12/21/12
Elev.	28.9	2.79	2.94	1.11	1.19	6.97	1.03	
200 year	26.9	2.39	2.52	0.89	1.00	0.00	1.20	
Elev.	24.0	1.83	1.94	0.57	0.73	0.00	1.35	

URS Results P1GER RD 17 December 2007

Water Level		Pre-Project Conditions						Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe of Berm	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Circular Failure Surface FOS UTexas4	
200 yr +3	29.90			1.10			0.70	2007 URS report used method not used by Corps, Corps uses different range of WSE to create curve.
200 yr	26.90			0.90			1.20	
100 yr	23.80			0.60			1.40	

SEEPAGE/STABILITY ANALYSES LOWER SAN JOAQUIN RIVER REACH LR-4

STA. 1815+00

Water Level		Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Global Failure Surface FOS UTexas4	
Crest	33.9	0.47	0.22	0.59	5.87	1.63	Seepage Complete 12/09/12 Stability Completed 12/10/12
200 year	31.3	0.40	0.18	0.53	3.20	1.78	
Elev.	27.5	0.28	0.12	0.41	1.69	1.98	
Elev.	23.7	0.16	0.06	0.19	0.80	2.14	

URS Results P1GER Task Order 21 December 2007

Water Level		Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Failure Surface FOS	
-	-						The 2007 URS report did not perform analysis on this Station.
-	-						
-	-						

SEEPAGE/STABILITY ANALYSES FRENCH CAMP SLOUGH REACH FL-1

STA. 1049+00

Water Level		USACE Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Global Failure Surface FOS UTexas4	
Crest	21.4	0.44	0.33	0.38	1.40	2.28	Seepage Completed 11/19/12 Stability Completed 11/19/12
Elev.	18.6	0.33	0.26	0.32	0.64	2.41	
200 year	15.9	0.23	0.18	0.22	0.45	2.50	
Elev.	13.0	0.14	0.11	0.14	0.22	2.58	

URS Results P1GER Task Order 21 December 2007

Water Level		Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Failure Surface FOS	
200 yr +3	18.90		0.10			1.50	The 2007 URS report used method not used by Corps, Corps uses different range of WSE to create curve.
200 yr	15.90		0.10			2.00	
100 yr	15.30		0.10			2.00	

SEEPAGE/STABILITY ANALYSES FRENCH CAMP SLOUGH REACH FR-1

STA. 1164+20

Water Level		Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Global Failure Surface FOS UTexas4	
Crest	21.8	0.94	1.11	0.52	8.63	1.52	Seepage Complete 12/12/12 Stability Completed 12/12/12
Elev.	18.8	0.82	0.96	0.44	7.80	1.65	
200 year	15.9	0.69	0.81	0.35	1.89	1.76	
Elev.	12.9	0.56	0.65	0.24	0.94	1.88	

URS Results GER Volume 1, Appendix B (No date)

Water Level		Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Failure Surface FOS	Report was not available. Data was obtained from an electronic ULE file: \\crystal\Dirt\Levee Historical Information\RD 404\ULE
HTOL	-		1.07			1.71	
200 yr	15.90		1.00			1.80	
1955/1957	-		0.58			2.07	

SEEPAGE/STABILITY ANALYSES STOCKTON DIVERTING CANAL REACH SL-1

STA. 846+68

Water Level		Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Global Failure Surface FOS	
Crest	39.2	1.24	1.10	0.48	5.72	1.40	Seepage Complete 8/27/12 Stability Completed 9/19/12
Elev.	36.1	0.99	0.87	0.45	2.87	1.74	
Elev.	33.1	0.74	0.64	0.32	1.92	1.87	
200 year	30.2	0.48	0.41	0.29	1.44	2.01	

URS Results P1GER SJAFCA Calaveras July 2011

Water Level		Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Failure Surface FOS	
200 yr +3	33.22		0.68		2.78	1.76	Data was obtained from P1GER, P1GDR, AND SGDR
200 yr	30.22		0.43		1.90	1.95	
100 yr	29.91		0.40		1.60	1.97	

SEEPAGE/STABILITY ANALYSES STOCKTON DIVERTING CANAL REACH SL-2

STA. 976+00

Water Level		Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Global Failure Surface FOS	
Crest	44.6	1.04	0.99	0.47	4.57	1.68	Seepage Complete 8/27/12 Stability Completed 9/19/12
200 year	40.4	0.65	0.62	0.47	2.64	2.02	
Elev.	38.8	0.50	0.48	0.38	0.97	2.13	
Elev.	37.2	0.33	0.33	0.32	0.14	2.25	

URS Results P1GER SJAFCA Calaveras July 2011

Water Level		Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Failure Surface FOS	
200 yr +3	43.44		0.83		3.90	1.66	Data was obtained from P1GER, P1GDR, AND SGDR
200 yr	40.44		0.58		3.00	1.94	
100 yr	40.10		0.56		2.60	1.97	

<table>
<tr><th colspan="8">SEEPAGE/STABILITY ANALYSES CALAVERAS RIVER REACH CL-1/CL-2</th></tr>
<tr><th colspan="8">STA. 6757+00</th></tr>
<tr><th colspan="2" rowspan="2">Water Level</th><th colspan="5">USACE Pre-Project Conditions</th><th>Notes</th></tr>
<tr><th>Point Gradient at Toe</th><th>Average Vertical Exit Gradient at Toe</th><th>Horizontal Gradient</th><th>Breakout Above Landside Toe (ft)</th><th>Global Failure Surface FOS</th><td rowspan="5">Seepage Completed 8/28/12 Stability Completed 9/6/12</td></tr>
<tr><td>Crest</td><td>31.4</td><td>0.34</td><td>0.14</td><td>0.38</td><td>4.66</td><td>2.05</td></tr>
<tr><td>Elev.</td><td>29.4</td><td>0.29</td><td>0.12</td><td>0.22</td><td>2.42</td><td>2.28</td></tr>
<tr><td>Elev.</td><td>27.4</td><td>0.25</td><td>0.09</td><td>0.21</td><td>1.68</td><td>2.46</td></tr>
<tr><td>200 year</td><td>25.5</td><td>0.13</td><td>0.05</td><td>0.13</td><td>0.00</td><td>2.71</td></tr>
<tr><th colspan="8">URS Results P1GER SJAFCA Calaveras July 2011</th></tr>
<tr><th colspan="2" rowspan="2">Water Level</th><th colspan="5">Pre-Project Conditions</th><th>Notes</th></tr>
<tr><th>Point Gradient at Toe</th><th>Average Vertical Exit Gradient at Toe</th><th>Horizontal Gradient</th><th>Breakout Above Landside Toe (ft)</th><th>Failure Surface FOS</th><td rowspan="4">Analyses Completed By URS</td></tr>
<tr><td>200 yr +3</td><td>28.51</td><td></td><td>0.12</td><td></td><td>2.60</td><td>2.35</td></tr>
<tr><td>200 yr</td><td>25.51</td><td></td><td><0.1</td><td></td><td>0.30</td><td>2.69</td></tr>
<tr><td>100 yr</td><td>25.07</td><td></td><td><0.1</td><td></td><td>0.30</td><td>2.72</td></tr>
</table>

<table>
<tr><th colspan="8">SEEPAGE/STABILITY ANALYSES CALAVERAS RIVER REACH CR-1/CR-2</th></tr>
<tr><th colspan="8">STA. 3306+00</th></tr>
<tr><th colspan="2" rowspan="2">Water Level</th><th colspan="5">USACE Pre-Project Conditions</th><th>Notes</th></tr>
<tr><th>Point Gradient at Toe</th><th>Average Vertical Exit Gradient at Toe</th><th>Horizontal Gradient</th><th>Breakout Above Landside Toe (ft)</th><th>Global Failure Surface FOS</th><td rowspan="5">Seepage Completed on 8/30/12 Stability Completed 9/6/12</td></tr>
<tr><td>Crest</td><td>29.7</td><td>0.97</td><td>1.57</td><td>0.22</td><td>1.85</td><td>2.91</td></tr>
<tr><td>Elev.</td><td>28.2</td><td>0.62</td><td>1.10</td><td>0.18</td><td>0.92</td><td>3.13</td></tr>
<tr><td>200 yr</td><td>26.9</td><td>0.19</td><td>0.47</td><td>0.14</td><td>0.21</td><td>3.37</td></tr>
<tr><td>Elev.</td><td>25.3</td><td>0.00</td><td><0.1</td><td>0.00</td><td>0.00</td><td>3.73</td></tr>
<tr><th colspan="8">URS Results P1GER SJAFCA Calaveras July 2011</th></tr>
<tr><th colspan="2" rowspan="2">Water Level</th><th colspan="5">Pre-Project Conditions</th><th>Notes</th></tr>
<tr><th>Point Gradient at Toe</th><th>Average Vertical Exit Gradient at Toe</th><th>Horizontal Gradient</th><th>Breakout Above Landside Toe (ft)</th><th>Failure Surface FOS</th><td rowspan="5">URS Results from P1GER July 2011. Exit gradient results appear lower due to the fact URS used the same permeability for materials 1 & 2 and chose to take the gradient inbetween the two layers.</td></tr>
<tr><td>200 yr +3</td><td>29.88</td><td></td><td>0.67</td><td></td><td>2.20</td><td>2.87</td></tr>
<tr><td>200 yr</td><td>26.88</td><td></td><td>0.20</td><td></td><td>0.40</td><td>3.29</td></tr>
<tr><td>100 yr</td><td>26.45</td><td></td><td>0.20</td><td></td><td>0.20</td><td>3.41</td></tr>
<tr><td colspan="7"></td></tr>
</table>

SEEPAGE/STABILITY ANALYSES CALAVERAS RIVER REACH D-4

STA. 3092+00

Water Level		USACE Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Global Failure Surface FOS	
Crest	18.8	1.33	1.21	0.48	9.92	0.95	Seepage Completed on 8/30/12 Stability Completed 9/6/12
Elev.	16.5	1.10	1.00	0.43	4.14	1.18	
200 year	14.2	0.87	0.79	0.41	2.83	1.57	
Elev.	11.8	0.63	0.57	0.35	1.65	1.89	

URS Results P1GER SJAFCA Calaveras July 2011

Water Level		Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Failure Surface FOS	
200 yr +3	17.16		0.79		8.80	1.10	URS Results from P1GER July 2011
200 yr	14.16		0.55		1.90	1.56	
100 yr	13.77		0.52		1.90	1.60	

SEEPAGE/STABILITY ANALYSES CALAVERAS RIVER REACH D-5

STA. 6535+00

Water Level		USACE Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Global Failure Surface FOS UTexas4	
Crest	17.5	0.53	0.23	0.33	6.76	1.86	Seepage Complete 8/27/12 Stability Completed 9/5/12
200 year	13.2	0.41	0.15	0.29	4.05	2.15	
Elev.	10.0	0.29	0.09	0.28	1.19	2.38	
Elev.	7.2	0.09	0.04	0.09	0.00	2.60	

URS Results P1GER SJAFCA Calaveras July 2011

Water Level		Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Failure Surface FOS	
200 yr +3	16.16		0.18		3.80	1.18	URS and Corps results for FOS are different. After some study of materials properties and cross-section obtained from URS, the FOS generated by UTexas4 appear correct.
200 yr	13.16		0.13		1.60	1.38	
100 yr	12.81		0.12		1.40	1.40	

SEEPAGE/STABILITY ANALYSES DELTA FRONT BROOKSIDE REACH D-BS

STA. 166+50

Water Level		USACE Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Circular Failure Surface FOS UTexas4	**USACE** Seepage Complete 3/11/13 Stability Completed 3/14/13
Crest	18.0	0.99	0.80	0.41	5.21	0.94	
Elev.	14.0	0.81	0.64	0.30	3.50	1.22	
Elev.	10.0	0.62	0.49	0.26	2.30	1.27	
Elev.	6.0	0.44	0.35	0.21	1.10	1.30	

NO RESULTS FROM A-E REPORTS

Water Level		Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Circular Failure Surface FOS UTexas4	No analyses. Historic Kleinfelder data used to generate cross-section in USACE analysis. Material properites listed were modified based on discussions with Levee Safety.
200 yr +3							
200 yr							
100 yr							

SEEPAGE/STABILITY ANALYSES DELTA FRONT LINCOLN VILLALGE REACH D-LV

STA. 162+50

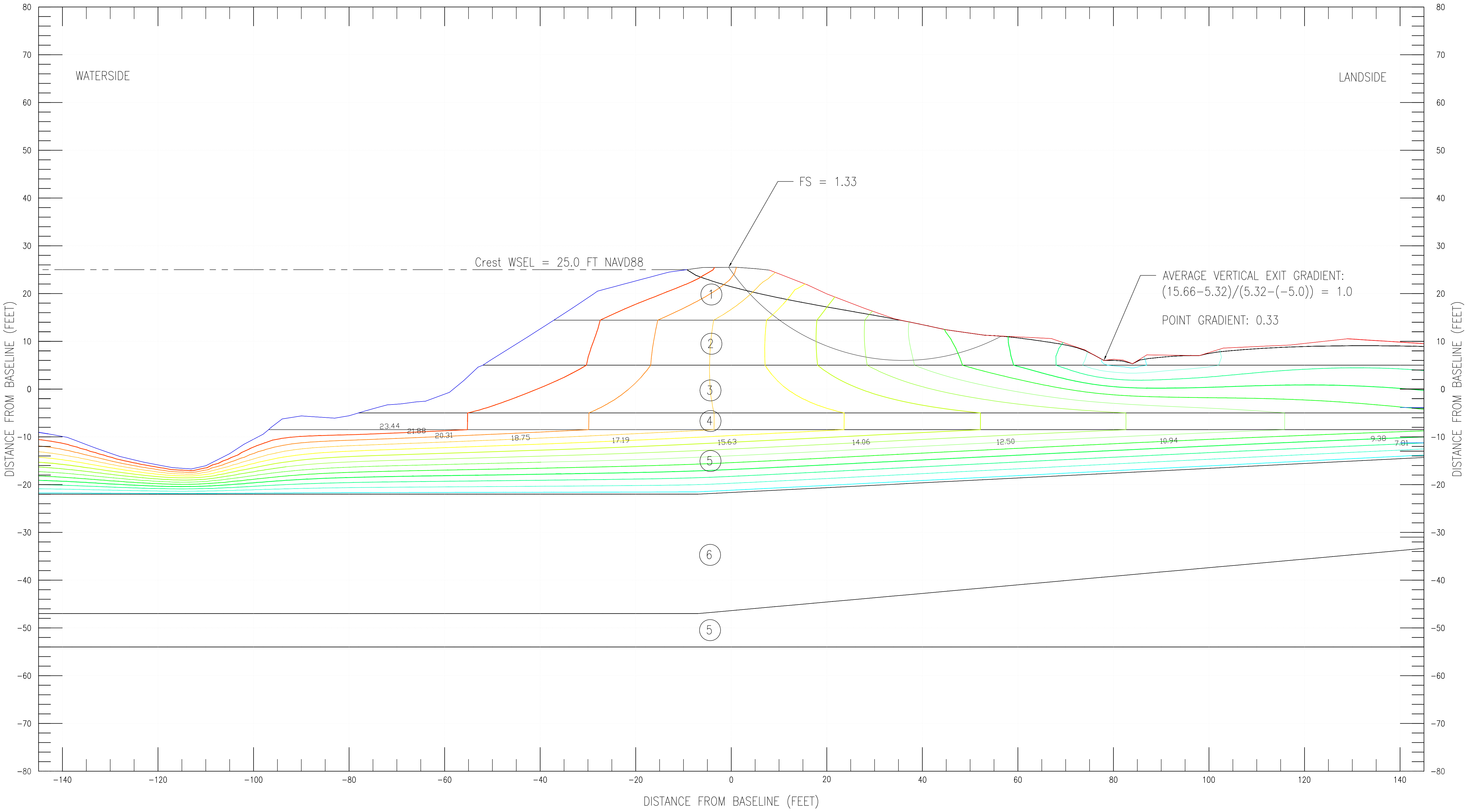
Water Level		USACE Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Circular Failure Surface FOS UTexas4	**USACE** Seepage Complete 4/02/13 Stability Completed 4/03/13
Crest	13.2	0.51	0.63	0.16	1.00	1.83	
Elev.	11.0	0.35	0.44	0.14	0.67	1.94	
Elev.	8.5	0.18	0.24	0.10	0.00	2.04	
Elev.	6.0	0.01	0.01	0.00	0.00	2.13	

NO RESULTS FROM A-E REPORTS

Water Level		Pre-Project Conditions					Notes
		Point Gradient at Toe	Average Vertical Exit Gradient at Toe	Horizontal Gradient	Breakout Above Landside Toe (ft)	Circular Failure Surface FOS UTexas4	No analyses. Historic Kleinfelder data used to generate cross-section in USACE analysis. Material properites listed were modified based on discussions with Levee Safety.
200 yr +3							
200 yr							
100 yr							

LOWER SAN JOAQUIN RIVER
FEASIBILITY STUDY

CROSS-SECTIONS WITH STRATIGRAPHY
HEAD CONTOURS AND
FAILURE SURFACE



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR0017_036B	1	Clay Levee	1.00E-06	4	2.50E-07	0.00284	0.00071	28	50	120
	2	Silt Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	28	0	120
	3	Clay Blanket	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120
	4	Silty Sand	1.00E-03	4	2.50E-04	2.83500	0.70875	32	0	125
	5	Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120
	6	Sand	5.00E-03	4	1.25E-03	14.17500	3.54375	32	0	125

0 10' 20'
HORIZONTAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

0 10' 20'
VERTICAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

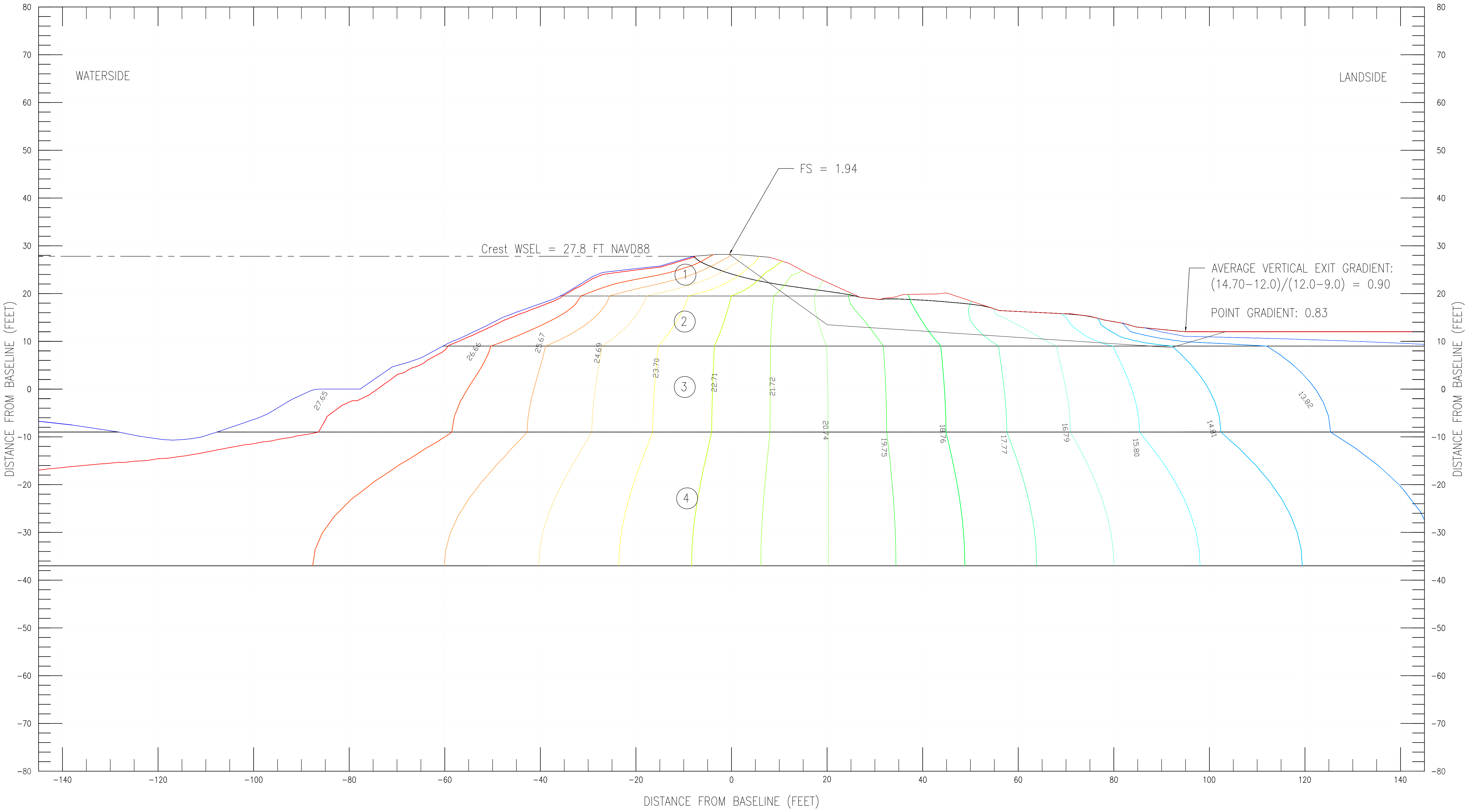
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

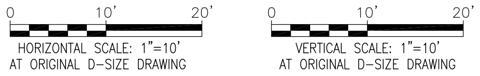
Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results
CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
SAN JOAQUIN RIVER RD–17 REACH LR–1 STA. 1292+00

DATE: 12 - August - 13
SCALE: As Shown
SHEET NO. 1 of 14

PLOT BY: L2EDGGAJ - Aug 21, 2013 - 2:29:32pm
DRAWING: I:\draft-final geotechnical appendix\2 - enclosures\enclosure seepage stability results\lsj seepage stability plates\GMS\RD 17\ReacReach LR1 Sta 1292+00.dwg



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR0017_0 52B	1	Levee Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	28	100	120
	2	Silty Sand Blanket	1.00E-05	4	2.50E-06	0.02835	0.00709	30	0	125
	3	Poorly Graded Sand wSilt	1.00E-04	4	2.50E-05	0.28350	0.07088	32	0	130
	4	Foundation Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	28	100	120



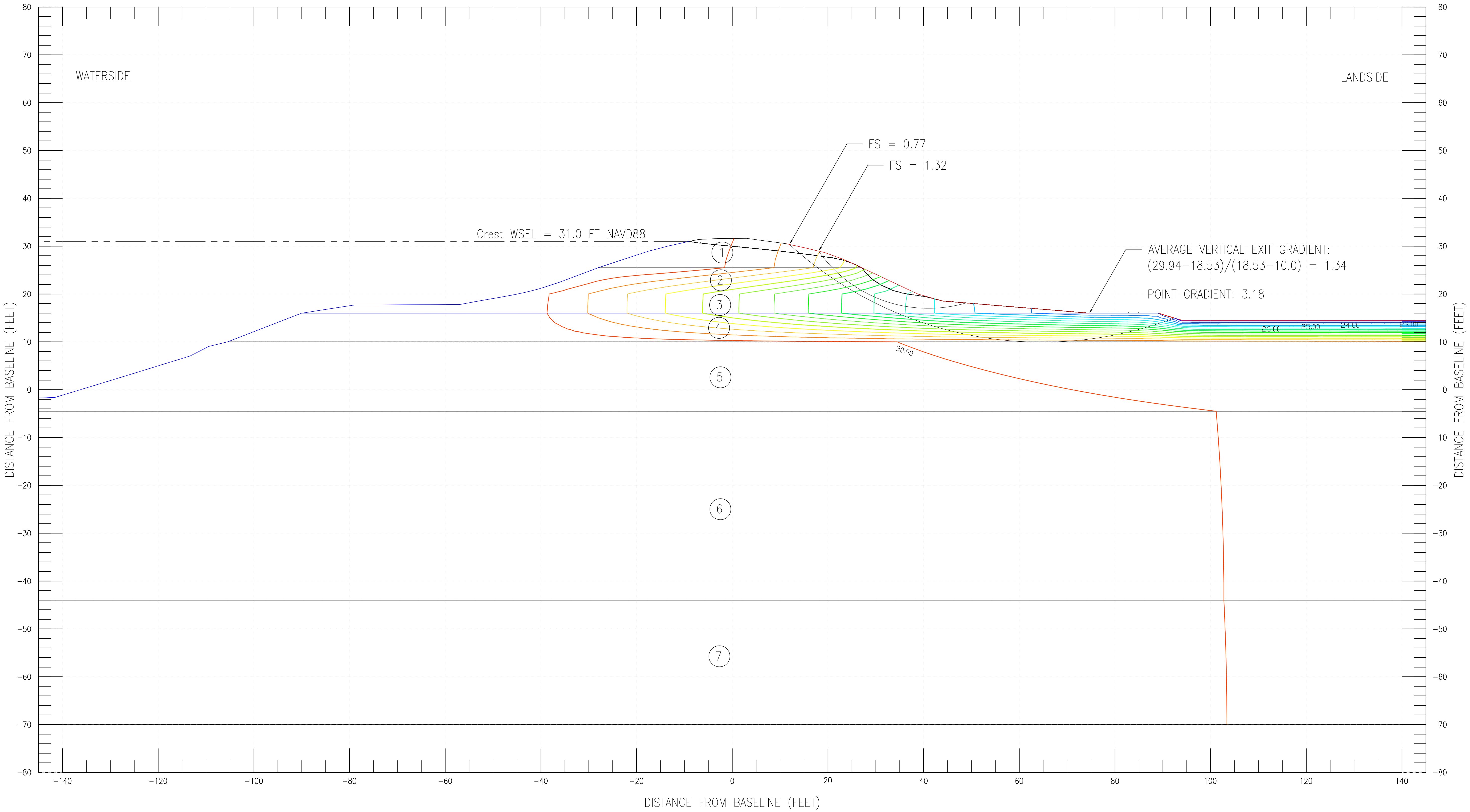
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results
CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
SAN JOAQUIN RIVER RD−17 REACH LR−2 STA. 1417+00

DATE: 12 - August - 13
SCALE: As Shown
SHEET NO. 2 of 14

PLOT BY: L2EDGGAJ - Aug 21, 2013 - 2:49:00pm
DRAWING: I:\draft-final geotechnical appendix\2 - enclosures\enclosure seepage stability results\lsj seepage stability plates\GMS\RD 17\ReacReach LR2 Sta. 1417+00.dwg



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR0017_085B	1	Poorly Graded Sand wSilt	5.00E-03	4	1.25E-03	14.17500	3.54375	32	0	125
	2	Clayey Sand	5.00E-05	4	1.25E-05	0.14175	0.03544	30	50	125
	3	Poorly Graded Sand	5.00E-03	4	1.25E-03	14.17500	3.54375	32	0	125
	4	Sandy Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	28	140	120
	5	Silty Sand	1.00E-04	4	2.50E-05	0.28350	0.07088	30	0	120
	6	Poorly Graded Sand wSilt	1.00E-02	4	2.50E-03	28.35000	7.08750	32	0	125
	7	Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120

0 10' 20'
HORIZONTAL SCALE: 1"=10'
AT ORIGINAL D–SIZE DRAWING

0 10' 20'
VERTICAL SCALE: 1"=10'
AT ORIGINAL D–SIZE DRAWING

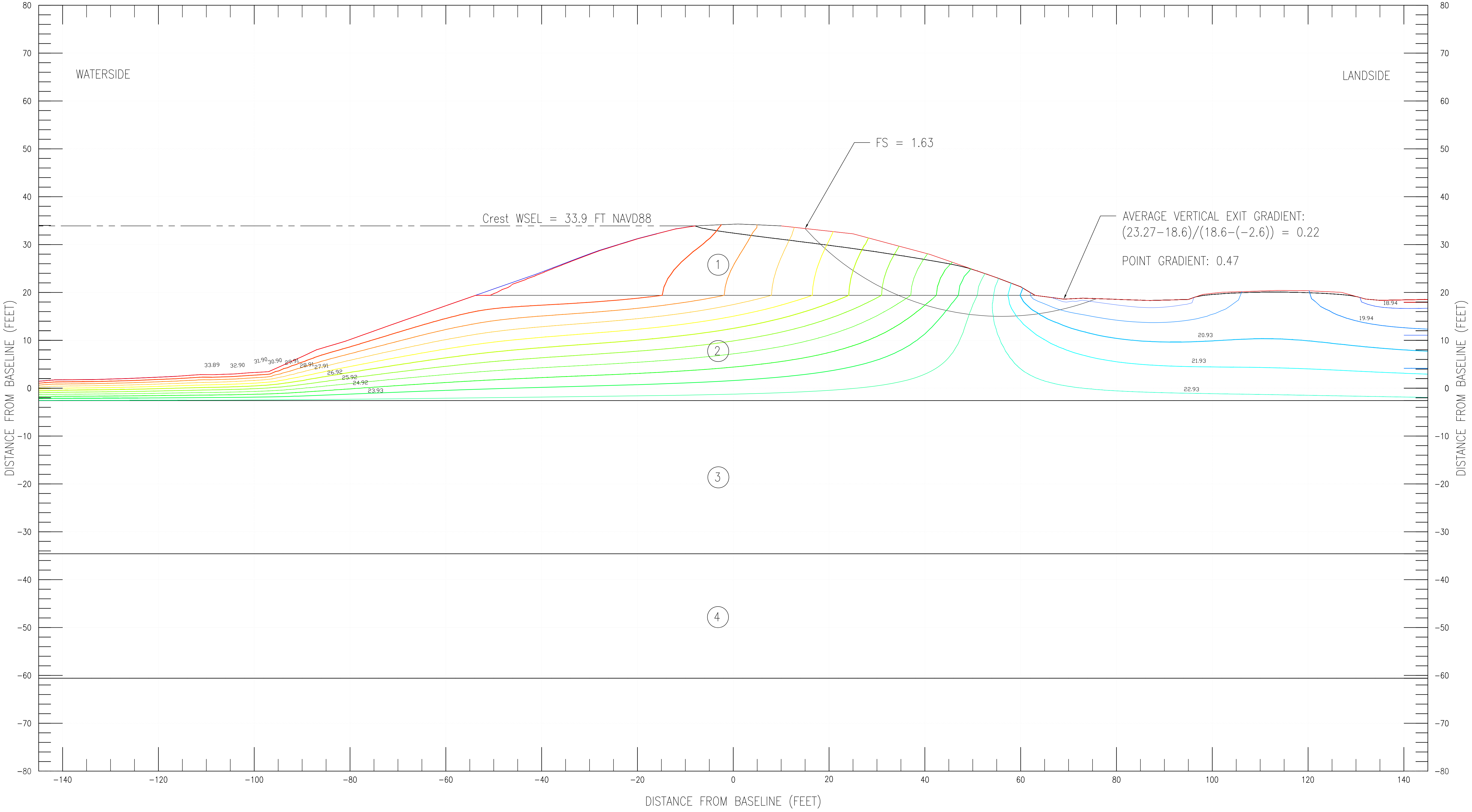
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results
CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
SAN JOAQUIN RIVER RD–17 REACH LR–3 STA. 1685+00

DATE:	SCALE:	SHEET NO.
12 - August - 13	As Shown	3 of 14

PLOT BY: L2EDGGAJ - Aug 28, 2013 - 2:16:30pm
DRAWING: I:\CAD\CAD for Enclosures\GMS\RD 17\Reach LR3 Sta. 1685+00\Reach LR3 Sta. 1685+00.dwg



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR0017_ 100C	1	Clayey Sand	1.00E-04	4	2.50E-05	0.28350	0.07088	30	50	125
	2	Lean Clay Blanket	1.00E-06	4	2.50E-07	0.00284	0.00071	28	100	120
	3	Poorly Graded Sand w/Silt	1.00E-03	4	2.50E-04	2.83500	0.70875	32	0	130
	4	Lean Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120

0 10' 20'
HORIZONTAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

0 10' 20'
VERTICAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

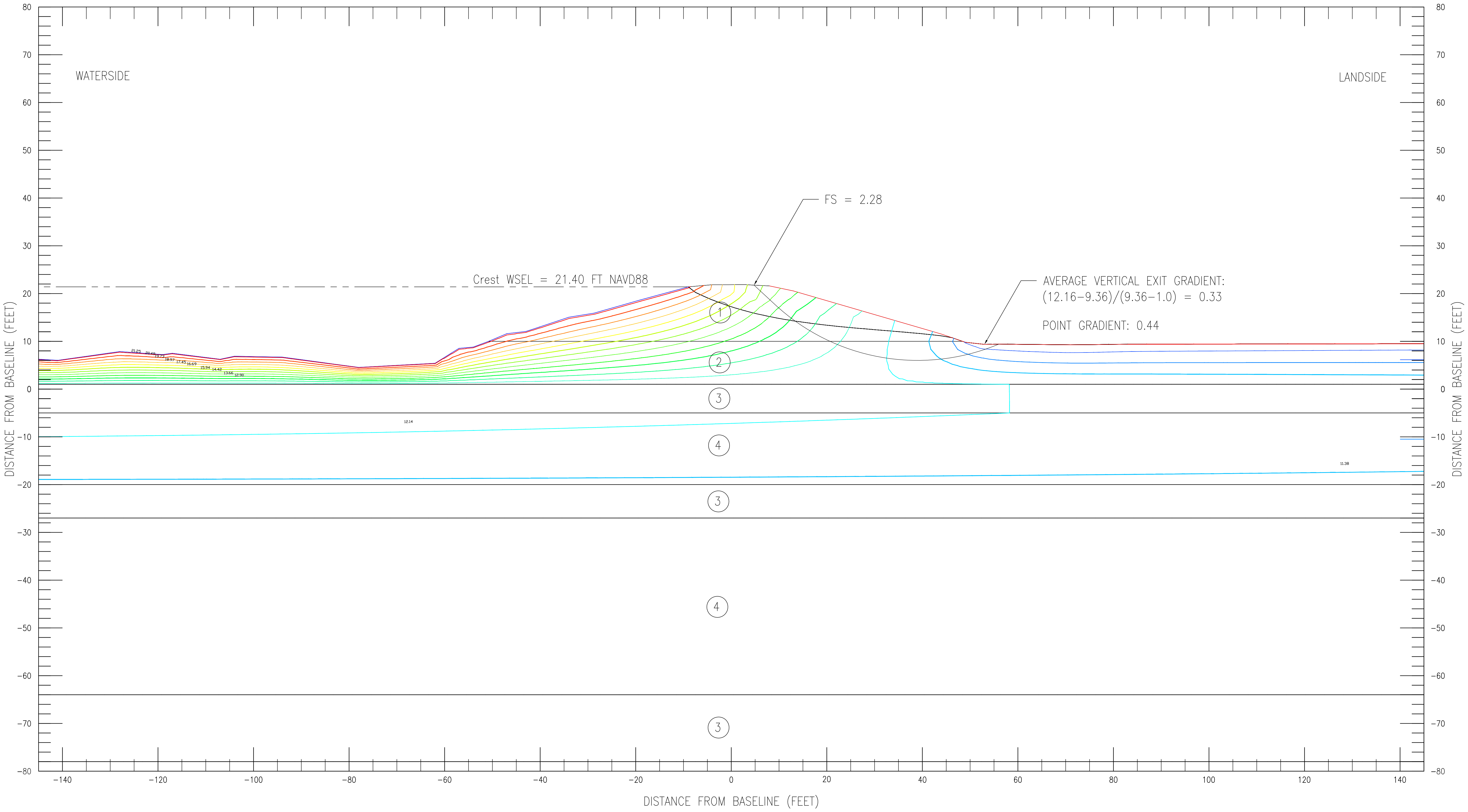
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO
CALIFORNIA

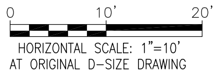
Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results
CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
SAN JOAQUIN RIVER RD–17 REACH LR–4 STA. 1815+00

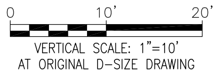
DATE:	SCALE:	SHEET NO.
12 - August - 13	As Shown	4 of 14

PLOT BY: L2EDGGAJ - Aug 21, 2013 - 3:24:59pm
DRAWING: I:\draft-final geotechnical appendix\2 - enclosures\enclosure seepage stability results\lsj seepage stability plates\GMS\RD 17\ReacReach LR4 Sta. 1815+00.dwg



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR0017_ 007B	1	Levee Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	28	100	120
	2	Lean Clay Blanket	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120
	3	Foundation Silty Sand	5.00E-03	4	1.25E-03	14.17500	3.54375	32	0	125
	4	Foundation Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120





DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO
CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results

CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
FRENCH CAMP REACH FL–1 STA. 1049+00

DATE:	SCALE:	SHEET NO.
12 - August - 13	As Shown	5 of 14

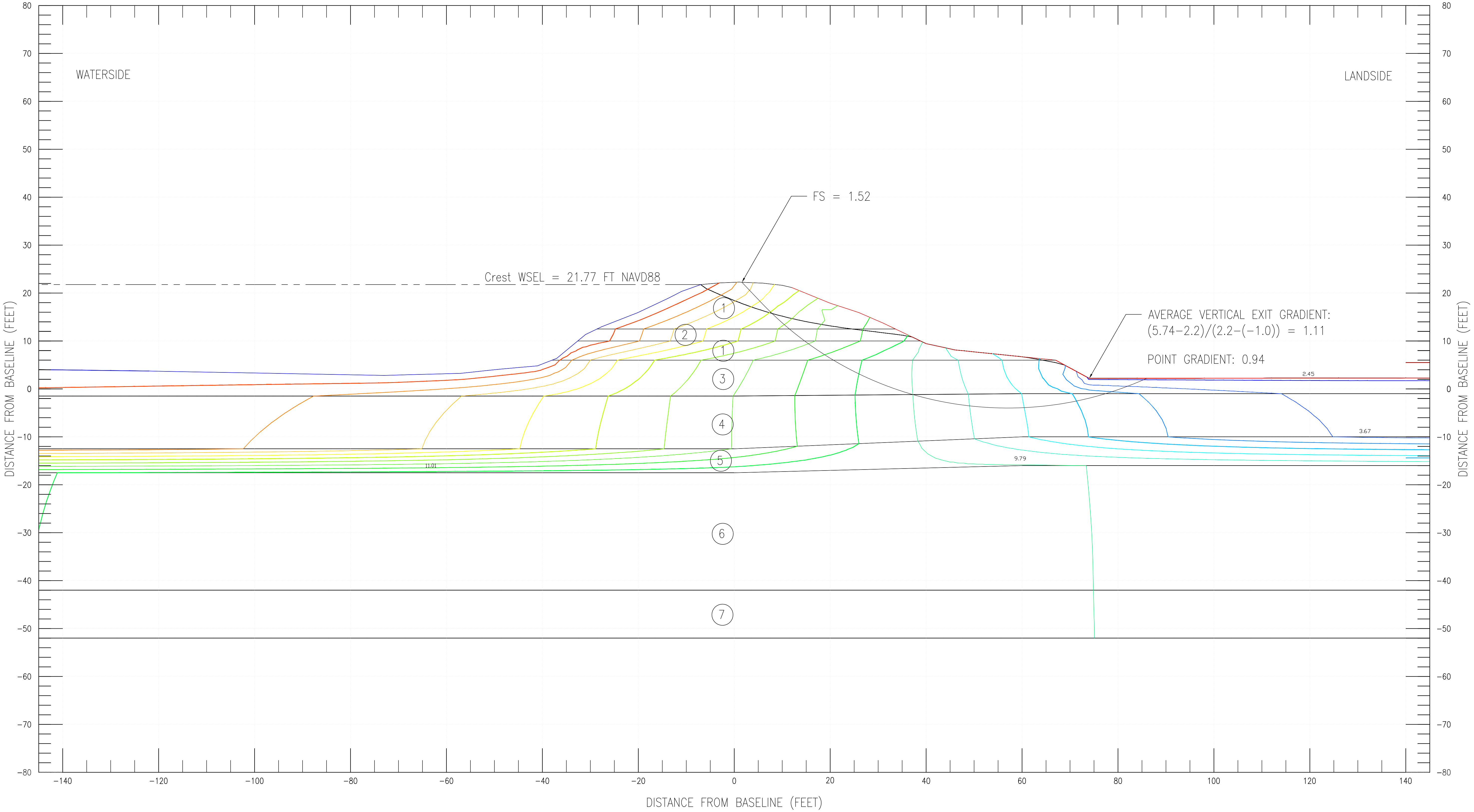
PLOT BY: L2EDGGAJ - Aug 21, 2013 - 4:47:12pm
DRAWING: I:\draft-final geotechnical appendix\2 - enclosures\enclosure seepage stability results\s seepage stability plates\GMS\French CamReach FL1 Sta. 1049+00.dwg

4 3 2 1

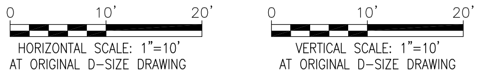
C

B

A



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR0404_042B	1	Clay Levee	1.00E-06	4	2.50E-07	0.00284	0.00071	28	100	120
	2	Silt Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	30	0	120
	3	Clayey Sand Blanket	1.00E-05	4	2.50E-06	0.02835	0.00709	28	50	125
	4	Silty Sand	1.00E-04	4	2.50E-05	0.28350	0.07088	32	0	125
	5	Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120
	6	Silty Sand	1.00E-03	4	2.50E-04	2.83500	0.70875	32	0	125
	7	Silt and Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	30	100	120



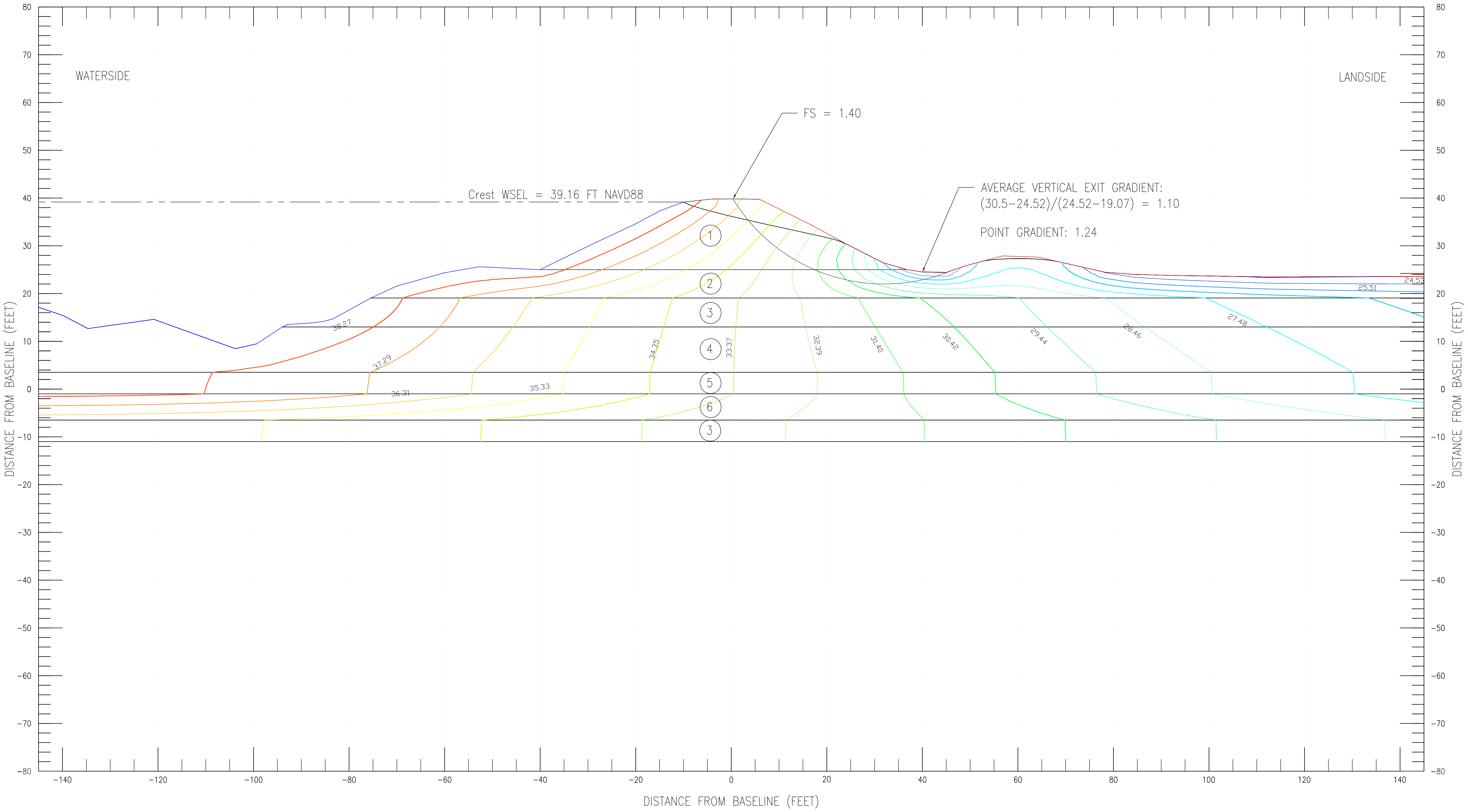
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results
CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
FRENCH CAMP REACH FR−1 STA. 1164+20

DATE:	SCALE:	SHEET NO.
12 - August - 13	As Shown	6 of 14

PLOT BY: L2EDGGAJ - Aug 21, 2013 - 4:45:30pm
DRAWING: I:\draft-final geotechnical appendix\2 - enclosures\enclosure seepage stability results\lsj seepage stability plates\GMS\French CamReach FR1 Sta. 1164+20.dwg



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WCSBDC_004B	1	Sandy Lean Clay Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	34	100	115
	2	Lean Clay Blanket	1.00E-05	4	2.50E-06	0.02835	0.00709	31	150	120
	3	Silty Sand	1.00E-04	4	2.50E-05	0.28350	0.07088	35	0	120
	4	Sandy Silt	1.00E-04	4	2.50E-05	0.28350	0.07088	35	0	120
	5	Silty Sand (more permeable)	1.00E-03	4	2.50E-04	2.83500	0.70875	35	0	120
	6	Lean Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	120

0 10' 20'
HORIZONTAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

0 10' 20'
VERTICAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

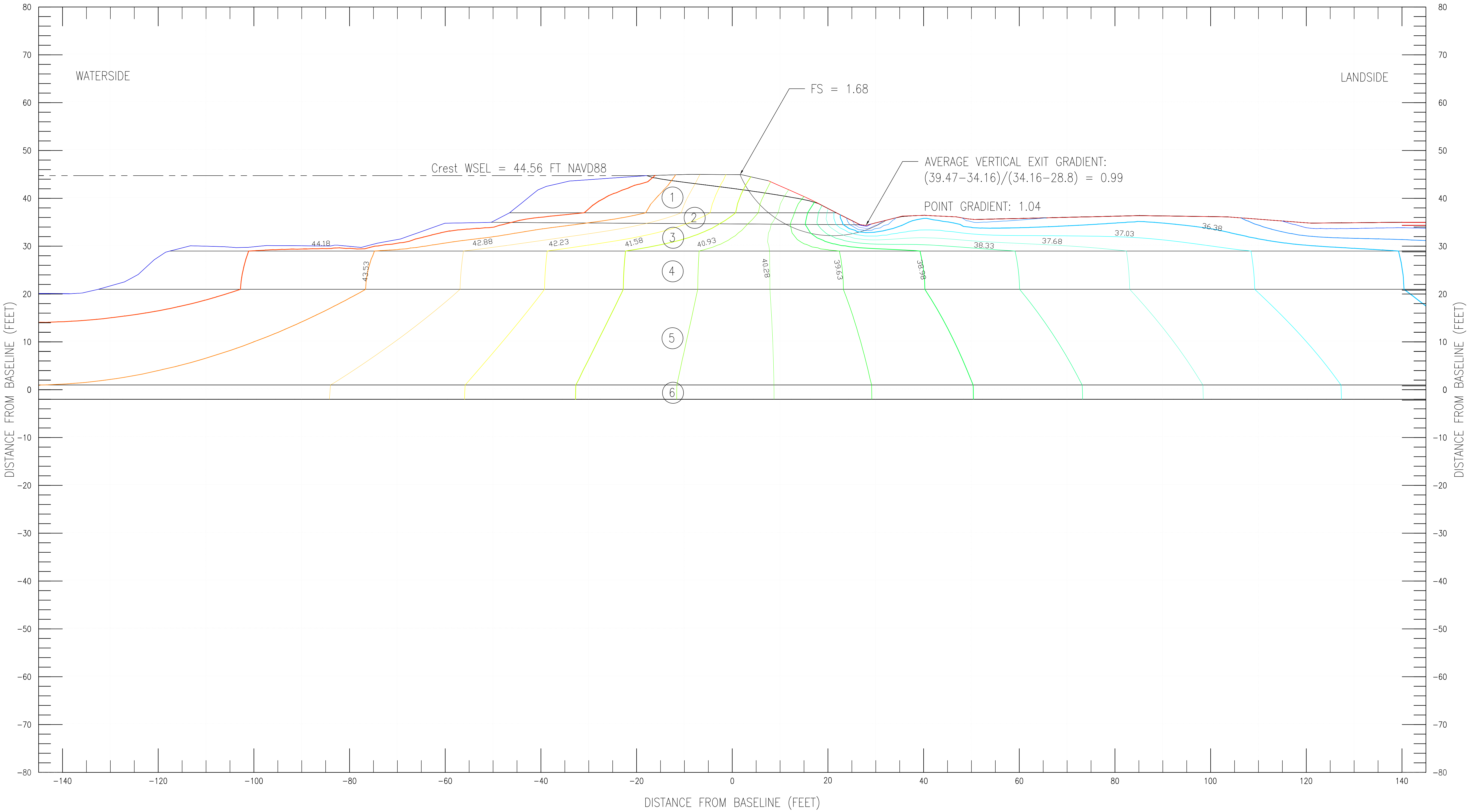
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results
CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
STOCKTON DIVERTING CANAL REACH SL−1 STA. 846+68

DATE:	SCALE:	SHEET NO.
12 - August - 13	As Shown	7 of 14

PLOT BY: L2EDGGAJ - Aug 21, 2013 - 4:02:35pm
DRAWING: I:\draft-final geotechnical appendix\2 - enclosures\enclosure seepage stability results\lsj seepage stability plates\GMS\Stockton DReach SL1 Sta. 846+68.dwg



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WCSBDC_025C	1	Sandy Silt Levee	1.00E-04	4	2.50E-05	0.28350	0.07088	34	0	115
	2	Lean Clay/Silty Lean Clay Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	31	150	115
	3	Lean Clay/Silty Lean Clay Blanket	1.00E-05	4	2.50E-06	0.02835	0.00709	31	150	115
	4	Sand to Silty Sand	5.00E-04	4	1.25E-04	1.41750	0.35438	35	0	125
	5	Lean Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	115
	6	Sandy Silt	1.00E-04	4	2.50E-05	0.28350	0.07088	35	0	120

0 10' 20'
HORIZONTAL SCALE: 1"=10'
AT ORIGINAL D–SIZE DRAWING

0 10' 20'
VERTICAL SCALE: 1"=10'
AT ORIGINAL D–SIZE DRAWING

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

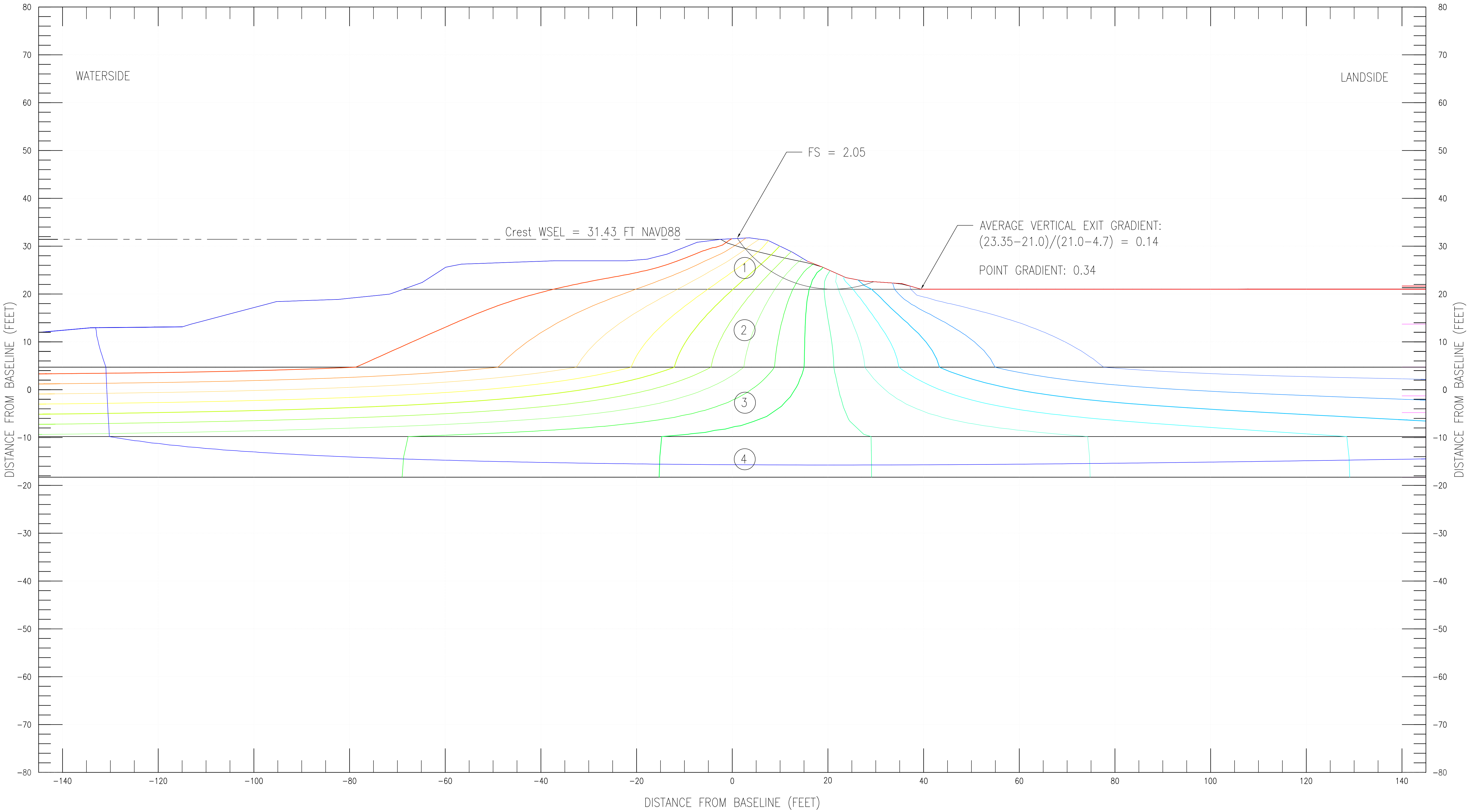
SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results

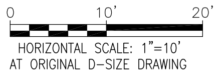
CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
STOCKTON DIVERTING CANAL REACH SL–2 STA. 976+00

DATE:	SCALE:	SHEET NO.
12 - August - 13	As Shown	8 of 14

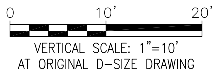
PLOT BY: L2EDGGAJ - Aug 21, 2013 - 4:27:09pm
DRAWING: I:\draft-final geotechnical appendix\2 - enclosures\enclosure seepage stability results\lsj seepage stability plates\GMS\Stockton DReach SL2 Sta. 976+00.dwg



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WCSBCR_ 004B	1	Silt Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	34	100	115
	2	Silt Blanket	1.00E-05	4	2.50E-06	0.02835	0.00709	31	150	115
	3	Lean Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	120
	4	Sandy Silt Foundation	1.00E-04	4	2.50E-05	0.28350	0.07088	35	0	120



HORIZONTAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING



VERTICAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

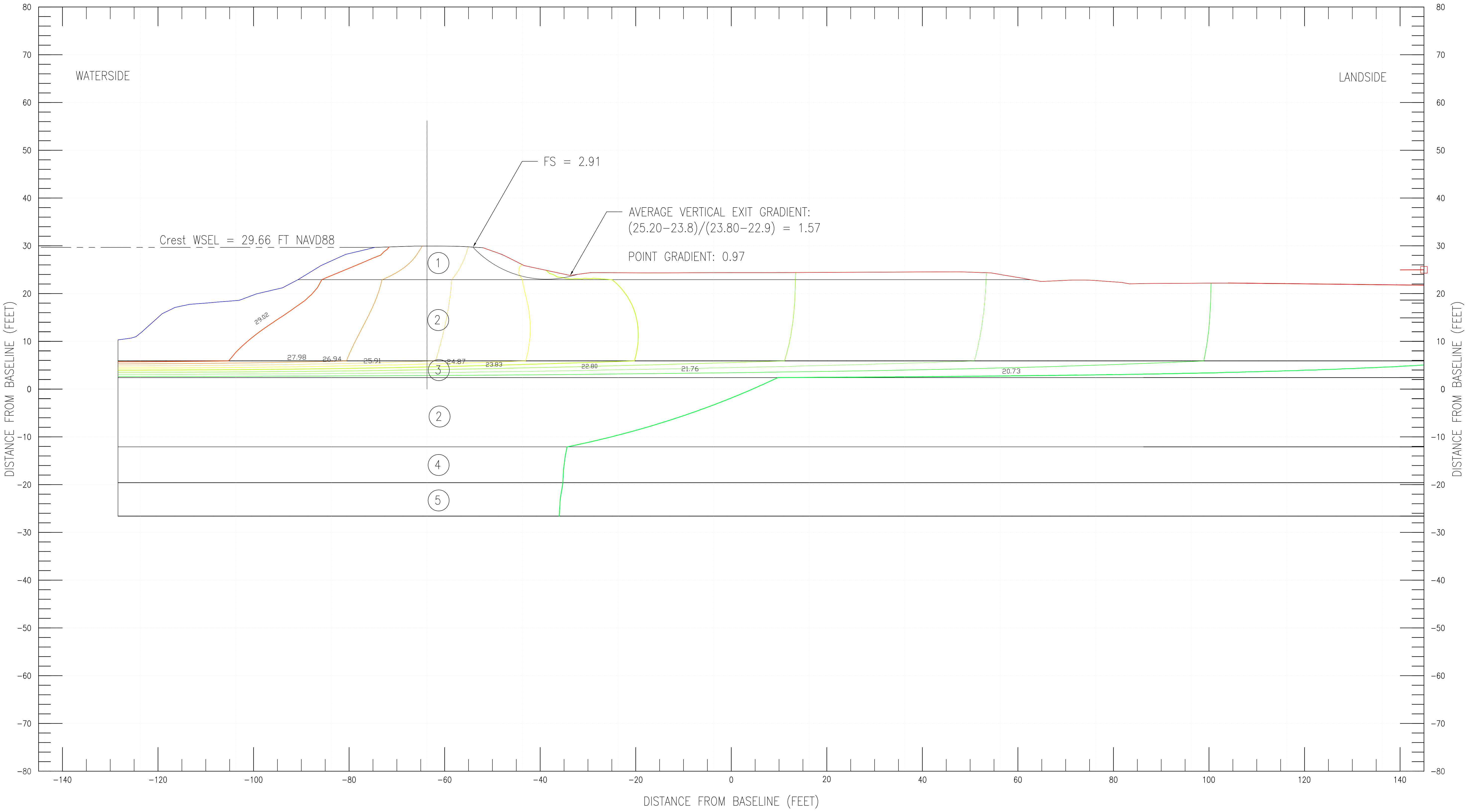
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results
CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
CALAVERAS RIVER REACH CL−1/CL−2 STA. 6757+00

DATE: 12 - August - 13
SCALE: As Shown
SHEET NO. 9 of 14

PLOT BY: L2EDGGAJ - Aug 21, 2013 - 4:39:15pm
DRAWING: I:\draft-final geotechnical appendix\2 - enclosures\enclosure seepage stability results\lsj seepage stability plates\GMS\Calaveras Reach CL1 Sta. 6757+00.dwg



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WCNBCR_010 A	1	Lean Clay wSand Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	34	100	120
	2	Sandy Silt	1.00E-05	4	2.50E-06	0.02835	0.00709	31	150	115
	3	Lean Clay wSand	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	120
	4	Pooly Graded Sand wSilt	2.10E-03	4	5.25E-04	5.95350	1.48838	35	0	120
	5	Silt	1.00E-05	4	2.50E-06	0.02835	0.00709	31	150	115

0 10' 20'
HORIZONTAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

0 10' 20'
VERTICAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

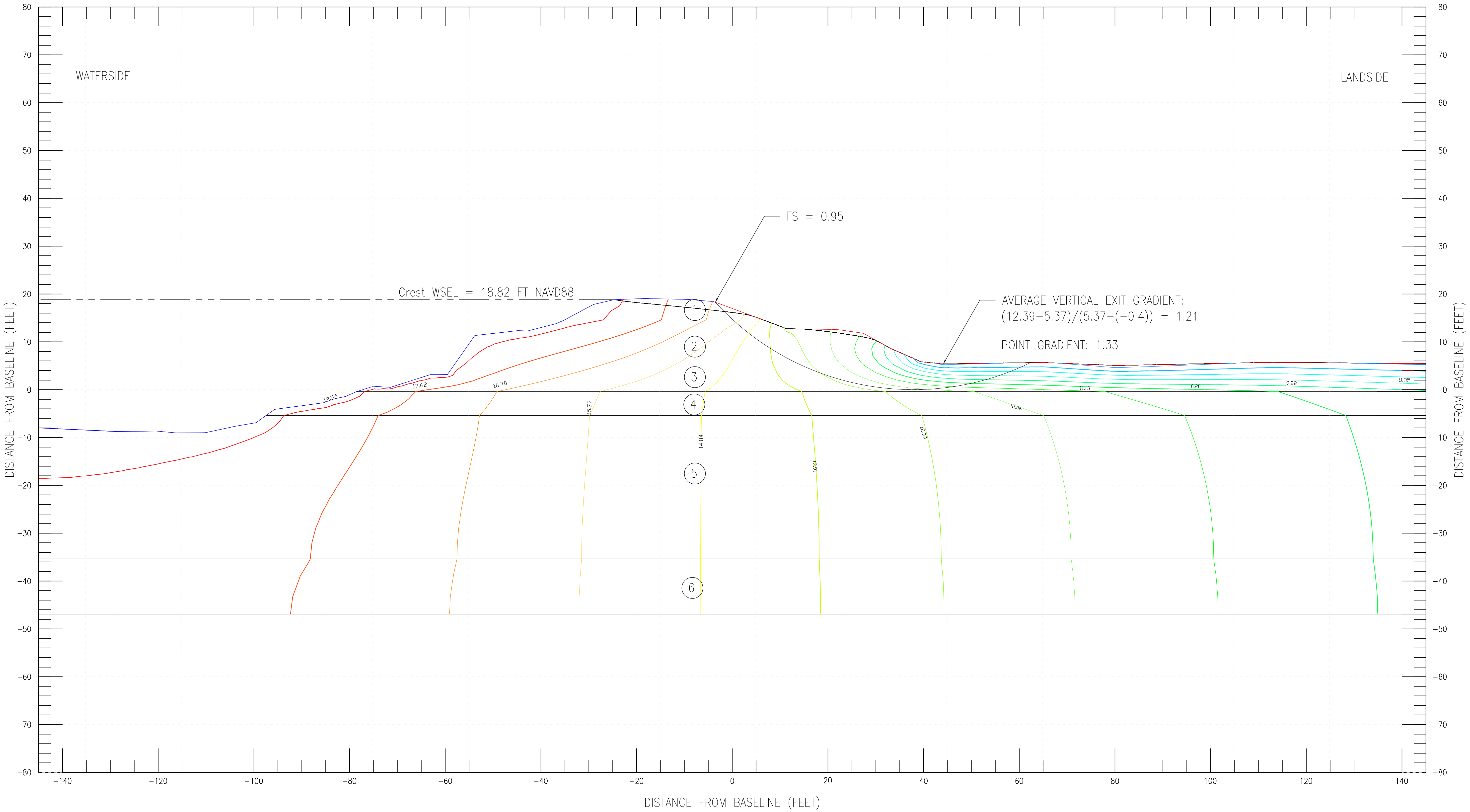
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

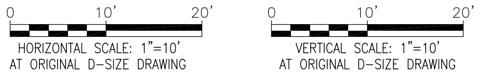
Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results
CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
CALAVERAS RIVER REACH CR−1/CR−2 STA. 3306+00

DATE: 12 - August - 13
SCALE: As Shown
SHEET NO. 10of 14

PLOT BY: L2EDGGAJ - Aug 21, 2013 - 4:40:58pm
DRAWING: I:\draft-final geotechnical appendix\2 - enclosures\enclosure seepage stability results\lsj seepage stability plates\GMS\Calaveras Reach CR1 Sta. 3306+00.dwg



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WCNBCR_003B	1	Sandy Silt Levee	1.00E-04	4	2.50E-05	0.28350	0.07088	31	0	110
	2	Lean Clay wSand to CH Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	34	100	110
	3	FAT Clay wSand Blanket	1.00E-05	4	2.50E-06	0.02835	0.00709	27	50	110
	4	Sandy Silt	1.00E-04	4	2.50E-05	0.28350	0.07088	31	0	115
	5	Pooly Graded Sand wSilt	6.40E-04	4	1.60E-04	1.81440	0.45360	32	0	120
	6	Lean Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	120



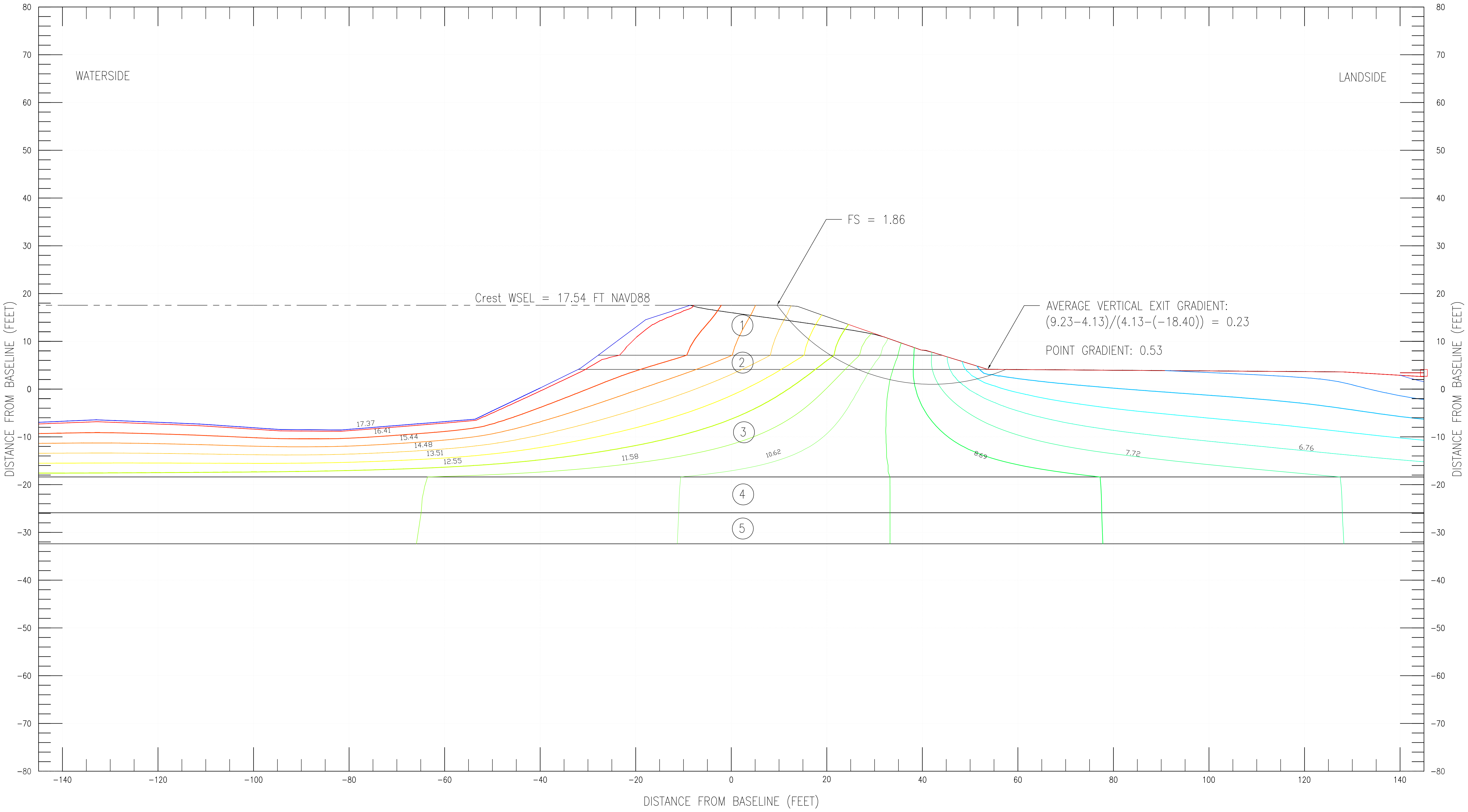
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO
CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results
CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
CALAVERAS RIVER REACH D-4 STA. 3092+00

DATE:	SCALE:	SHEET NO.
12 - August - 13	As Shown	11of 14

PLOT BY: L2EDGGAJ - Aug 21, 2013 - 4:42:26pm
DRAWING: I:\draft-final geotechnical appendix\2 - enclosures\enclosure seepage stability results\lsj seepage stability plates\GMS\Calaveras Reach D4 Sta. 3092+00.dwg



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR1614_004B	1	Silt to Sandy Silt Levee	1.00E-05	4	2.50E-06	0.02835	0.00709	31	0	110
	2	Lean Clay Levee	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	115
	3	Lean Clay Blanket	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	115
	4	Silty Sand	1.00E-04	4	2.50E-05	0.28350	0.07088	35	0	120
	5	Lean Clay	1.00E-06	4	2.50E-07	0.00284	0.00071	31	150	115

0 10' 20'
HORIZONTAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

0 10' 20'
VERTICAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

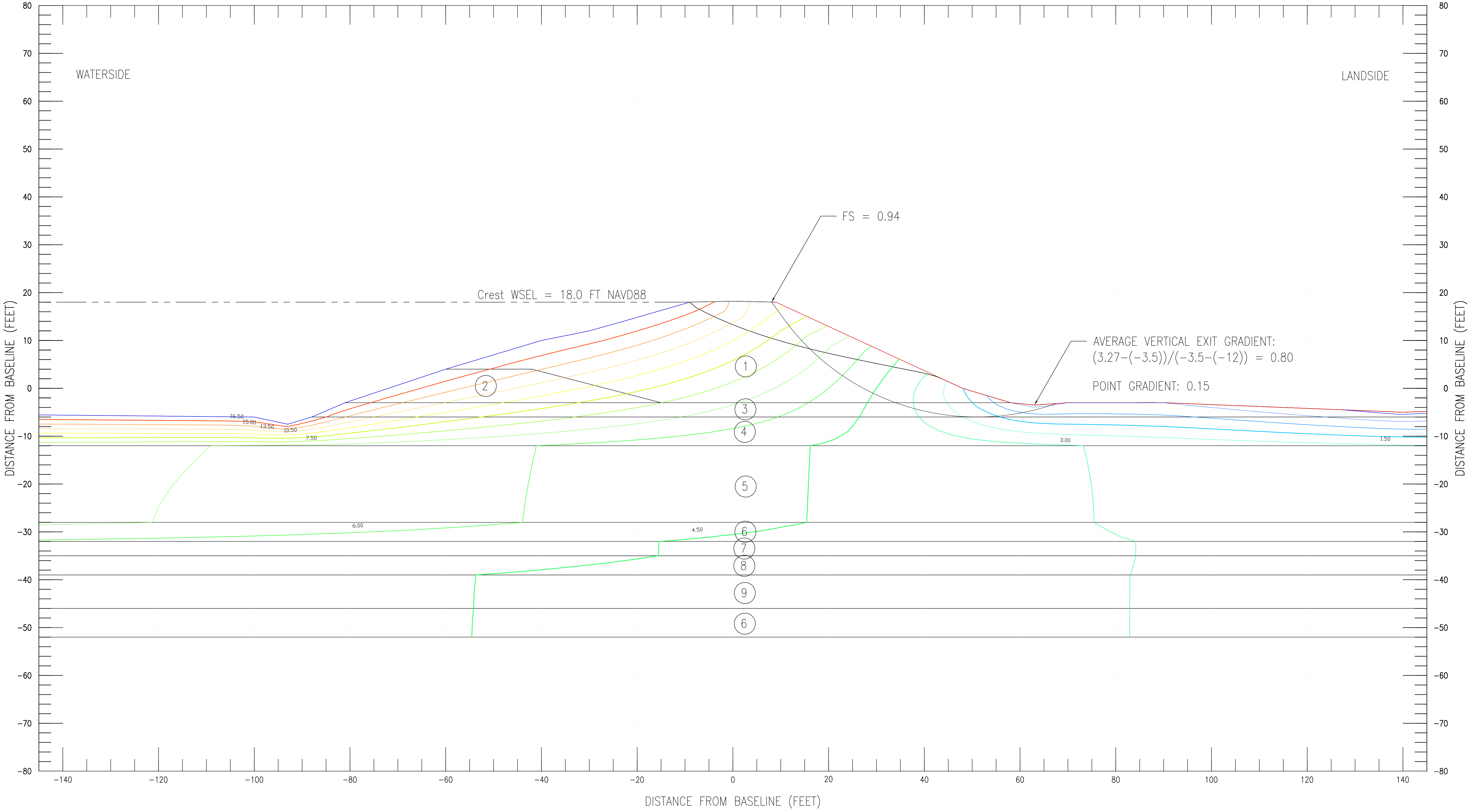
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO
CALIFORNIA

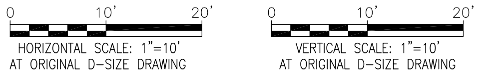
Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results
CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
CALAVERAS RIVER REACH D−5 STA. 6535+00

DATE:	SCALE:	SHEET NO.
12 - August - 13	As Shown	12of 14

PLOT BY: L2EDGGAJ - Aug 21, 2013 - 4:43:48pm
DRAWING: I:\draft-final geotechnical appendix\2 - enclosures\enclosure seepage stability results\lsj seepage stability plates\GMS\Calaveras Reach D5 Sta. 6535+00.dwg



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR2074_015C	1	Clay Levee	4.00E-06	4	1.00E-06	0.01134	0.00284	30	50	120
	2	Farm Levee	4.00E-06	4	1.00E-06	0.01134	0.00284	30	50	110
	3	Organic Soil	4.00E-06	4	1.00E-06	0.01134	0.00284	26	50	80
	4	Blanket	4.00E-06	4	1.00E-06	0.01134	0.00284	30	100	120
	5	Silty Sand	4.00E-04	4	1.00E-04	1.13400	0.28350	32	0	125
	6	Clay	4.00E-06	4	1.00E-06	0.01134	0.00284	30	100	120
	7	Poorly graded Sand w/silt	1.00E-03	4	2.50E-04	2.83500	0.70875	34	0	125
	8	Silt	4.00E-06	4	1.00E-06	0.01134	0.00284	32	0	120
	9	Silty Sand	4.00E-04	4	1.00E-04	1.13400	0.28350	32	0	125



HORIZONTAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

VERTICAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

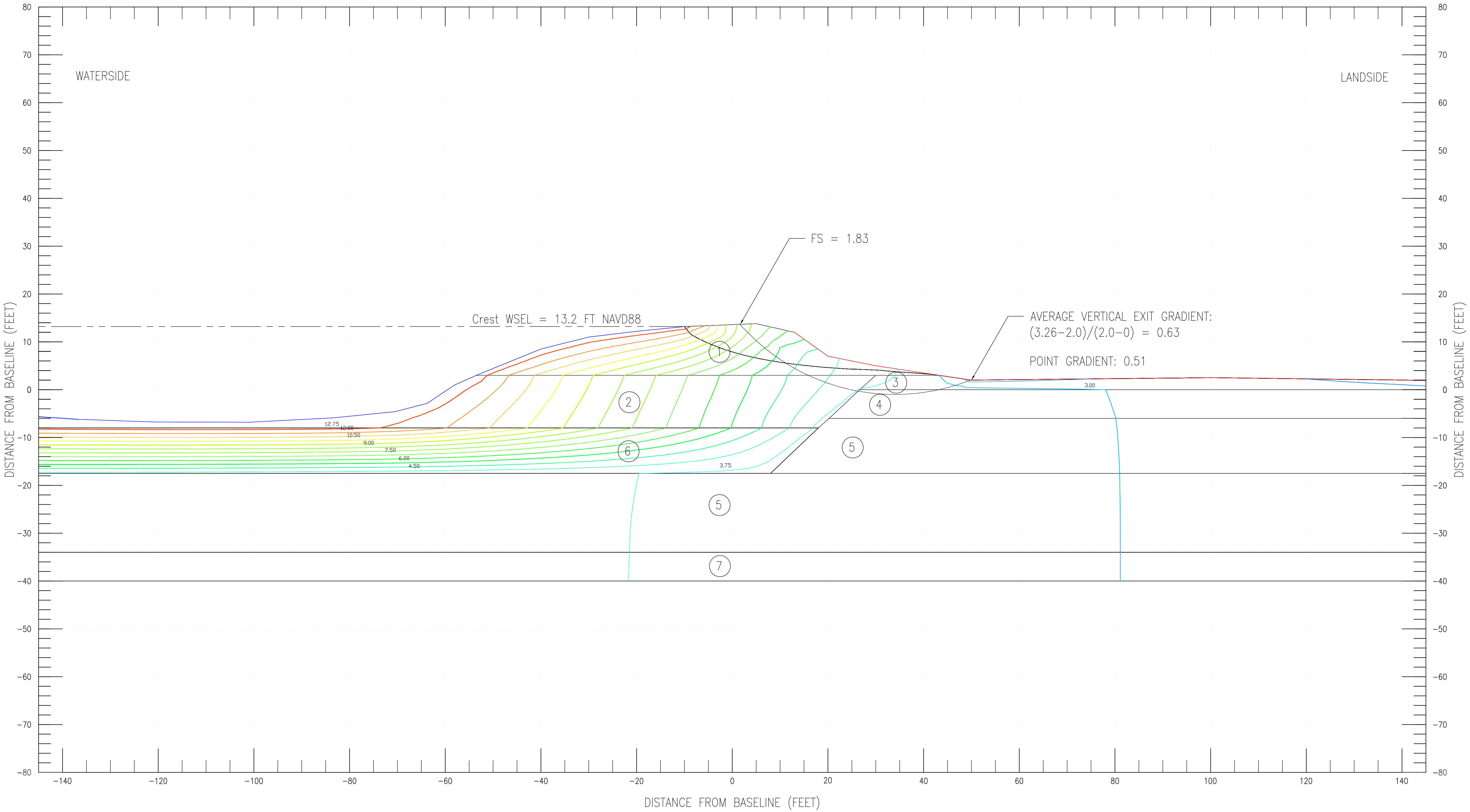
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO
CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results
CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
DELTA BROOKSIDE REACH D-BS STA. 166+50

DATE:	SCALE:	SHEET NO.
12 - August - 13	As Shown	13of 14

PLOT BY: L2EDGGAJ - Aug 21, 2013 - 11:35:28am
DRAWING: I:\draft-final geotechnical appendix\2 - enclosures\enclosure seepage stability results\lsj seepage stability plates\GMS\Delta FronBrookside.dwg



Boring Number	Layer ID	Soil Classification	Estimated Permeability for Seepage Analysis					Estimated Strength Parameters		
			Horizontal kh (kx) cm/sec	Anisotropy Ratio kh/kv	Vertical kv (ky) cm/sec	Horizontal kh (kx) ft/day	Vertical kv (ky) ft/day	Φ'	C' (psf)	γ (pcf)
WR1608_001B	1	Clay Levee	4.00E-06	4	1.00E-06	0.01134	0.00284	27	50	120
	2	Organic Soil	1.00E-04	10	1.00E-05	0.28350	0.02835	28	25	80
	3	Blanket	4.00E-06	4	1.00E-06	0.01134	0.00284	28	50	120
	4	Silty Sand	4.00E-04	4	1.00E-04	1.13400	0.28350	32	0	125
	5	Poorly graded Sand w/silt	1.00E-03	4	2.50E-04	2.83500	0.70875	34	0	125
	6	Foundation Clay	4.00E-06	4	1.00E-06	0.01134	0.00284	30	100	120
	7	Deep Clay Layer	4.00E-06	4	1.00E-06	0.01134	0.00284	30	100	120

0 10' 20'
HORIZONTAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

0 10' 20'
VERTICAL SCALE: 1"=10'
AT ORIGINAL D-SIZE DRAWING

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Seepage and Stability Results
CREST WSE STEADY STATE SEEPAGE AND SLOPE STABILITY
DELTA LINCOLN VILLAGE REACH D-LV STA. 162+50

DATE:	SCALE:	SHEET NO.
12 - August - 13	As Shown	14of 14

PLOT BY: L2EDGGAJ - Aug 21, 2013 - 1:23:39pm
DRAWING: I:\draft-final geotechnical appendix\2 - enclosures\enclosure seepage stability results\lsj seepage stability plates\GMS\Delta FronLincolnVillage.dwg

LOWER SAN JOAQUIN RIVER
FEASIBILITY STUDY

GEOTECHNICAL REPORT

ENCLOSURE E3
RISK AND UNCERTAINTY ANALYSES

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: San Joaquin River
Basin and Reach: Index Point LR1

Levee Mile: 1292+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 25.00
L/S Toe Elev.: 12.42
W/S Toe Elev.: 11.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 12/18/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material Material	Aquifer Material Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
WR0017_016C	16	13	6	41	46	40	28	16	288	57	CL	0.0007	SP-SM	14.18	20257	14838	8004	75668237	54
WR0017_017C	22					35					CL	0.0007	SP-SM	14.18	20257				
WR0017_020C	18					38					CL	0.0007	SP-SM	14.18	20257				
WR0017_021C	14					46					CL	0.0007	SP-SM	14.18	20257				
WR0017_025C	14					20					CL	0.0007	SM	2.8	4000				
WR0017_027C	12					28					CL	0.0007	SM	2.8	4000				
WR0017_029B	4					12					CL	0.0007	SP	14.18	20257				
WR0017_031C	12					8					CL	0.0007	SM	2.8	4000				
WR0017_034C	16					54					CL	0.0007	SM	14.18	20257				
WR0017_036B	10					5					CL	0.0007	SM	2.8	4000				
WR0017_041B	2					32					CL	0.0007	SP-SM	14.18	20257				
WR0017_039C	14					14					CL	0.0007	SP-SM	14.18	20257				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WR0017_016C	CL	16	0.0007				16	SP-SM	40	14.18							14.18
WR0017_017C	CL	22	0.0007				22	SP-SM	35	14.18							14.18
WR0017_020C	CL	18	0.0007				18	SP-SM	38	14.18							14.18
WR0017_021C	CL	14	0.0007				14	SP-SM	46	14.18							14.18
WR0017_025C	CL	14	0.0007				14	SM	20	2.8							2.8
WR0017_027C	CL	12	0.0007				12	SM	28	2.8							2.8
WR0017_029B	CL	4	0.0007				4	SP	12	14.18							14.18
WR0017_031C	CL	12	0.0007				12	SM	8	2.8							2.8
WR0017_034C	CL	16	0.0007				16	SM	54	14.18							14.18
WR0017_036B	CL	10	0.0007				10	SM	5	2.8							2.8
WR0017_041B	CL	2	0.0007				2	SP-SM	32	14.18							14.18
WR0017_039C	CL	14	0.0007				14	SP-SM	14	14.18							14.18

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR1

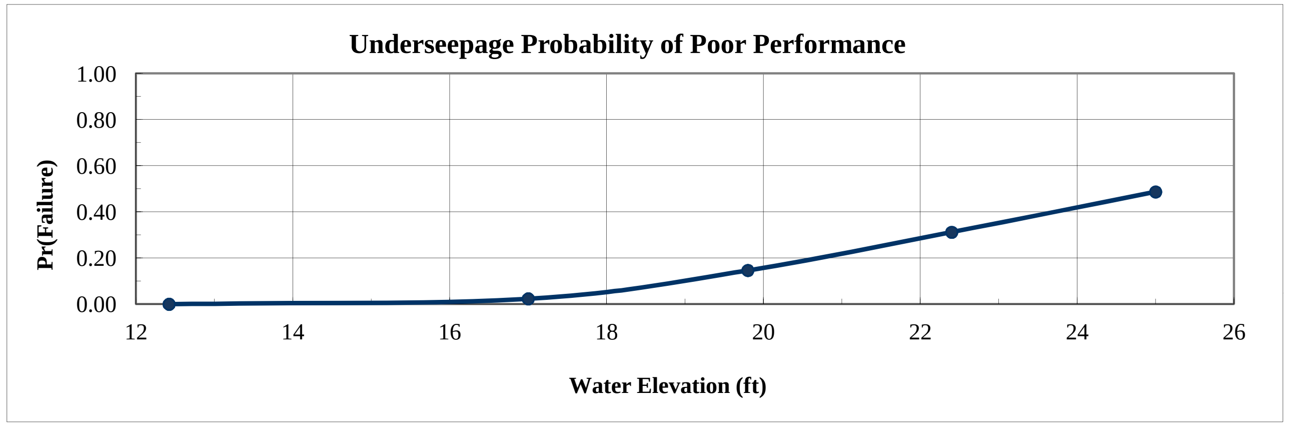
Levee Mile: 1292+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 25.00
L/S Toe Elev.: 12.42
W/S Toe Elev.: 11.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 12/18/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permaebility Ratio	14838	8004	54
Blanket Thickness (z)	13	6	46
Aquifer Thickness (d)	28	16	57

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	90	95	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	12.42	0.0000
Elev. 17.0	4.58	17.00	0.0234
200 yr	7.38	19.80	0.1465
Elev. 22.4	9.98	22.40	0.3121
Crest	12.58	25.00	0.4868

Crest	Rh
Head = 12.58	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	14838	13.00	28.00	89.96	2324.01	0.0112	11.65	0.90		
2	22842	13.00	28.00	89.97	2883.49	0.0091	11.82	0.91	0.000400	0.16
3	6834	13.00	28.00	89.90	1577.21	0.0159	11.26	0.87		
4	14838	19.00	28.00	89.97	2809.59	0.0094	11.80	0.62	0.250000	99.59
5	14838	7.00	28.00	89.92	1705.36	0.0148	11.35	1.62		
6	14838	13.00	44.00	89.97	2913.30	0.0142	11.83	0.91	0.000625	0.25
7	14838	13.00	12.00	89.90	1521.42	0.0070	11.22	0.86		
								Total	0.251025	100.00

E[I] = 0.900000
Var[I]= 0.251025
σ[I]= 0.501024
V(I) = 0.556693
E[ln I] = -0.240339
σ [ln I] = 0.519573
Ic= 0.80
ln(I crit) = -0.223144

β =	-0.462569
F(z) =	0.513201
Pr(f) % =	48.679948

Elev. 22.4	Rh
Head = 9.98	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	14838	13.00	28.00	89.96	2324.01	0.0112	9.24	0.71		
2	22842	13.00	28.00	89.97	2883.49	0.0091	9.38	0.72	0.000225	0.14
3	6834	13.00	28.00	89.90	1577.21	0.0159	8.93	0.69		
4	14838	19.00	28.00	89.97	2809.59	0.0094	9.36	0.49	0.160000	99.61
5	14838	7.00	28.00	89.92	1705.36	0.0148	9.00	1.29		
6	14838	13.00	44.00	89.97	2913.30	0.0142	9.38	0.72	0.000400	0.25
7	14838	13.00	12.00	89.90	1521.42	0.0070	8.90	0.68		
								Total	0.160625	100.00

E[I] = 0.710000
Var[I]= 0.160625
σ[I]= 0.400780
V(I) = 0.564480
E[ln I] = -0.480790
σ [ln I] = 0.525927
Ic= 0.80
ln(I crit) = -0.223144

β =	-0.914176
F(z) =	0.687894
Pr(f) % =	31.210589

200 yr	Rh
Head = 7.38	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	14838	13.00	28.00	89.96	2324.01	0.0112	6.84	0.53		
2	22842	13.00	28.00	89.97	2883.49	0.0091	6.94	0.53	0.000100	0.11
3	6834	13.00	28.00	89.90	1577.21	0.0159	6.61	0.51		
4	14838	19.00	28.00	89.97	2809.59	0.0094	6.92	0.36	0.087025	99.77
5	14838	7.00	28.00	89.92	1705.36	0.0148	6.66	0.95		
6	14838	13.00	44.00	89.97	2913.30	0.0142	6.94	0.53	0.000100	0.11
7	14838	13.00	12.00	89.90	1521.42	0.0070	6.58	0.51		
								Total	0.087225	100.00

E[I] = 0.530000
Var[I]= 0.087225
σ[I]= 0.295339
V(I) = 0.557243
E[ln I] = -0.770090
σ [ln I] = 0.520023
Ic= 0.80
ln(I crit) = -0.223144

β =	-1.480877
F(z) =	0.853548
Pr(f) % =	14.645160

Elev. 17.0	Rh
Head = 4.58	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	14838	13.00	28.00	89.96	2324.01	0.0112	4.24	0.33		
2	22842	13.00	28.00	89.97	2883.49	0.0091	4.30	0.33	0.000025	0.08
3	6834	13.00	28.00	89.90	1577.21	0.0159	4.10	0.32		
4	14838	19.00	28.00	89.97	2809.59	0.0094	4.30	0.23	0.032400	99.62
5	14838	7.00	28.00	89.92	1705.36	0.0148	4.13	0.59		
6	14838	13.00	44.00	89.97	2913.30	0.0142	4.31	0.33	0.000100	0.31
7	14838	13.00	12.00	89.90	1521.42	0.0070	4.08	0.31		
								Total	0.032525	100.00

E[I] = 0.330000
Var[I]= 0.032525
σ[I]= 0.180347
V(I) = 0.546506
E[ln I] = -1.239332
σ [ln I] = 0.511214
Ic= 0.80
ln(I crit) = -0.223144

β =	-2.424294
F(z) =	0.976583
Pr(f) % =	2.341711

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

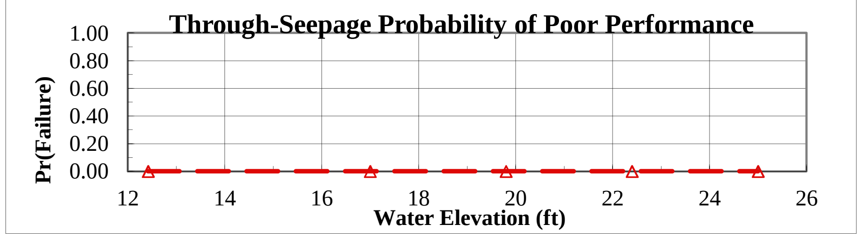
Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR1

Levee Mile: 1292+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 25.00
L/S Toe Elev.: 12.42
W/S Toe Elev.: 11.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 12/18/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	50	5.0	10.00
Initial Porosity (n)	0.4	0.04	10.00
Initial Permeability (Ko)	1.00E-10	3.00E-11	30.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	12.42	0.0000
Elev. 17.0	4.58	17.00	0.000000
200 yr	7.38	19.80	0.000000
Elev. 22.4	9.98	22.40	0.000000
Crest	12.58	25.00	0.000000

Pr(f)=0
NO

Crest	**Head =**	12.58	**Horizontal Gradient (Ix) =**	0.440

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	50.00	0.40	1.00E-10	1140.21	2591.38		
2	45.00	0.40	1.00E-10	1026.19	2332.25	67152.701113	26.44
3	55.00	0.40	1.00E-10	1254.23	2850.52		
4	50.00	0.36	1.00E-10	1081.70	2458.40	16830.356890	6.63
5	50.00	0.44	1.00E-10	1195.86	2717.87		
6	50.00	0.40	7.00E-11	1362.81	3097.30	169950.936279	66.93
7	50.00	0.40	1.30E-10	1000.03	2272.79		

E[FS] =	2591.383822	E[ln FS] =	7.841389	Total	253933.994282	100.00
Var[FS]=	253933.994282					
σ[FS]=	503.918639	σ[ln FS]=	0.192658	β =	40.701152
V(FS) =	0.194459			F(z) =	0.000000
FS req'd =	1.00	ln(FS req'd) =	0.000000	**Pr(f) % =**	0.000000

Elev. 22.4	**Head =**	9.98	**Horizontal Gradient (Ix) =**	0.430

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	50.00	0.40	1.00E-10	1140.21	2651.65		
2	45.00	0.40	1.00E-10	1026.19	2386.48	70312.400949	26.44
3	55.00	0.40	1.00E-10	1254.23	2916.81		
4	50.00	0.36	1.00E-10	1081.70	2515.57	17622.266597	6.63
5	50.00	0.44	1.00E-10	1195.86	2781.07		
6	50.00	0.40	7.00E-11	1362.81	3169.33	177947.546045	66.93
7	50.00	0.40	1.30E-10	1000.03	2325.65		

E[FS] =	2651.648562	E[ln FS] =	7.864378	Total	265882.213591	100.00
Var[FS]=	265882.213591					
σ[FS]=	515.637677	σ[ln FS]=	0.192658	β =	40.820480	
V(FS) =	0.194459			F(z) =	0.000000	
FS req'd =	1.00	ln(FS req'd) =	0.000000	**Pr(f) % =**	0.000000	

200 yr	**Head =**	7.38	**Horizontal Gradient (Ix) =**	0.410

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	50.00	0.40	1.00E-10	1140.21	2781.00		
2	45.00	0.40	1.00E-10	1026.19	2502.90	77339.458272	26.44
3	55.00	0.40	1.00E-10	1254.23	3059.10		
4	50.00	0.36	1.00E-10	1081.70	2638.29	19383.444936	6.63
5	50.00	0.44	1.00E-10	1195.86	2916.73		
6	50.00	0.40	7.00E-11	1362.81	3323.93	195731.714835	66.93
7	50.00	0.40	1.30E-10	1000.03	2439.10		

E[FS] =	2780.997272	E[ln FS] =	7.912006	Total	292454.618043	100.00
Var[FS]=	292454.618043					
σ[FS]=	540.790734	σ[ln FS]=	0.192658	β =	41.067696	
V(FS) =	0.194459			F(z) =	0.000000	
FS req'd =	1.00	ln(FS req'd) =	0.000000	**Pr(f) % =**	0.000000	

Elev. 17.0	**Head =**	4.58	**Horizontal Gradient (Ix) =**	0.370

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	50.00	0.40	1.00E-10	1140.21	3081.65		
2	45.00	0.40	1.00E-10	1026.19	2773.48	94965.397630	26.44
3	55.00	0.40	1.00E-10	1254.23	3389.81		
4	50.00	0.36	1.00E-10	1081.70	2923.51	23801.001416	6.63
5	50.00	0.44	1.00E-10	1195.86	3232.06		
6	50.00	0.40	7.00E-11	1362.81	3683.27	240339.673219	66.93
7	50.00	0.40	1.30E-10	1000.03	2702.78		

E[FS] =	3081.645626	E[ln FS] =	8.014661	Total	359106.072265	100.00
Var[FS]=	359106.072265					
σ[FS]=	599.254597	σ[ln FS]=	0.192658	β =	41.600528	
V(FS) =	0.194459			F(z) =	0.000000	
FS req'd =	1.00	ln(FS req'd) =	0.000000	**Pr(f) % =**	0.000000	

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

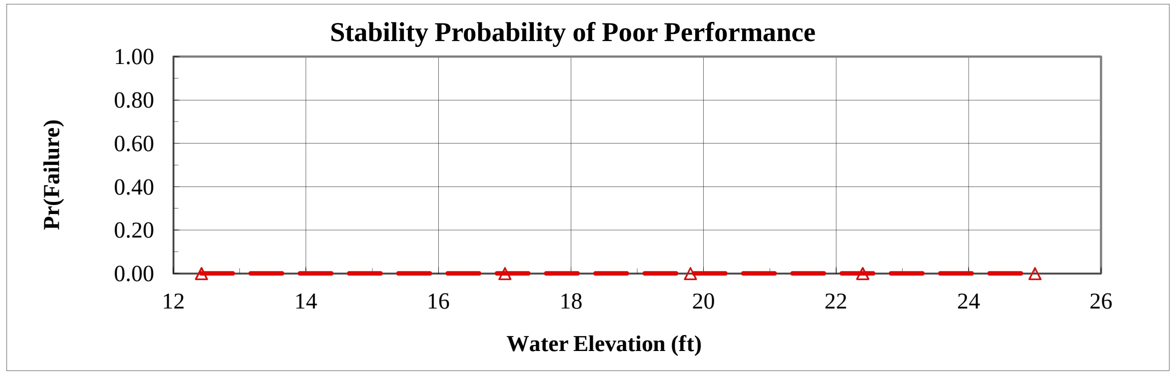
Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR1

Levee Mile: 1292+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 25.00
L/S Toe Elev.: 12.42
W/S Toe Elev.: 11.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 12/18/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	28	4	13.00
Levee Cohesion	50	20	40.00
Levee γ	120	8	7.00
Foundation Φ	30	4	13.00
Foundation Cohesion	100	40	40.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	12.42	0.0000
Elev. 17.0	4.58	17.00	0.000000
200 yr	7.38	19.80	0.000000
Elev. 22.4	9.98	22.40	0.000000
Crest	12.58	25.00	0.000000

Crest	**Head =**	12.58	**Pr(f)=0**	NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	28	50	120	30	100	1.33		
2	24	50	120	30	100	1.30	0.001024	50.28
3	32	50	120	30	100	1.36		
4	28	30	120	30	100	1.31	0.000380	18.67
5	28	70	120	30	100	1.35		
6	28	50	112	30	100	1.36	0.000240	11.80
7	28	50	128	30	100	1.32		
8	28	50	120	26	100	1.36	0.000196	9.62
9	28	50	120	34	100	1.33		
10	28	50	120	30	60	1.36	0.000196	9.62
11	28	50	120	30	140	1.33		

E[FS] =	1.329000	E[ln FS] =	0.283851	Total	0.002037	100.00
Var[FS]=	0.002037					
σ[FS]=	0.045128	σ[ln FS]=	0.033946		β =	8.361761
V(FS) =	0.033956				F(z) =	0.000000
FS req'd =	1.00	ln(FS req'd) =	0.000000		**Pr(f) % =**	0.000000

Elev. 22.4	**Head =**	9.98	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	28	50	120	30	100	1.56		
2	24	50	120	30	100			
3	32	50	120	30	100			
4	28	30	120	30	100			
5	28	70	120	30	100			
6	28	50	112	30	100			
7	28	50	128	30	100			
8	28	50	120	26	100			
9	28	50	120	34	100			
10	28	50	120	30	60			
11	28	50	120	30	140			

E[FS] =		E[ln FS] =		Total		
Var[FS]=						
σ[FS]=		σ[ln FS]=			β =	
V(FS) =					F(z) =	
FS req'd =	1.00	ln(FS req'd) =	0.000000		**Pr(f) % =**	0.000000

200 yr	**Head =**	7.38	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	28	50	120	30	100	1.66		
2	24	50	120	30	100			
3	32	50	120	30	100			
4	28	30	120	30	100			
5	28	70	120	30	100			
6	28	50	112	30	100			
7	28	50	128	30	100			
8	28	50	120	26	100			
9	28	50	120	34	100			
10	28	50	120	30	60			
11	28	50	120	30	140			

E[FS] =		E[ln FS] =		Total		
Var[FS]=						
σ[FS]=		σ[ln FS]=			β =	
V(FS) =					F(z) =	
FS req'd =	1.00	ln(FS req'd) =	0.000000		**Pr(f) % =**	0.000000

Elev. 17.0	**Head =**	4.58	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	28	50	120	30	100	1.83		
2	24	50	120	30	100			
3	32	50	120	30	100			
4	28	30	120	30	100			
5	28	70	120	30	100			
6	28	50	112	30	100			
7	28	50	128	30	100			
8	28	50	120	26	100			
9	28	50	120	34	100			
10	28	50	120	30	60			
11	28	50	120	30	140			

E[FS] =		E[ln FS] =		Total		
Var[FS]=						
σ[FS]=		σ[ln FS]=			β =	
V(FS) =					F(z) =	
FS req'd =	1.00	ln(FS req'd) =	0.000000		**Pr(f) % =**	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

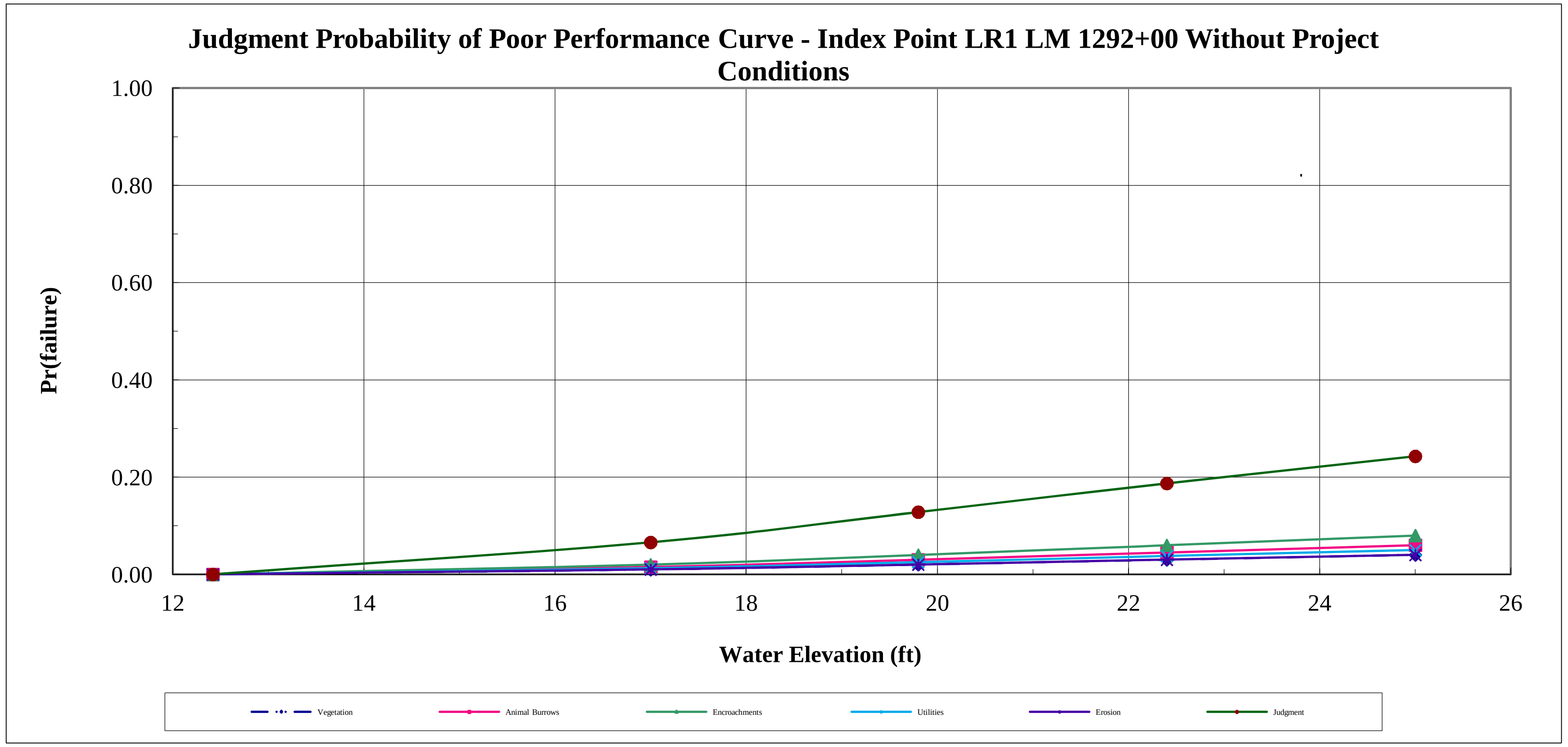
Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR1

Levee Mile: 1292+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 25.00
L/S Toe Elev.: 12.42
W/S Toe Elev.: 11.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. F
Date: 12/18/2012

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
12.42	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
17.00	0.0100	0.9900	0.0150	0.9850	0.0200	0.9800	0.0125	0.9875	0.0100	0.9900	0.0657	0.9343
19.80	0.0200	0.9800	0.0300	0.9700	0.0400	0.9600	0.0250	0.9750	0.0200	0.9800	0.1280	0.8720
22.40	0.0300	0.9700	0.0450	0.9550	0.0600	0.9400	0.0375	0.9625	0.0300	0.9700	0.1870	0.8130
25.00	0.0400	0.9600	0.0600	0.9400	0.0800	0.9200	0.0500	0.9500	0.0400	0.9600	0.2429	0.7571


Judgment Probability of Poor Performance Curve - Index Point LR1 LM 1292+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

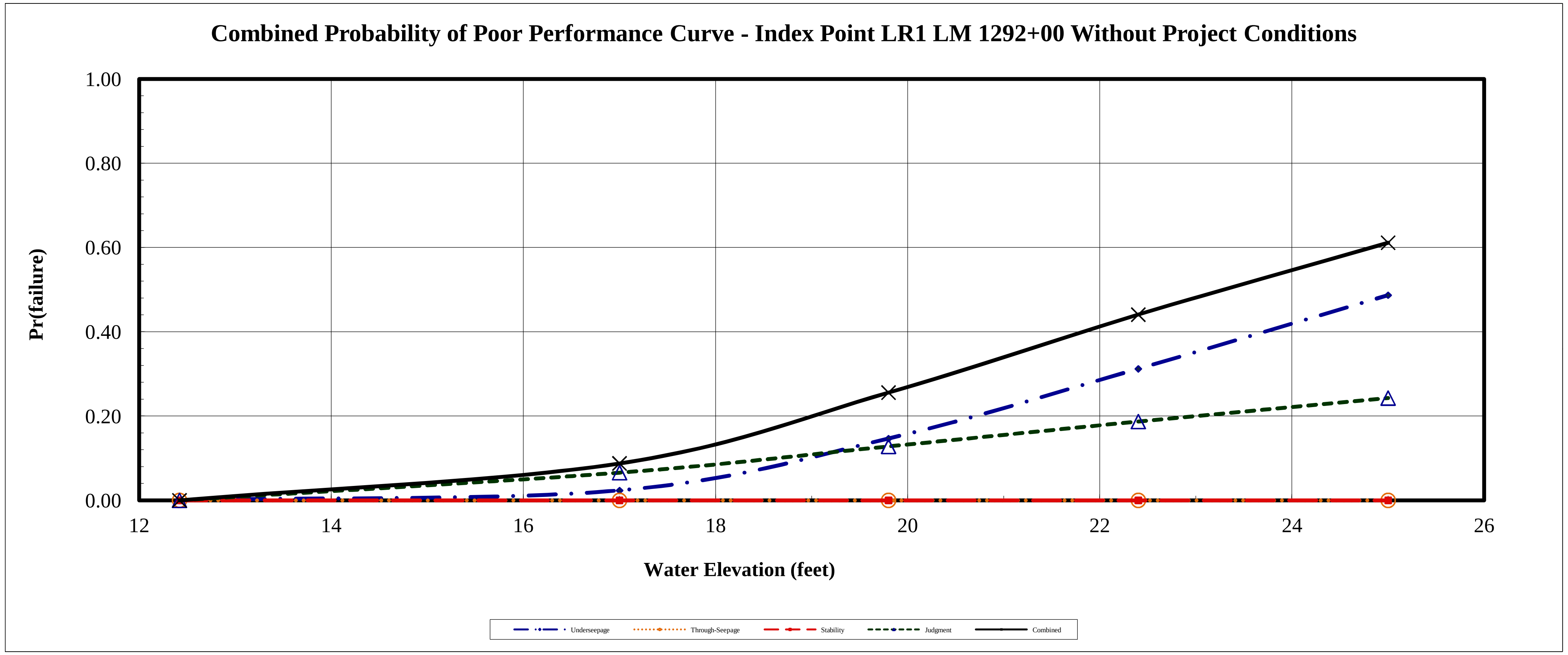
Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR1

Levee Mile: 1292+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 25.00
L/S Toe Elev.: 12.42
W/S Toe Elev.: 11.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Per
Date: 12/18/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
12.42	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
17.00	0.0234	0.9766	0.0000	1.0000	0.0000	1.0000	0.0657	0.9343	0.0876	0.9124
19.80	0.1465	0.8535	0.0000	1.0000	0.0000	1.0000	0.1280	0.8720	0.2557	0.7443
22.40	0.3121	0.6879	0.0000	1.0000	0.0000	1.0000	0.1870	0.8130	0.4408	0.5592
25.00	0.4868	0.5132	0.0000	1.0000	0.0000	1.0000	0.2429	0.7571	0.6114	0.3886


Combined Probability of Poor Performance Curve - Index Point LR1 LM 1292+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: Right Bank San Joaquin River
Basin and Reach: Index Point LR2

Levee Mile: STA 1417+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 27.80
L/S Toe Elev.: 12.00
W/S Toe Elev.: 12.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket		Aquifer Material		Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
											Material	Kb (ft/day)	Material	Kf (ft/day)					
WR0017_047B	10	7	4	20	57	28	18	7	95	39	CL	0.0007	SP-SM	0.28	400	126	170	25246	98
WR0017_049C	12					26					CL	0.0007	SP-SM	0.28	400				
WR0017_052B	8					10					SM	0.007	SP-SM	0.28	40				
WR0017_055C	6					12					SM	0.007	SP-SM	0.28	40				
WR0017_057B	4					20					SM	0.007	SM	0.028	4				
WR0017_063B	11					22					CL	0.0007	SM	0.028	40				
WR0017_064C	3					16					CL	0.0007	SM	0.028	40				
WR0017_065C	2					12					CL	0.0007	SM	0.028	40				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WR0017_047B	CL	10	0.0007				10	SP-SM	28	0.28							0.28
WR0017_049C	CL	12	0.0007				12	SP-SM	26	0.28							0.28
WR0017_052B	SM	8	0.007				8	SP-SM	10	0.28							0.28
WR0017_055C	SM	6	0.007				6	SP-SM	12	0.28							0.28
WR0017_057B	SM	4	0.007				4	SM	20	0.028							0.028
WR0017_063B	CL	11	0.0007				11	SM	22	0.028							0.028
WR0017_064C	CL	3	0.0007				3	SM	16	0.028							0.028
WR0017_065C	CL	2	0.0007				2	SM	12	0.028							0.028

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: Right Bank San Joaquin River
River Section: Index Point LR2

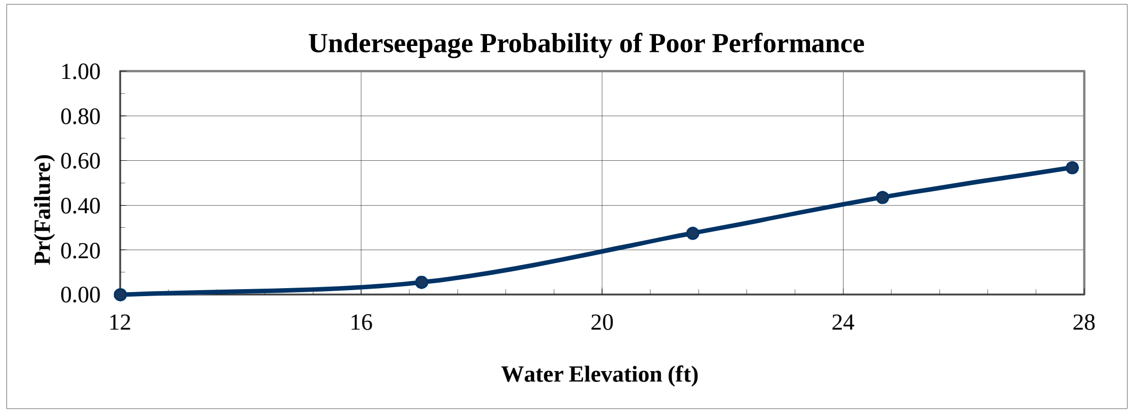
Levee Mile: STA 1417+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 27.80
L/S Toe Elev.: 12.00
W/S Toe Elev.: 12.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permaebility Ratio	126	123	98
Blanket Thickness (z)	7	4	57
Aquifer Thickness (d)	18	7	39

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	75	62	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	12.00	0.0000
Elev. 17.0	5.00	17.00	0.0555
200 year	9.50	21.50	0.2749
Elev. 24.65	12.65	24.65	0.4353
Crest	15.80	27.80	0.5685

Crest	Rh
Head = 15.80	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	126	7.00	18.00	67.24	126.00	0.0705	7.80	1.11		
2	249	7.00	18.00	70.82	177.30	0.0580	9.03	1.29	0.193600	29.32
3	3	7.00	18.00	17.81	17.82	0.1844	2.88	0.41		
4	126	11.00	18.00	69.83	157.95	0.0621	8.61	0.78	0.455625	69.01
5	126	3.00	18.00	59.45	82.49	0.0883	6.39	2.13		
6	126	7.00	25.00	69.21	148.49	0.0894	8.39	1.20	0.011025	1.67
7	126	7.00	11.00	63.23	98.50	0.0492	6.96	0.99		
								Total	0.660250	100.00

E[I] = 1.110000
E[ln I] = -0.110190
Var[I]= 0.660250
σ[I]= 0.812558
σ [ln I] = 0.655057
V(I) = 0.732034
β = -0.168214
F(z) = 0.431548
Ic= 0.80
ln(I crit) = -0.223144
Pr(f) % = 56.845171

Elev. 24.65	Rh
Head = 12.65	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	126	7.00	18.00	67.24	126.00	0.0705	6.24	0.89		
2	249	7.00	18.00	70.82	177.30	0.0580	7.23	1.03	0.122500	29.13
3	3	7.00	18.00	17.81	17.82	0.1844	2.31	0.33		
4	126	11.00	18.00	69.83	157.95	0.0621	6.90	0.63	0.291600	69.35
5	126	3.00	18.00	59.45	82.49	0.0883	5.12	1.71		
6	126	7.00	25.00	69.21	148.49	0.0894	6.72	0.96	0.006400	1.52
7	126	7.00	11.00	63.23	98.50	0.0492	5.57	0.80		
								Total	0.420500	100.00

E[I] = 0.890000
E[ln I] = -0.329451
Var[I]= 0.420500
σ[I]= 0.648460
σ [ln I] = 0.652560
V(I) = 0.728606
β = -0.504859
F(z) = 0.564705
Ic= 0.80
ln(I crit) = -0.223144
Pr(f) % = 43.529528

200 year	Rh
Head = 9.50	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	126	7.00	18.00	67.24	126.00	0.0705	4.69	0.67		
2	249	7.00	18.00	70.82	177.30	0.0580	5.43	0.78	0.070225	29.52
3	3	7.00	18.00	17.81	17.82	0.1844	1.73	0.25		
4	126	11.00	18.00	69.83	157.95	0.0621	5.18	0.47	0.164025	68.96
5	126	3.00	18.00	59.45	82.49	0.0883	3.84	1.28		
6	126	7.00	25.00	69.21	148.49	0.0894	5.04	0.72	0.003600	1.51
7	126	7.00	11.00	63.23	98.50	0.0492	4.18	0.60		
								Total	0.237850	100.00

E[I] = 0.670000
E[ln I] = -0.613063
Var[I]= 0.237850
σ[I]= 0.487699
σ [ln I] = 0.652051
V(I) = 0.727908
β = -0.940207
F(z) = 0.725076
Ic= 0.80
ln(I crit) = -0.223144
Pr(f) % = 27.492367

Elev. 17.0	Rh
Head = 5.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	126	7.00	18.00	67.24	126.00	0.0705	2.47	0.35		
2	249	7.00	18.00	70.82	177.30	0.0580	2.86	0.41	0.019600	30.19
3	3	7.00	18.00	17.81	17.82	0.1844	0.91	0.13		
4	126	11.00	18.00	69.83	157.95	0.0621	2.73	0.25	0.044100	67.92
5	126	3.00	18.00	59.45	82.49	0.0883	2.02	0.67		
6	126	7.00	25.00	69.21	148.49	0.0894	2.65	0.38	0.001225	1.89
7	126	7.00	11.00	63.23	98.50	0.0492	2.20	0.31		
								Total	0.064925	100.00

E[I] = 0.350000
E[ln I] = -1.262456
Var[I]= 0.064925
σ[I]= 0.254804
σ [ln I] = 0.652126
V(I) = 0.728011
β = -1.935909
F(z) = 0.944502
Ic= 0.80
ln(I crit) = -0.223144
Pr(f) % = 5.549819

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Lower San Joaquin
Study Area: Right Bank San Joaquin River
River Section: Index Point LR2

Levee Mile: STA 1417+00
River Mile: XX.XX
Analysis Case Without Project Conditions

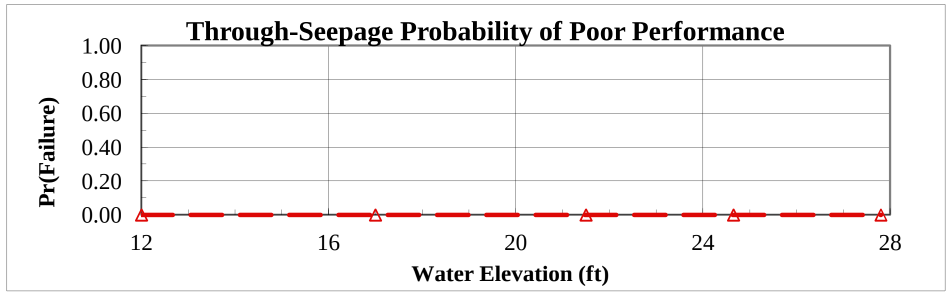
Crest Elev.: 27.80
L/S Toe Elev.: 12.00
W/S Toe Elev.: 12.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Random Variables

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Tractive Stress (Tc)	5	0.5	10.00
Initial Porosity (n)	0.4	0.04	10.00
Initial Permeability (Ko)	1.00E-10	3.00E-11	30.00

Pr(f)=0
NO



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	12.00	0.0000
Elev. 17.0	5.00	17.00	0.000000
200 year	9.50	21.50	0.000000
Elev. 24.65	12.65	24.65	0.000000
Crest	15.80	27.80	0.000000

Crest **Head =** 15.80 **Horizontal Gradient (Ix) =** 0.240

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	5.00	0.40	1.00E-10	114.02	475.09		
2	4.50	0.40	1.00E-10	102.62	427.58	2257.076899	26.44
3	5.50	0.40	1.00E-10	125.42	522.60		
4	5.00	0.36	1.00E-10	108.17	450.71	565.686995	6.63
5	5.00	0.44	1.00E-10	119.59	498.28		
6	5.00	0.40	7.00E-11	136.28	567.84	5712.239803	66.93
7	5.00	0.40	1.30E-10	100.00	416.68		

E[FS] = 475.087034 E[ln FS] = 6.144940 Total 8535.003697 100.00
Var[FS]= 8535.003697
σ[FS]= 92.385084 σ[ln FS]= 0.192658 β = 31.895640
V(FS) = 0.194459 F(z) = 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Elev. 24.65 **Head =** 12.65 **Horizontal Gradient (Ix) =** 0.190

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	5.00	0.40	1.00E-10	114.02	600.11		
2	4.50	0.40	1.00E-10	102.62	540.10	3601.319373	26.44
3	5.50	0.40	1.00E-10	125.42	660.12		
4	5.00	0.36	1.00E-10	108.17	569.31	902.591993	6.63
5	5.00	0.44	1.00E-10	119.59	629.40		
6	5.00	0.40	7.00E-11	136.28	717.27	9114.266278	66.93
7	5.00	0.40	1.30E-10	100.00	526.33		

E[FS] = 600.109938 E[ln FS] = 6.378554 Total 13618.177643 100.00
Var[FS]= 13618.177643
σ[FS]= 116.696948 σ[ln FS]= 0.192658 β = 33.108231
V(FS) = 0.194459 F(z) = 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

200 year **Head =** 9.50 **Horizontal Gradient (Ix) =** 0.140

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	5.00	0.40	1.00E-10	114.02	814.43		
2	4.50	0.40	1.00E-10	102.62	732.99	6633.042314	26.44
3	5.50	0.40	1.00E-10	125.42	895.88		
4	5.00	0.36	1.00E-10	108.17	772.64	1662.427089	6.63
5	5.00	0.44	1.00E-10	119.59	854.19		
6	5.00	0.40	7.00E-11	136.28	973.44	16786.990441	66.93
7	5.00	0.40	1.30E-10	100.00	714.31		

E[FS] = 814.434915 E[ln FS] = 6.683936 Total 25082.459843 100.00
Var[FS]= 25082.459843
σ[FS]= 158.374429 σ[ln FS]= 0.192658 β = 34.693331
V(FS) = 0.194459 F(z) = 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Elev. 17.0 **Head =** 5.00 **Horizontal Gradient (Ix) =** 0.090

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	5.00	0.40	1.00E-10	114.02	1266.90		
2	4.50	0.40	1.00E-10	102.62	1140.21	16050.324612	26.44
3	5.50	0.40	1.00E-10	125.42	1393.59		
4	5.00	0.36	1.00E-10	108.17	1201.89	4022.663079	6.63
5	5.00	0.44	1.00E-10	119.59	1328.73		
6	5.00	0.40	7.00E-11	136.28	1514.23	40620.371930	66.93
7	5.00	0.40	1.30E-10	100.00	1111.14		

E[FS] = 1266.898757 E[ln FS] = 7.125769 Total 60693.359621 100.00
Var[FS]= 60693.359621
σ[FS]= 246.360223 σ[ln FS]= 0.192658 β = 36.986687
V(FS) = 0.194459 F(z) = 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

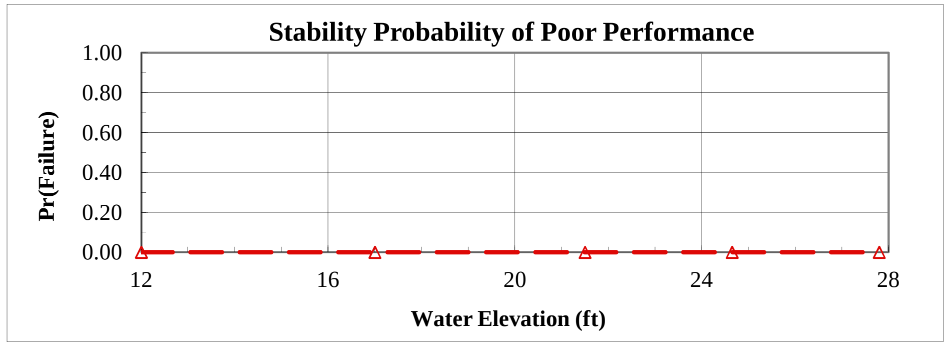
Project: Lower San Joaquin
Study Area: Right Bank San Joaquin River
River Section: Index Point LR2

Levee Mile: STA 1417+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 27.80
L/S Toe Elev.: 12.00
W/S Toe Elev.: 12.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	28	4	13.00
Levee Cohesion	100	40	40.00
Levee γ	120	8	7.00
Foundation Φ	30	4	13.00
Foundation Cohesion	0	0	40.00



Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	12.00	0.0000
Elev. 17.0	5.00	17.00	0.000000
200 year	9.50	21.50	0.000000
Elev. 24.65	12.65	24.65	0.000000
Crest	15.80	27.80	0.000000

Crest	**Head =**	15.80	**Pr(f)=0**	NO

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	28	100	120	30	0	1.94		
2	24	100	120	30	0	1.90	0.001225	1.86
3	32	100	120	30	0	1.97		
4	28	60	120	30	0	1.89	0.002352	3.57
5	28	140	120	30	0	1.98		
6	28	100	112	30	0	1.97	0.001296	1.97
7	28	100	128	30	0	1.90		
8	28	100	120	26	0	1.70	0.061009	92.60
9	28	100	120	34	0	2.19		
10	28	100	120	30	0	1.94	0.000000	0.00
11	28	100	120	30	0	1.94		

E[FS] = 1.940000 E[ln FS] = 0.654011 Total 0.065882 100.00
Var[FS]= 0.065882
σ[FS]= 0.256675 σ[ln FS]= 0.131733
V(FS) = 0.132307
FS req'd = 1.00 ln(FS req'd) = 0.000000

β =	4.964660
F(z) =	0.000000
Pr(f) % =	0.000034

Elev. 24.65	**Head =**	12.65	**Pr(f)=0**	YES

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	28	100	120	30	0	2.20		
2	24	100	120	30	0			
3	32	100	120	30	0			
4	28	60	120	30	0			
5	28	140	120	30	0			
6	28	100	112	30	0			
7	28	100	128	30	0			
8	28	100	120	26	0			
9	28	100	120	34	0			
10	28	100	120	30	0			
11	28	100	120	30	0			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]=
V(FS) =
FS req'd = 1.00 ln(FS req'd) = 0.000000

β =	
F(z) =	
Pr(f) % =	0.000000

200 year	**Head =**	9.50	**Pr(f)=0**	YES

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	28	100	120	30	0	2.48		
2	24	100	120	30	0			
3	32	100	120	30	0			
4	28	60	120	30	0			
5	28	140	120	30	0			
6	28	100	112	30	0			
7	28	100	128	30	0			
8	28	100	120	26	0			
9	28	100	120	34	0			
10	28	100	120	30	0			
11	28	100	120	30	0			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]=
V(FS) =
FS req'd = 1.00 ln(FS req'd) = 0.000000

β =	
F(z) =	
Pr(f) % =	0.000000

Elev. 17.0	**Head =**	5.00	**Pr(f)=0**	YES

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	28	100	120	30	0	2.88		
2	24	100	120	30	0			
3	32	100	120	30	0			
4	28	60	120	30	0			
5	28	140	120	30	0			
6	28	100	112	30	0			
7	28	100	128	30	0			
8	28	100	120	26	0			
9	28	100	120	34	0			
10	28	100	120	30	0			
11	28	100	120	30	0			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]=
V(FS) =
FS req'd = 1.00 ln(FS req'd) = 0.000000

β =	
F(z) =	
Pr(f) % =	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

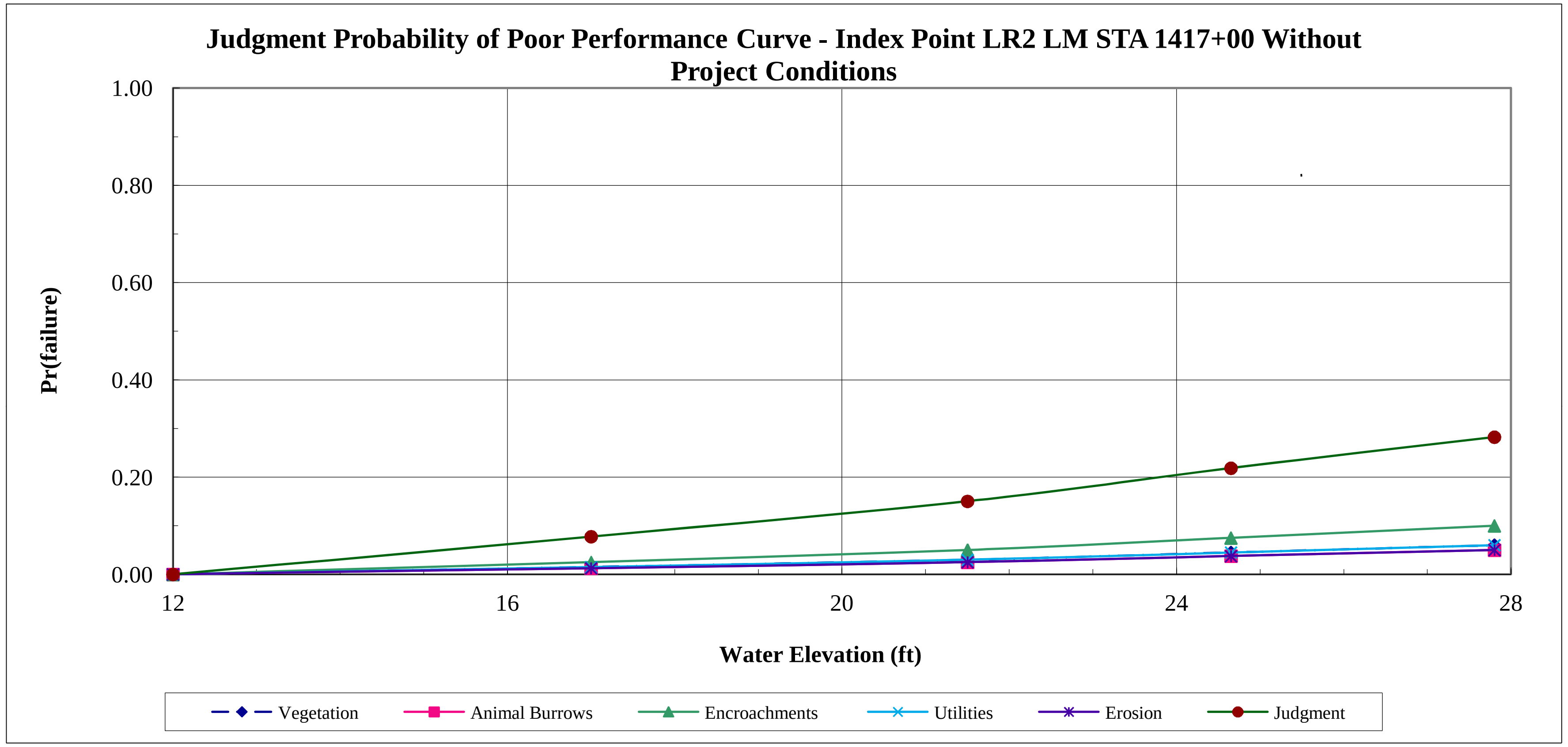
Project: Lower San Joaquin
Study Area: Right Bank San Joaquin River
River Section: Index Point LR2

Levee Mile: STA 1417+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 27.80
L/S Toe Elev.: 12.00
W/S Toe Elev.: 12.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
12.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
17.00	0.0150	0.9850	0.0125	0.9875	0.0250	0.9750	0.0150	0.9850	0.0125	0.9875	0.0775	0.9225
21.50	0.0300	0.9700	0.0250	0.9750	0.0500	0.9500	0.0300	0.9700	0.0250	0.9750	0.1503	0.8497
24.65	0.0450	0.9550	0.0375	0.9625	0.0750	0.9250	0.0450	0.9550	0.0375	0.9625	0.2185	0.7815
27.80	0.0600	0.9400	0.0500	0.9500	0.1000	0.9000	0.0600	0.9400	0.0500	0.9500	0.2823	0.7177



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

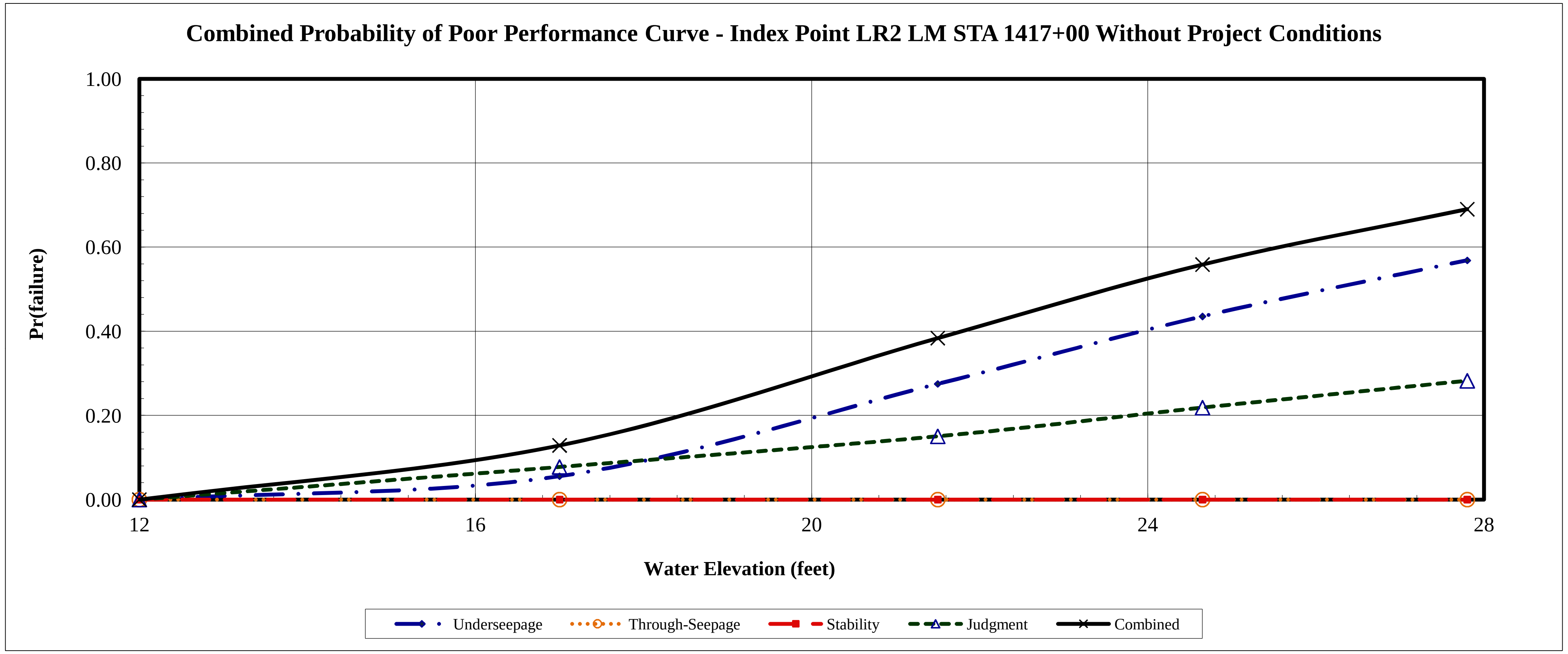
Project: Lower San Joaquin
Study Area: Right Bank San Joaquin River
River Section: Index Point LR2

Levee Mile: STA 1417+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 27.80
L/S Toe Elev.: 12.00
W/S Toe Elev.: 12.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
12.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
17.00	0.0555	0.9445	0.0000	1.0000	0.0000	1.0000	0.0775	0.9225	0.1287	0.8713
21.50	0.2749	0.7251	0.0000	1.0000	0.0000	1.0000	0.1503	0.8497	0.3839	0.6161
24.65	0.4353	0.5647	0.0000	1.0000	0.0000	1.0000	0.2185	0.7815	0.5587	0.4413
27.80	0.5685	0.4315	0.0000	1.0000	0.0000	1.0000	0.2823	0.7177	0.6903	0.3097


Combined Probability of Poor Performance Curve - Index Point LR2 LM STA 1417+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: San Joaquin River
Basin and Reach: Index Point LR3

Levee Mile: 1685+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 31.00
L/S Toe Elev.: 18.53
W/S Toe Elev.: 17.80

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 12/19/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket		Aquifer Material		Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
											Material	Kb (ft/day)	Material	Kf (ft/day)					
WR0017_067C	16	11	6	43	55	26	35	12	239	34	CL	0.0007	SM	0.28	400	6933	9800	90176000	98
WR0017_070C	18					24					CL	0.0007	SM	0.28	400				
WR0017_071C	8					45					CL	0.0007	SM	0.28	400				
WR0017_072C	16					52					CL	0.0007	SM	0.28	400				
WR0017_075C	18					18					CL	0.0007	SP	14	20000				
WR0017_076C	10					26					CL	0.0007	SP	14	20000				
WR0017_080B	3					42					CL	0.0007	SM	0.28	400				
WR0017_081C	10					40					CL	0.0007	SM	0.28	400				
WR0017_085B	4					40					CL	0.0007	SP-SM	14	20000				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WR0017_067C	CL	16	0.0007				16	SM	26	0.28							0.28
WR0017_069B	ML	6	0.035				6	SP-SM	16	14							14
WR0017_070C	CL	18	0.0007				18	SM	24	0.28							0.28
WR0017_071C	CL	8	0.0007				8	SM	45	0.28							0.28
WR0017_072C	CL	16	0.0007				16	SM	52	0.28							0.28
WR0017_075C	CL	18	0.0007				18	SP	18	14							14
WR0017_076C	CL	10	0.0007				10	SP	26	14							14
WR0017_080B	CL	3	0.0007				3	SM	42	0.28							0.28
WR0017_081C	CL	10	0.0007				10	SM	40	0.28							0.28
WR0017_085B	CL	4	0.0007				**4**	SP-SM	40	14							14

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR3

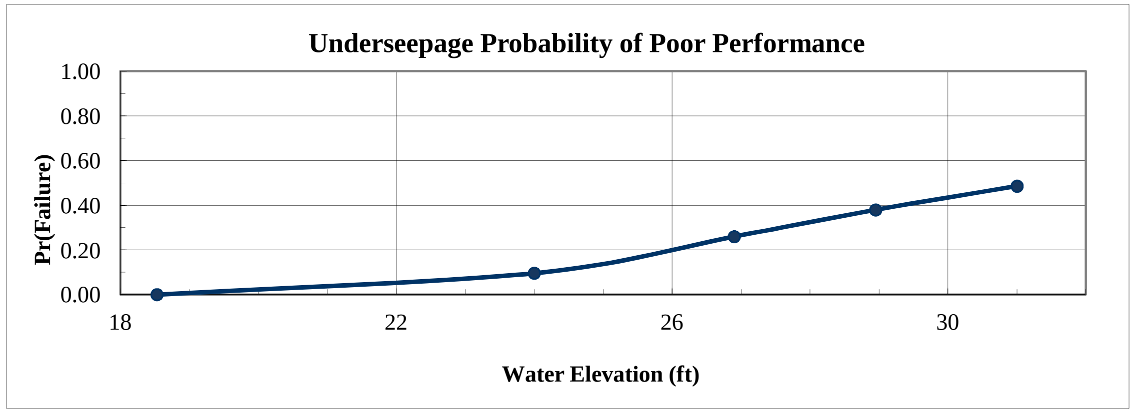
Levee Mile: 1685+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 31.00
L/S Toe Elev.: 18.53
W/S Toe Elev.: 17.80

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 12/19/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permaebility Ratio	6933	6794	98
Blanket Thickness (z)	11	6	55
Aquifer Thickness (d)	35	12	34

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	190	90	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	18.53	0.0000
Elev.	5.47	24.00	0.0961
Elev.	8.37	26.90	0.2596
Elev.	10.42	28.95	0.3790
Crest	12.47	31.00	0.4857

Crest	Rh
Head = 12.47	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	6933	11.00	35.00	189.15	1633.77	0.0183	10.65	0.97		
2	13727	11.00	35.00	189.57	2298.92	0.0136	11.12	1.01	0.052900	10.39
3	139	11.00	35.00	156.27	231.05	0.0733	6.04	0.55		
4	6933	17.00	35.00	189.45	2031.04	0.0151	10.96	0.64	0.455625	89.49
5	6933	5.00	35.00	188.14	1101.49	0.0254	9.96	1.99		
6	6933	11.00	47.00	189.36	1893.24	0.0216	10.87	0.99	0.000625	0.12
7	6933	11.00	23.00	188.71	1324.41	0.0143	10.30	0.94		
								Total	0.509150	100.00

E[I] = 0.970000
Var[I]= 0.509150
σ[I]= 0.713547
V(I) = 0.735616

E[ln I] = -0.246717
σ [ln I] = 0.657660

β =	-0.375144
F(z) =	0.514297
Pr(f) % =	48.570294

Ic= 0.80

ln(I crit) = -0.223144

Elev.	Rh
Head = 10.42	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	6933	11.00	35.00	189.15	1633.77	0.0183	8.90	0.81		
2	13727	11.00	35.00	189.57	2298.92	0.0136	9.29	0.84	0.036100	10.30
3	139	11.00	35.00	156.27	231.05	0.0733	5.04	0.46		
4	6933	17.00	35.00	189.45	2031.04	0.0151	9.16	0.54	0.313600	89.52
5	6933	5.00	35.00	188.14	1101.49	0.0254	8.32	1.66		
6	6933	11.00	47.00	189.36	1893.24	0.0216	9.08	0.83	0.000625	0.18
7	6933	11.00	23.00	188.71	1324.41	0.0143	8.61	0.78		
								Total	0.350325	100.00

E[I] = 0.810000
Var[I]= 0.350325
σ[I]= 0.591883
V(I) = 0.730719

E[ln I] = -0.424644
σ [ln I] = 0.654100

β =	-0.649204
F(z) =	0.620981
Pr(f) % =	37.901906

Ic= 0.80

ln(I crit) = -0.223144

Elev.	Rh
Head = 8.37	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	6933	11.00	35.00	189.15	1633.77	0.0183	7.15	0.65		
2	13727	11.00	35.00	189.57	2298.92	0.0136	7.46	0.68	0.024025	10.39
3	139	11.00	35.00	156.27	231.05	0.0733	4.05	0.37		
4	6933	17.00	35.00	189.45	2031.04	0.0151	7.36	0.43	0.207025	89.51
5	6933	5.00	35.00	188.14	1101.49	0.0254	6.68	1.34		
6	6933	11.00	47.00	189.36	1893.24	0.0216	7.29	0.66	0.000225	0.10
7	6933	11.00	23.00	188.71	1324.41	0.0143	6.91	0.63		
								Total	0.231275	100.00

E[I] = 0.650000
Var[I]= 0.231275
σ[I]= 0.480911
V(I) = 0.739862

E[ln I] = -0.649070
σ [ln I] = 0.660737

β =	-0.982342
F(z) =	0.740414
Pr(f) % =	25.958586

Ic= 0.80

ln(I crit) = -0.223144

Elev.	Rh
Head = 5.47	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	6933	11.00	35.00	189.15	1633.77	0.0183	4.67	0.42		
2	13727	11.00	35.00	189.57	2298.92	0.0136	4.88	0.44	0.010000	10.30
3	139	11.00	35.00	156.27	231.05	0.0733	2.65	0.24		
4	6933	17.00	35.00	189.45	2031.04	0.0151	4.81	0.28	0.087025	89.60
5	6933	5.00	35.00	188.14	1101.49	0.0254	4.37	0.87		
6	6933	11.00	47.00	189.36	1893.24	0.0216	4.77	0.43	0.000100	0.10
7	6933	11.00	23.00	188.71	1324.41	0.0143	4.52	0.41		
								Total	0.097125	100.00

E[I] = 0.420000
Var[I]= 0.097125
σ[I]= 0.311649
V(I) = 0.742021

E[ln I] = -1.086820
σ [ln I] = 0.662298

β =	-1.640983
F(z) =	0.903893
Pr(f) % =	9.610659

Ic= 0.80

ln(I crit) = -0.223144

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR3

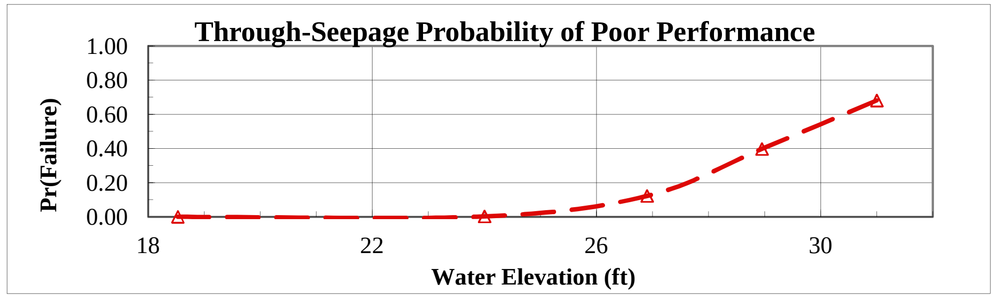
Levee Mile: 1685+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 31.00
L/S Toe Elev.: 18.53
W/S Toe Elev.: 17.80

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 12/19/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	2	0.2	10.00
Initial Porosity (n)	0.25	0.03	10.00
Initial Permeability (Ko)	8.00E-08	2.40E-08	30.00

Pr(f)=0
NO



Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	18.53	0.0000
Elev.	5.47	24.00	0.002576
Elev.	8.37	26.90	0.122242
Elev.	10.42	28.95	0.397071
Crest	12.47	31.00	0.680891

Crest	**Head =**	12.47	**Horizontal Gradient (Ix) =**	1.370

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	2.00	0.25	8.00E-08	1.27	0.93		
2	1.80	0.25	8.00E-08	1.15	0.84	0.008658	26.44
3	2.20	0.25	8.00E-08	1.40	1.02		
4	2.00	0.23	8.00E-08	1.21	0.88	0.002170	6.63
5	2.00	0.28	8.00E-08	1.34	0.98		
6	2.00	0.25	5.60E-08	1.52	1.11	0.021913	66.93
7	2.00	0.25	1.04E-07	1.12	0.82		
E[FS] =	0.930505		E[ln FS] =	-0.090586	Total	0.032741	100.00
Var[FS]=	0.032741						
σ[FS]=	0.180945		σ[ln FS]=	0.192658		**β =**	-0.470191
V(FS) =	0.194459					**F(z) =**	0.680891
FS req'd =	1.00		ln(FS req'd) =	0.000000		**Pr(f) % =**	68.089086

Elev.	**Head =**	10.42	**Horizontal Gradient (Ix) =**	1.190

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	2.00	0.25	8.00E-08	1.27	1.07		
2	1.80	0.25	8.00E-08	1.15	0.96	0.011476	26.44
3	2.20	0.25	8.00E-08	1.40	1.18		
4	2.00	0.23	8.00E-08	1.21	1.02	0.002876	6.63
5	2.00	0.28	8.00E-08	1.34	1.12		
6	2.00	0.25	5.60E-08	1.52	1.28	0.029043	66.93
7	2.00	0.25	1.04E-07	1.12	0.94		
E[FS] =	1.071254		E[ln FS] =	0.050271	Total	0.043395	100.00
Var[FS]=	0.043395						
σ[FS]=	0.208315		σ[ln FS]=	0.192658		**β =**	0.260937
V(FS) =	0.194459					**F(z) =**	0.397071
FS req'd =	1.00		ln(FS req'd) =	0.000000		**Pr(f) % =**	39.707066

Elev.	**Head =**	8.37	**Horizontal Gradient (Ix) =**	1.000

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	2.00	0.25	8.00E-08	1.27	1.27		
2	1.80	0.25	8.00E-08	1.15	1.15	0.016251	26.44
3	2.20	0.25	8.00E-08	1.40	1.40		
4	2.00	0.23	8.00E-08	1.21	1.21	0.004073	6.63
5	2.00	0.28	8.00E-08	1.34	1.34		
6	2.00	0.25	5.60E-08	1.52	1.52	0.041128	66.93
7	2.00	0.25	1.04E-07	1.12	1.12		
E[FS] =	1.274792		E[ln FS] =	0.224225	Total	0.061452	100.00
Var[FS]=	0.061452						
σ[FS]=	0.247895		σ[ln FS]=	0.192658		**β =**	1.163851
V(FS) =	0.194459					**F(z) =**	0.122242
FS req'd =	1.00		ln(FS req'd) =	0.000000		**Pr(f) % =**	12.224225

Elev.	**Head =**	5.47	**Horizontal Gradient (Ix) =**	0.730

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	2.00	0.25	8.00E-08	1.27	1.75		
2	1.80	0.25	8.00E-08	1.15	1.57	0.030495	26.44
3	2.20	0.25	8.00E-08	1.40	1.92		
4	2.00	0.23	8.00E-08	1.21	1.66	0.007643	6.63
5	2.00	0.28	8.00E-08	1.34	1.83		
6	2.00	0.25	5.60E-08	1.52	2.09	0.077178	66.93
7	2.00	0.25	1.04E-07	1.12	1.53		
E[FS] =	1.746291		E[ln FS] =	0.538936	Total	0.115316	100.00
Var[FS]=	0.115316						
σ[FS]=	0.339582		σ[ln FS]=	0.192658		**β =**	2.797374
V(FS) =	0.194459					**F(z) =**	0.002576
FS req'd =	1.00		ln(FS req'd) =	0.000000		**Pr(f) % =**	0.257599

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

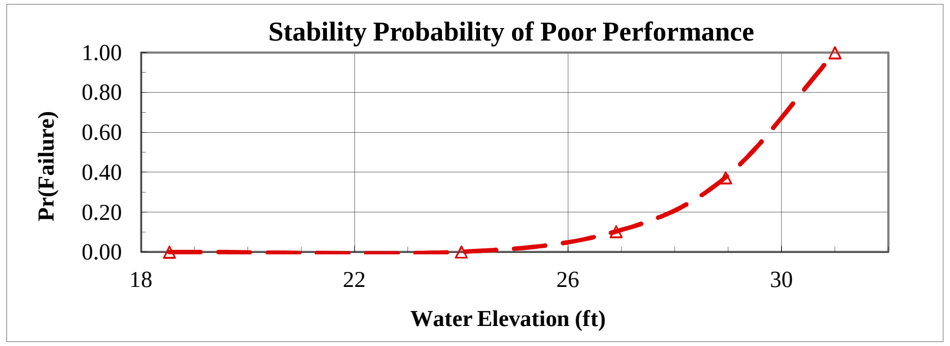
Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR3

Levee Mile: 1685+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 31.00
L/S Toe Elev.: 18.53
W/S Toe Elev.: 17.80

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 12/19/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	30	4	13.00
Levee Cohesion	50	20	40.00
Levee γ	125	9	7.00
Foundation Φ	28	4	13.00
Foundation Cohesion	100	40	40.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	18.53	0.0000
Elev.	5.47	24.00	0.000272
Elev.	8.37	26.90	0.102531
Elev.	10.42	28.95	0.372477
Crest	12.47	31.00	0.999333

Crest	**Head =**	12.47	**Pr(f)=0**	NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	30	50	125	28	100	0.77		
2	26	50	125	28	100	0.73	0.002162	53.37
3	34	50	125	28	100	0.82		
4	30	30	125	28	100	0.73	0.000992	24.49
5	30	70	125	28	100	0.80		
6	30	50	116	28	100	0.76	0.000121	2.99
7	30	50	134	28	100	0.78		
8	30	50	125	24	100	0.76	0.000100	2.47
9	30	50	125	32	100	0.78		
10	30	50	125	28	60	0.74	0.000676	16.69
11	30	50	125	28	140	0.79		

E[FS] = 0.770000 E[ln FS] = -0.264770 Total 0.004052 100.00
Var[FS]= 0.004052
σ[FS]= 0.063651 σ[ln FS]= 0.082523 **β =** -3.208419
V(FS) = 0.082664 **F(z) =** 0.999333
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 99.933267

Elev.	**Head =**	10.42	**Pr(f)=0**	NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	30	50	125	28	100	1.03		
2	26	50	125	28	100	0.98	0.004900	70.47
3	34	50	125	28	100	1.12		
4	30	30	125	28	100	1.01	0.000441	6.34
5	30	70	125	28	100	1.05		
6	30	50	116	28	100	1.06	0.000081	1.16
7	30	50	134	28	100	1.04		
8	30	50	125	24	100	1.01	0.000306	4.40
9	30	50	125	32	100	1.05		
10	30	50	125	28	60	0.99	0.001225	17.62
11	30	50	125	28	140	1.06		

E[FS] = 1.030000 E[ln FS] = 0.026292 Total 0.006953 100.00
Var[FS]= 0.006953
σ[FS]= 0.083386 σ[ln FS]= 0.080825 **β =** 0.325300
V(FS) = 0.080957 **F(z) =** 0.372477
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 37.247706

Elev.	**Head =**	8.37	**Pr(f)=0**	NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	30	50	125	28	100	1.20		
2	26	50	125	28	100	1.12	0.007310	26.99
3	34	50	125	28	100	1.29		
4	30	30	125	28	100	1.16	0.001122	4.14
5	30	70	125	28	100	1.23		
6	30	50	116	28	100	1.19	0.000004	0.01
7	30	50	134	28	100	1.20		
8	30	50	125	24	100	1.20	0.000020	0.07
9	30	50	125	32	100	1.19		
10	30	50	125	28	60	0.93	0.018632	68.78
11	30	50	125	28	140	1.21		

E[FS] = 1.200000 E[ln FS] = 0.173003 Total 0.027089 100.00
Var[FS]= 0.027089
σ[FS]= 0.164587 σ[ln FS]= 0.136518 **β =** 1.267258
V(FS) = 0.137156 **F(z) =** 0.102531
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 10.253149

Elev.	**Head =**	5.47	**Pr(f)=0**	NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	30	50	125	28	100	1.35		
2	26	50	125	28	100	1.30	0.002916	21.69
3	34	50	125	28	100	1.40		
4	30	30	125	28	100	1.30	0.001600	11.90
5	30	70	125	28	100	1.38		
6	30	50	116	28	100	1.35	0.000016	0.12
7	30	50	134	28	100	1.34		
8	30	50	125	24	100	1.32	0.000900	6.70
9	30	50	125	32	100	1.38		
10	30	50	125	28	60	1.31	0.008010	59.59
11	30	50	125	28	140	1.49		

E[FS] = 1.350000 E[ln FS] = 0.296430 Total 0.013442 100.00
Var[FS]= 0.013442
σ[FS]= 0.115941 σ[ln FS]= 0.085724 **β =** 3.457950
V(FS) = 0.085882 **F(z) =** 0.000272
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.027215

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

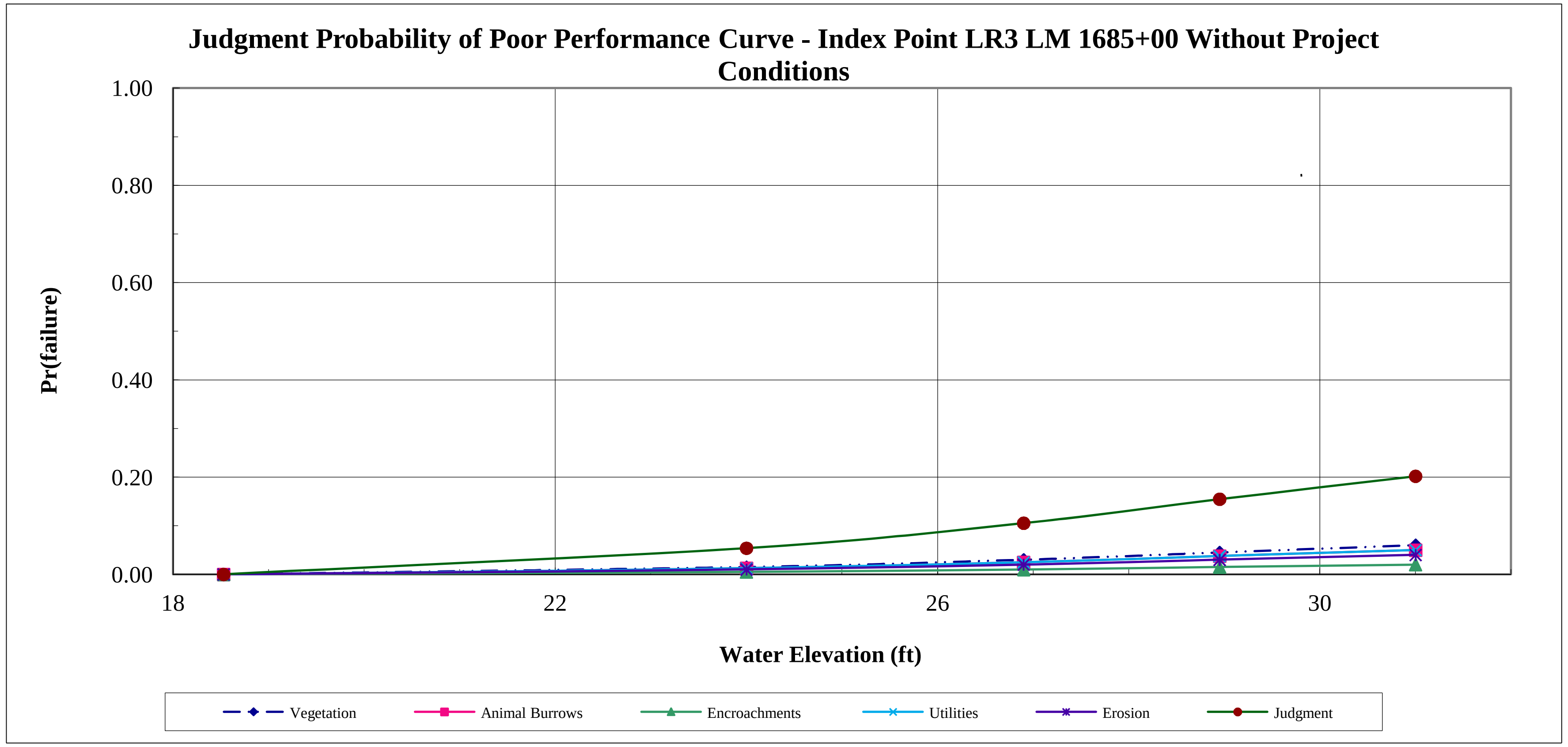
Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR3

Levee Mile: 1685+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 31.00
L/S Toe Elev.: 18.53
W/S Toe Elev.: 17.80

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 12/19/2012

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
18.53	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
24.00	0.0150	0.9850	0.0125	0.9875	0.0050	0.9950	0.0125	0.9875	0.0100	0.9900	0.0538	0.9462
26.90	0.0300	0.9700	0.0250	0.9750	0.0100	0.9900	0.0250	0.9750	0.0200	0.9800	0.1054	0.8946
28.95	0.0450	0.9550	0.0375	0.9625	0.0150	0.9850	0.0375	0.9625	0.0300	0.9700	0.1547	0.8453
31.00	0.0600	0.9400	0.0500	0.9500	0.0200	0.9800	0.0500	0.9500	0.0400	0.9600	0.2019	0.7981


Judgment Probability of Poor Performance Curve - Index Point LR3 LM 1685+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

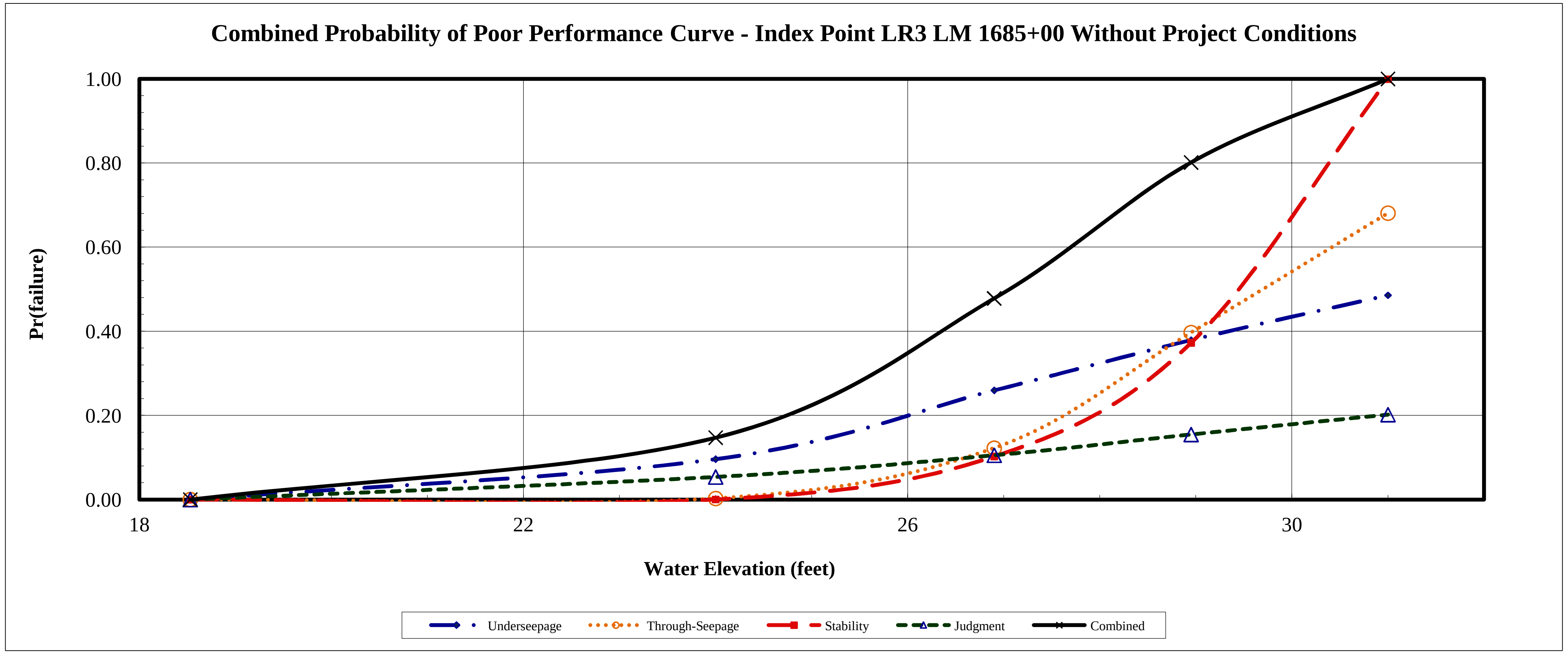
Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR3

Levee Mile: 1685+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 31.00
L/S Toe Elev.: 18.53
W/S Toe Elev.: 17.80

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 12/19/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
18.53	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
24.00	0.0961	0.9039	0.0026	0.9974	0.0003	0.9997	0.0538	0.9462	0.1472	0.8528
26.90	0.2596	0.7404	0.1222	0.8778	0.1025	0.8975	0.1054	0.8946	0.4782	0.5218
28.95	0.3790	0.6210	0.3971	0.6029	0.3725	0.6275	0.1547	0.8453	0.8014	0.1986
31.00	0.4857	0.5143	0.6809	0.3191	0.9993	0.0007	0.2019	0.7981	0.9999	0.0001


Combined Probability of Poor Performance Curve - Index Point LR3 LM 1685+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: Right Bank San Joaquin River
Basin and Reach: Index Point LR4

Levee Mile: STA 1815+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 33.90
L/S Toe Elev.: 18.60
W/S Toe Elev.: 19.40

Analysis By: G. Johnson
Checked By: M. Perlea 12/13/2012
Date: 12/13/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket		Aquifer Material		Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
											Material	Kb (ft/day)	Material	Kf (ft/day)					
WR0017_098C	28	23	3	154	13	20	33	8	324	24	CL	0.007	SP-SM	14	2000	3200	1095	3377778	34
WR0017_099C	20					38					CL	0.007	SP-SM	14	2000				
WR0017_100C	22					32					CL	0.0007	SP-SM	2.8	4000				
WR0017_101C	24					38					CL	0.0007	SP-SM	2.8	4000				
WR0017_103C	22					36					CL	0.0007	SP-SM	2.8	4000				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WR0017_098C	CL	28	0.007				28	SP-SM	20	14							14
WR0017_099C	CL	20	0.007				20	SP-SM	38	14							14
WR0017_100C	CL	22	0.0007				22	SP-SM	32	2.8							2.8
WR0017_101C	CL	24	0.0007				24	SP-SM	38	2.8							2.8
WR0017_103C	CL	22	0.0007				22	SP-SM	36	2.8							2.8

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: Right Bank San Joaquin River
River Section: Index Point LR4

Levee Mile: STA 1815+00
River Mile: XX.XX
Analysis Case Without Project Conditions

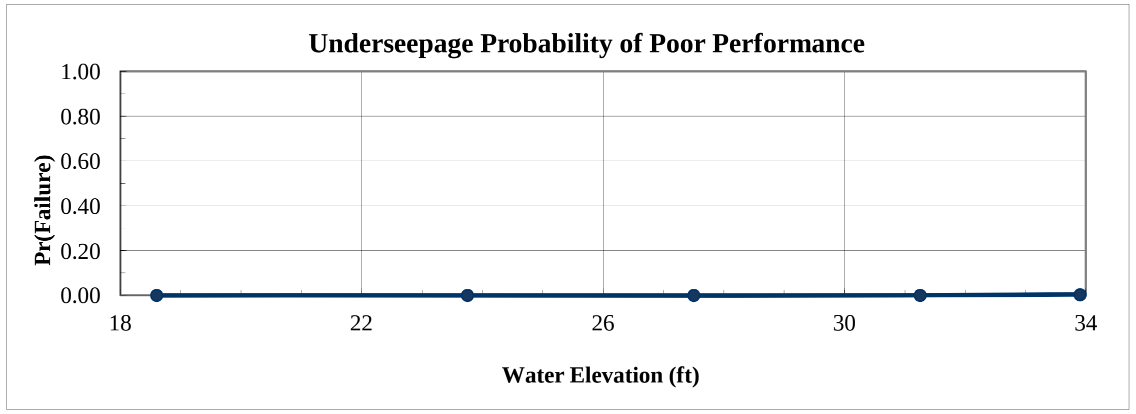
Crest Elev.: 33.90
L/S Toe Elev.: 18.60
W/S Toe Elev.: 19.40

Analysis By: G. Johnson
Checked By: M. Perlea 12/13/2012
Date: 12/13/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permaebility Ratio	3200	1095	34
Blanket Thickness (z)	23	3	13
Aquifer Thickness (d)	33	8	24

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	153	110	∞	112

Underseepage Probability of Poor Performance



Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	18.60	0.0000
Elev. 23.75	5.15	23.75	0.0000
Elev. 27.5	8.90	27.50	0.0000
200 yr.	12.65	31.25	0.0000
Crest	15.30	33.90	0.0030

Crest	**Rh**
Head = 15.30	

Run	**Kf/Kb**	**z**	**d**	**x1**	**x3**	**$**	**hx**	**I**	**Variance Component**	**% Variance**
1 (Mean)	3200	23.00	33.00	152.51	1558.46	0.0181	13.09	0.57		
2	4295	23.00	33.00	152.63	1805.52	0.0160	13.36	0.58	0.000225	4.31
3	2105	23.00	33.00	152.26	1264.00	0.0216	12.67	0.55		
4	3200	26.00	33.00	152.57	1656.99	0.0172	13.21	0.51	0.004900	93.78
5	3200	20.00	33.00	152.44	1453.27	0.0192	12.96	0.65		
6	3200	23.00	41.00	152.61	1737.12	0.0205	13.29	0.58	0.000100	1.91
7	3200	23.00	25.00	152.35	1356.47	0.0154	12.82	0.56		
								Total	0.005225	100.00

E[I] = 0.570000
Var[I]= 0.005225
σ[I]= 0.072284
V(I) = 0.126814
E[ln I] = -0.570096
σ [ln I] = 0.126309
Ic= 0.80
ln(I crit) = -0.223144

β =	-4.513507
F(z) =	0.996992
Pr(f) % =	0.300847

200 yr.	**Rh**
Head = 12.65	

Run	**Kf/Kb**	**z**	**d**	**x1**	**x3**	**$**	**hx**	**I**	**Variance Component**	**% Variance**
1 (Mean)	3200	23.00	33.00	152.51	1558.46	0.0181	10.83	0.47		
2	4295	23.00	33.00	152.63	1805.52	0.0160	11.04	0.48	0.000100	2.63
3	2105	23.00	33.00	152.26	1264.00	0.0216	10.48	0.46		
4	3200	26.00	33.00	152.57	1656.99	0.0172	10.92	0.42	0.003600	94.74
5	3200	20.00	33.00	152.44	1453.27	0.0192	10.72	0.54		
6	3200	23.00	41.00	152.61	1737.12	0.0205	10.99	0.48	0.000100	2.63
7	3200	23.00	25.00	152.35	1356.47	0.0154	10.60	0.46		
								Total	0.003800	100.00

E[I] = 0.470000
Var[I]= 0.003800
σ[I]= 0.061644
V(I) = 0.131158
E[ln I] = -0.763551
σ [ln I] = 0.130599
Ic= 0.80
ln(I crit) = -0.223144

β =	-5.846532
F(z) =	0.999982
Pr(f) % =	0.001752

Elev. 27.5	**Rh**
Head = 8.90	

Run	**Kf/Kb**	**z**	**d**	**x1**	**x3**	**$**	**hx**	**I**	**Variance Component**	**% Variance**
1 (Mean)	3200	23.00	33.00	152.51	1558.46	0.0181	7.62	0.33		
2	4295	23.00	33.00	152.63	1805.52	0.0160	7.77	0.34	0.000100	5.56
3	2105	23.00	33.00	152.26	1264.00	0.0216	7.37	0.32		
4	3200	26.00	33.00	152.57	1656.99	0.0172	7.68	0.30	0.001600	88.89
5	3200	20.00	33.00	152.44	1453.27	0.0192	7.54	0.38		
6	3200	23.00	41.00	152.61	1737.12	0.0205	7.73	0.34	0.000100	5.56
7	3200	23.00	25.00	152.35	1356.47	0.0154	7.46	0.32		
								Total	0.001800	100.00

E[I] = 0.330000
Var[I]= 0.001800
σ[I]= 0.042426
V(I) = 0.128565
E[ln I] = -1.116860
σ [ln I] = 0.128038
Ic= 0.80
ln(I crit) = -0.223144

β =	-8.722854
F(z) =	1.000000
Pr(f) % =	0.000000

Elev. 23.75	**Rh**
Head = 5.15	

Run	**Kf/Kb**	**z**	**d**	**x1**	**x3**	**$**	**hx**	**I**	**Variance Component**	**% Variance**
1 (Mean)	3200	23.00	33.00	152.51	1558.46	0.0181	4.41	0.19		
2	4295	23.00	33.00	152.63	1805.52	0.0160	4.50	0.20	0.000025	3.85
3	2105	23.00	33.00	152.26	1264.00	0.0216	4.27	0.19		
4	3200	26.00	33.00	152.57	1656.99	0.0172	4.45	0.17	0.000625	96.15
5	3200	20.00	33.00	152.44	1453.27	0.0192	4.36	0.22		
6	3200	23.00	41.00	152.61	1737.12	0.0205	4.47	0.19	0.000000	0.00
7	3200	23.00	25.00	152.35	1356.47	0.0154	4.32	0.19		
								Total	0.000650	100.00

E[I] = 0.190000
Var[I]= 0.000650
σ[I]= 0.025495
V(I) = 0.134185
E[ln I] = -1.669654
σ [ln I] = 0.133587
Ic= 0.80
ln(I crit) = -0.223144

β =	-12.498670
F(z) =	1.000000
Pr(f) % =	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Lower San Joaquin
Study Area: Right Bank San Joaquin River
River Section: Index Point LR4

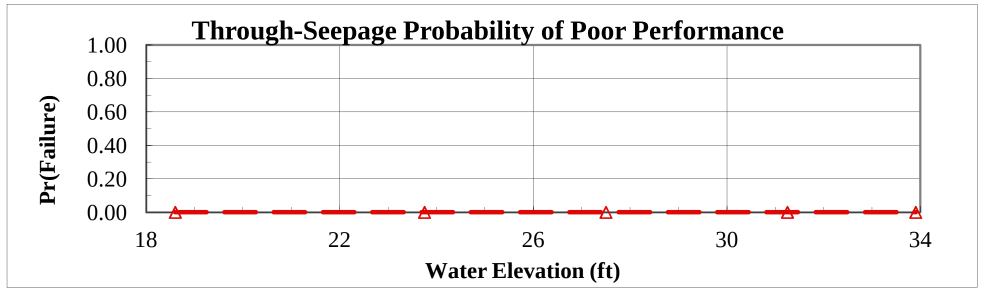
Levee Mile: STA 1815+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 33.90
L/S Toe Elev.: 18.60
W/S Toe Elev.: 19.40

Analysis By: G. Johnson
Checked By: M. Perlea 12/13/2012
Date: 12/13/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	5	0.5	10.00
Initial Porosity (n)	0.5	0.05	10.00
Initial Permeability (Ko)	1.00E-08	3.00E-09	30.00

Pr(f)=0
NO



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	18.60	0.0000
Elev. 23.75	5.15	23.75	0.000000
Elev. 27.5	8.90	27.50	0.000000
200 yr.	12.65	31.25	0.000000
Crest	15.30	33.90	0.000000

Crest	**Head =**	15.30	**Horizontal Gradient (Ix) =**	0.590

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	5.00	0.50	1.00E-08	12.75	21.61		
2	4.50	0.50	1.00E-08	11.47	19.45	4.668473	26.44
3	5.50	0.50	1.00E-08	14.02	23.77		
4	5.00	0.45	1.00E-08	12.09	20.50	1.170051	6.63
5	5.00	0.55	1.00E-08	13.37	22.66		
6	5.00	0.50	7.00E-09	15.24	25.82	11.815032	66.93
7	5.00	0.50	1.30E-08	11.18	18.95		
E[FS] =	21.606649		E[ln FS] =	3.054443	Total	17.653555	100.00

Var[FS]=	17.653555				
σ[FS]=	4.201613	σ[ln FS]=	0.192658	**β =**	15.854249
V(FS) =	0.194459			**F(z) =**	0.000000
FS req'd =	1.00	ln(FS req'd) =	0.000000	**Pr(f) % =**	0.000000

200 yr.	**Head =**	12.65	**Horizontal Gradient (Ix) =**	0.530

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	5.00	0.50	1.00E-08	12.75	24.05		
2	4.50	0.50	1.00E-08	11.47	21.65	5.785316	26.44
3	5.50	0.50	1.00E-08	14.02	26.46		
4	5.00	0.45	1.00E-08	12.09	22.82	1.449963	6.63
5	5.00	0.55	1.00E-08	13.37	25.23		
6	5.00	0.50	7.00E-09	15.24	28.75	14.641554	66.93
7	5.00	0.50	1.30E-08	11.18	21.10		
E[FS] =	24.052685		E[ln FS] =	3.161688	Total	21.876834	100.00

Var[FS]=	21.876834				
σ[FS]=	4.677268	σ[ln FS]=	0.192658	**β =**	16.410913
V(FS) =	0.194459			**F(z) =**	0.000000
FS req'd =	1.00	ln(FS req'd) =	0.000000	**Pr(f) % =**	0.000000

Elev. 27.5	**Head =**	8.90	**Horizontal Gradient (Ix) =**	0.410

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	5.00	0.50	1.00E-08	12.75	31.09		
2	4.50	0.50	1.00E-08	11.47	27.98	9.667432	26.44
3	5.50	0.50	1.00E-08	14.02	34.20		
4	5.00	0.45	1.00E-08	12.09	29.50	2.422931	6.63
5	5.00	0.55	1.00E-08	13.37	32.61		
6	5.00	0.50	7.00E-09	15.24	37.16	24.466464	66.93
7	5.00	0.50	1.30E-08	11.18	27.27		
E[FS] =	31.092495		E[ln FS] =	3.418408	Total	36.556827	100.00

Var[FS]=	36.556827				
σ[FS]=	6.046224	σ[ln FS]=	0.192658	**β =**	17.743431
V(FS) =	0.194459			**F(z) =**	0.000000
FS req'd =	1.00	ln(FS req'd) =	0.000000	**Pr(f) % =**	0.000000

Elev. 23.75	**Head =**	5.15	**Horizontal Gradient (Ix) =**	0.190

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	5.00	0.50	1.00E-08	12.75	67.09		
2	4.50	0.50	1.00E-08	11.47	60.38	45.016492	26.44
3	5.50	0.50	1.00E-08	14.02	73.80		
4	5.00	0.45	1.00E-08	12.09	63.65	11.282400	6.63
5	5.00	0.55	1.00E-08	13.37	70.37		
6	5.00	0.50	7.00E-09	15.24	80.19	113.928328	66.93
7	5.00	0.50	1.30E-08	11.18	58.85		
E[FS] =	67.094331		E[ln FS] =	4.187541	Total	170.227221	100.00

Var[FS]=	170.227221				
σ[FS]=	13.047115	σ[ln FS]=	0.192658	**β =**	21.735658
V(FS) =	0.194459			**F(z) =**	0.000000
FS req'd =	1.00	ln(FS req'd) =	0.000000	**Pr(f) % =**	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

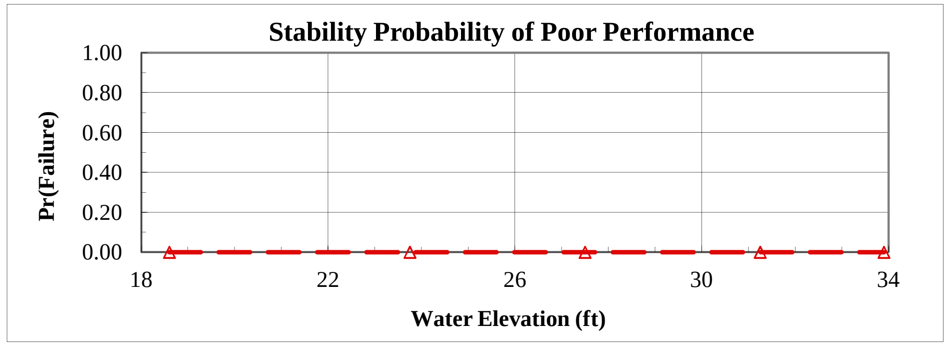
Project: Lower San Joaquin
Study Area: Right Bank San Joaquin River
River Section: Index Point LR4

Levee Mile: STA 1815+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 33.90
L/S Toe Elev.: 18.60
W/S Toe Elev.: 19.40

Analysis By: G. Johnson
Checked By: M. Perlea 12/13/2012
Date: 12/13/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	30	4	13.00
Levee Cohesion	50	20	40.00
Levee γ	125	9	7.00
Foundation Φ	28	4	13.00
Foundation Cohesion	100	40	40.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	18.60	0.0000
Elev. 23.75	5.15	23.75	0.000000
Elev. 27.5	8.90	27.50	0.000000
200 yr.	12.65	31.25	0.000000
Crest	15.30	33.90	0.000090

Crest	**Head =**	15.30	**Pr(f)=0**	NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	30	50	125	28	100	1.63		
2	26	50	125	28	100	1.57	0.003906	8.87
3	34	50	125	28	100	1.70		
4	30	30	125	28	100	1.60	0.001056	2.40
5	30	70	125	28	100	1.66		
6	30	50	116	28	100	1.65	0.000144	0.33
7	30	50	134	28	100	1.62		
8	30	50	125	24	100	1.50	0.018496	41.99
9	30	50	125	32	100	1.77		
10	30	50	125	28	60	1.49	0.020449	46.42
11	30	50	125	28	140	1.77		

E[FS] = 1.630000 E[ln FS] = 0.480358 Total 0.044052 100.00
Var[FS]= 0.044052
σ[FS]= 0.209884 σ[ln FS]= 0.128235
V(FS) = 0.128763
FS req'd = 1.00 ln(FS req'd) = 0.000000

β =	3.745934
F(z) =	0.000090
Pr(f) % =	0.008986

200 yr.	**Head =**	12.65	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	30	50	125	28	100	1.78		
2	26	50	125	28	100			
3	34	50	125	28	100			
4	30	30	125	28	100			
5	30	70	125	28	100			
6	30	50	116	28	100			
7	30	50	134	28	100			
8	30	50	125	24	100			
9	30	50	125	32	100			
10	30	50	125	28	60			
11	30	50	125	28	140			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]=
V(FS) =
FS req'd = 1.00 ln(FS req'd) = 0.000000

β =	
F(z) =	
Pr(f) % =	0.000000

Elev. 27.5	**Head =**	8.90	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	30	50	125	28	100	1.98		
2	26	50	125	28	100			
3	34	50	125	28	100			
4	30	30	125	28	100			
5	30	70	125	28	100			
6	30	50	116	28	100			
7	30	50	134	28	100			
8	30	50	125	24	100			
9	30	50	125	32	100			
10	30	50	125	28	60			
11	30	50	125	28	140			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]=
V(FS) =
FS req'd = 1.00 ln(FS req'd) = 0.000000

β =	
F(z) =	
Pr(f) % =	0.000000

Elev. 23.75	**Head =**	5.15	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	30	50	125	28	100	2.14		
2	26	50	125	28	100			
3	34	50	125	28	100			
4	30	30	125	28	100			
5	30	70	125	28	100			
6	30	50	116	28	100			
7	30	50	134	28	100			
8	30	50	125	24	100			
9	30	50	125	32	100			
10	30	50	125	28	60			
11	30	50	125	28	140			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]=
V(FS) =
FS req'd = 1.00 ln(FS req'd) = 0.000000

β =	
F(z) =	
Pr(f) % =	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

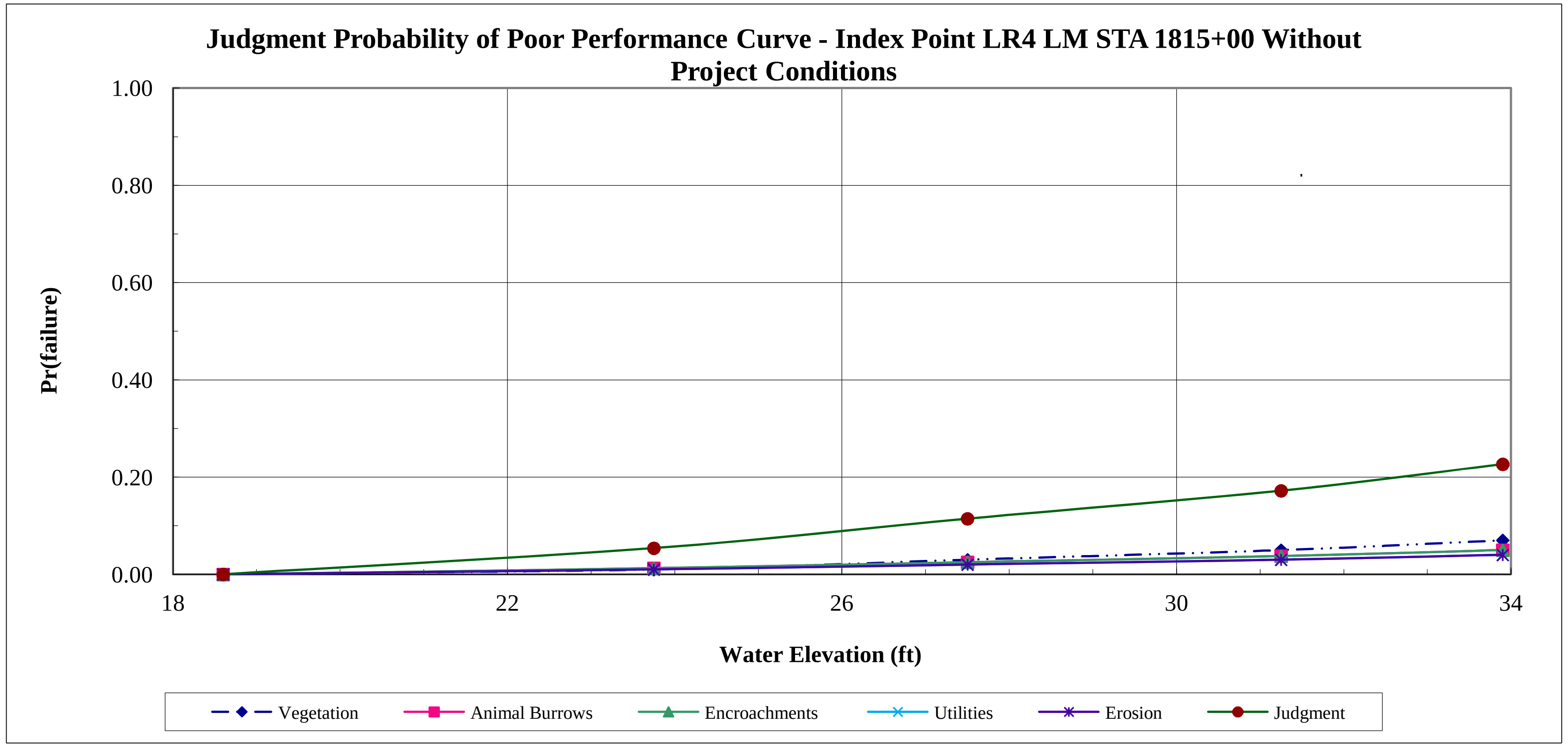
Project: Lower San Joaquin
Study Area: Right Bank San Joaquin River
River Section: Index Point LR4

Levee Mile: STA 1815+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 33.90
L/S Toe Elev.: 18.60
W/S Toe Elev.: 19.40

Analysis By: G. Johnson
Checked By: M. Perlea 12/13/2012
Date: 12/13/2012

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
18.60	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
23.75	0.0100	0.9900	0.0125	0.9875	0.0125	0.9875	0.0100	0.9900	0.0100	0.9900	0.0538	0.9462
27.50	0.0300	0.9700	0.0250	0.9750	0.0250	0.9750	0.0200	0.9800	0.0200	0.9800	0.1144	0.8856
31.25	0.0500	0.9500	0.0375	0.9625	0.0375	0.9625	0.0300	0.9700	0.0300	0.9700	0.1719	0.8281
33.90	0.0700	0.9300	0.0500	0.9500	0.0500	0.9500	0.0400	0.9600	0.0400	0.9600	0.2265	0.7735


Judgment Probability of Poor Performance Curve - Index Point LR4 LM STA 1815+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

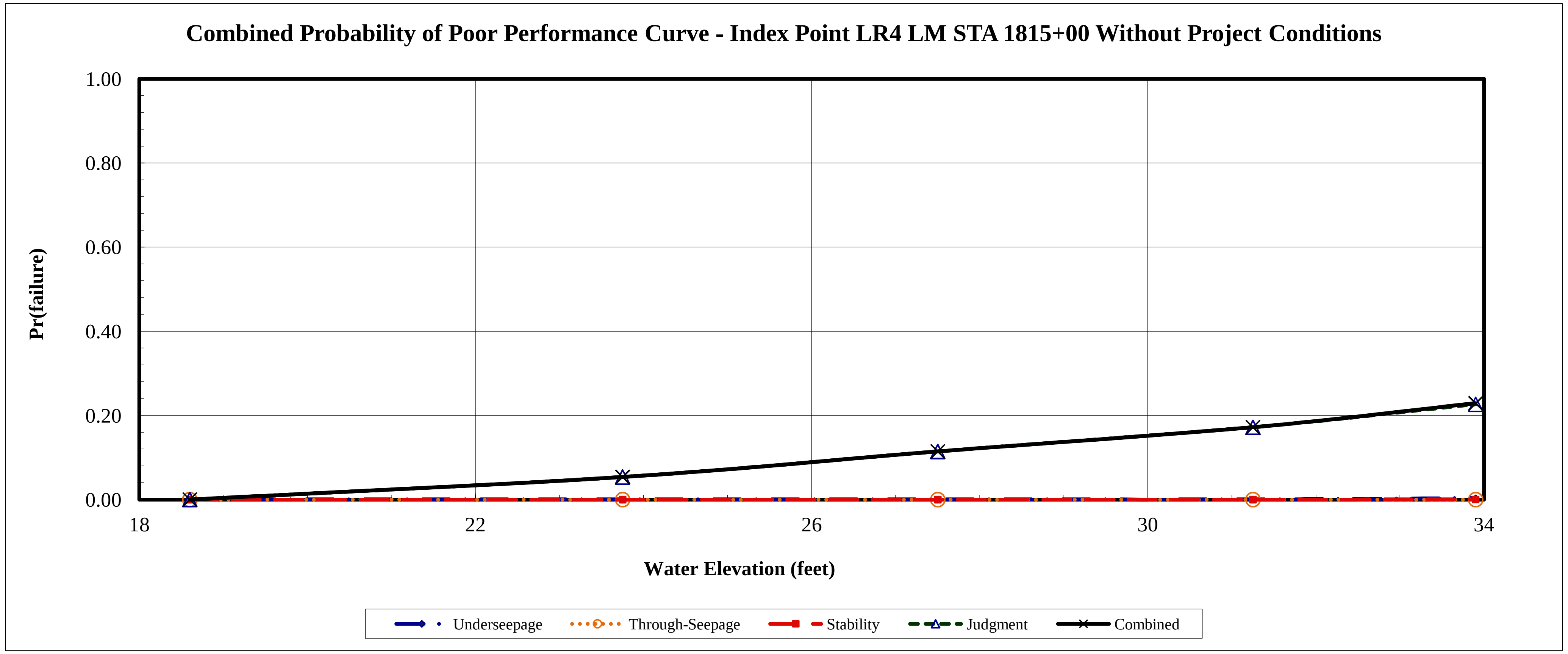
Project: Lower San Joaquin
Study Area: Right Bank San Joaquin River
River Section: Index Point LR4

Levee Mile: STA 1815+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 33.90
L/S Toe Elev.: 18.60
W/S Toe Elev.: 19.40

Analysis By: G. Johnson
Checked By: M. Perlea 12/13/2012
Date: 12/13/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
18.60	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
23.75	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0538	0.9462	0.0538	0.9462
27.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1144	0.8856	0.1144	0.8856
31.25	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1719	0.8281	0.1719	0.8281
33.90	0.0030	0.9970	0.0000	1.0000	0.0001	0.9999	0.2265	0.7735	0.2289	0.7711



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: Left Bank French Camp Slough
Basin and Reach: Index Point FL1

Levee Mile: STA 1049+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 21.40
L/S Toe Elev.: 9.36
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket		Aquifer Material		Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
											Material	Kb (ft/day)	Material	Kf (ft/day)					
WR0017_004C	10	10	1	29	10	6	9	6	35	67	CL	0.0007	SC	0.28	400	400	0	44444	0
WR0017_005C	8					4					CL	0.0007	SC	0.28	400				
WR0017_007B	10					6					CL	0.0007	SC	0.28	400				
WR0017_010C	10					10					CL	0.0007	SC	0.28	400				
WR0017_011C	12					18					CL	0.0007	SC	0.28	400				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WR0017_004C	CL	10	0.0007				10	SC	6	0.28							0.28
WR0017_005C	CL	8	0.0007				8	SC	4	0.28							0.28
WR0017_007B	CL	10	0.0007				10	SC	6	0.28							0.28
WR0017_010C	CL	10	0.0007				10	SC	10	0.28							0.28
WR0017_011C	CL	12	0.0007				12	SC	18	0.28							0.28

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: Left Bank French Camp Slough
River Section: Index Point FL1

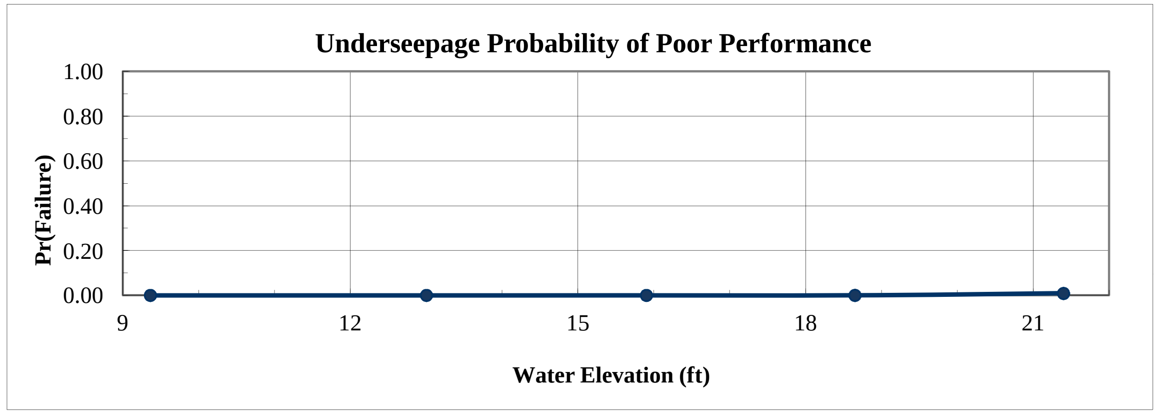
Levee Mile: STA 1049+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 21.40
L/S Toe Elev.: 9.36
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permeability Ratio	400	0	0
Blanket Thickness (z)	10	1	10
Aquifer Thickness (d)	9	6	67

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	175	103	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	9.36	0.0000
Elev. 13.0	3.64	13.00	0.0000
200 year	6.54	15.90	0.0000
Elev. 18.65	9.29	18.65	0.0000
Crest	12.04	21.40	0.0087

Crest	Rh
Head = 12.04	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	400	10.00	9.00	137.94	189.74	0.0209	5.30	0.53		
2	400	10.00	9.00	137.94	189.74	0.0209	5.30	0.53	0.000000	0.00
3	400	10.00	9.00	137.94	189.74	0.0209	5.30	0.53		
4	400	11.00	9.00	140.52	199.00	0.0203	5.41	0.49	0.002025	21.89
5	400	9.00	9.00	134.94	180.00	0.0215	5.19	0.58		
6	400	10.00	15.00	150.26	244.95	0.0301	5.92	0.59	0.007225	78.11
7	400	10.00	3.00	100.92	109.54	0.0096	4.21	0.42		
								Total	0.009250	100.00

E[I] = 0.530000
Var[I]= 0.009250
σ[I]= 0.096177
V(I) = 0.181466
E[ln I] = -0.651078
σ [ln I] = 0.179998
β = -3.617139
F(z) = 0.991283
Pr(f) % = 0.871666
Ic= 0.80
ln(I crit) = -0.223144

Elev. 18.65	Rh
Head = 9.29	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	400	10.00	9.00	137.94	189.74	0.0209	4.09	0.41		
2	400	10.00	9.00	137.94	189.74	0.0209	4.09	0.41	0.000000	0.00
3	400	10.00	9.00	137.94	189.74	0.0209	4.09	0.41		
4	400	11.00	9.00	140.52	199.00	0.0203	4.18	0.38	0.000900	17.56
5	400	9.00	9.00	134.94	180.00	0.0215	4.00	0.44		
6	400	10.00	15.00	150.26	244.95	0.0301	4.57	0.46	0.004225	82.44
7	400	10.00	3.00	100.92	109.54	0.0096	3.25	0.33		
								Total	0.005125	100.00

E[I] = 0.410000
Var[I]= 0.005125
σ[I]= 0.071589
V(I) = 0.174608
E[ln I] = -0.906614
σ [ln I] = 0.173298
β = -5.231525
F(z) = 0.999960
Pr(f) % = 0.004008
Ic= 0.80
ln(I crit) = -0.223144

200 year	Rh
Head = 6.54	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	400	10.00	9.00	137.94	189.74	0.0209	2.88	0.29		
2	400	10.00	9.00	137.94	189.74	0.0209	2.88	0.29	0.000000	0.00
3	400	10.00	9.00	137.94	189.74	0.0209	2.88	0.29		
4	400	11.00	9.00	140.52	199.00	0.0203	2.94	0.27	0.000400	16.49
5	400	9.00	9.00	134.94	180.00	0.0215	2.82	0.31		
6	400	10.00	15.00	150.26	244.95	0.0301	3.22	0.32	0.002025	83.51
7	400	10.00	3.00	100.92	109.54	0.0096	2.29	0.23		
								Total	0.002425	100.00

E[I] = 0.290000
Var[I]= 0.002425
σ[I]= 0.049244
V(I) = 0.169808
E[ln I] = -1.252088
σ [ln I] = 0.168603
β = -7.426268
F(z) = 1.000000
Pr(f) % = 0.000000
Ic= 0.80
ln(I crit) = -0.223144

Elev. 13.0	Rh
Head = 3.64	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	400	10.00	9.00	137.94	189.74	0.0209	1.60	0.16		
2	400	10.00	9.00	137.94	189.74	0.0209	1.60	0.16	0.000000	0.00
3	400	10.00	9.00	137.94	189.74	0.0209	1.60	0.16		
4	400	11.00	9.00	140.52	199.00	0.0203	1.64	0.15	0.000100	13.79
5	400	9.00	9.00	134.94	180.00	0.0215	1.57	0.17		
6	400	10.00	15.00	150.26	244.95	0.0301	1.79	0.18	0.000625	86.21
7	400	10.00	3.00	100.92	109.54	0.0096	1.27	0.13		
								Total	0.000725	100.00

E[I] = 0.160000
Var[I]= 0.000725
σ[I]= 0.026926
V(I) = 0.168286
E[ln I] = -1.846545
σ [ln I] = 0.167113
β = -11.049687
F(z) = 1.000000
Pr(f) % = 0.000000
Ic= 0.80
ln(I crit) = -0.223144

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Lower San Joaquin
Study Area: Left Bank French Camp Slough
River Section: Index Point FL1

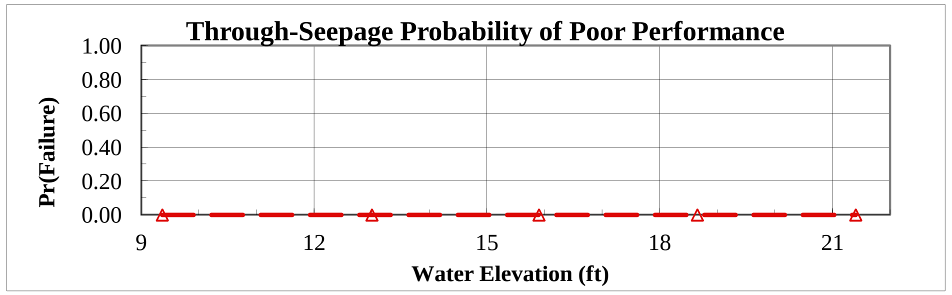
Levee Mile: STA 1049+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 21.40
L/S Toe Elev.: 9.36
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	5	0.5	10.00
Initial Porosity (n)	0.4	0.04	10.00
Initial Permeability (Ko)	1.00E-10	3.00E-11	30.00

Pr(f)=0
NO



Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	9.36	0.0000
Elev. 13.0	3.64	13.00	0.000000
200 year	6.54	15.90	0.000000
Elev. 18.65	9.29	18.65	0.000000
Crest	12.04	21.40	0.000000

Crest **Head =** 12.04 **Horizontal Gradient (Ix) =** 0.380

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	5.00	0.40	1.00E-10	114.02	300.05		
2	4.50	0.40	1.00E-10	102.62	270.05	900.329843	26.44
3	5.50	0.40	1.00E-10	125.42	330.06		
4	5.00	0.36	1.00E-10	108.17	284.66	225.647998	6.63
5	5.00	0.44	1.00E-10	119.59	314.70		
6	5.00	0.40	7.00E-11	136.28	358.63	2278.566570	66.93
7	5.00	0.40	1.30E-10	100.00	263.17		

E[FS] = 300.054969 E[ln FS] = 5.685407 Total 3404.544411 100.00
Var[FS]= 3404.544411
σ[FS]= 58.348474 σ[ln FS]= 0.192658 **β =** 29.510413
V(FS) = 0.194459 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Elev. 18.65 **Head =** 9.29 **Horizontal Gradient (Ix) =** 0.320

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	5.00	0.40	1.00E-10	114.02	356.32		
2	4.50	0.40	1.00E-10	102.62	320.68	1269.605755	26.44
3	5.50	0.40	1.00E-10	125.42	391.95		
4	5.00	0.36	1.00E-10	108.17	338.03	318.198935	6.63
5	5.00	0.44	1.00E-10	119.59	373.71		
6	5.00	0.40	7.00E-11	136.28	425.88	3213.134889	66.93
7	5.00	0.40	1.30E-10	100.00	312.51		

E[FS] = 356.315275 E[ln FS] = 5.857257 Total 4800.939579 100.00
Var[FS]= 4800.939579
σ[FS]= 69.288813 σ[ln FS]= 0.192658 **β =** 30.402411
V(FS) = 0.194459 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

200 year **Head =** 6.54 **Horizontal Gradient (Ix) =** 0.220

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	5.00	0.40	1.00E-10	114.02	518.28		
2	4.50	0.40	1.00E-10	102.62	466.45	2686.108045	26.44
3	5.50	0.40	1.00E-10	125.42	570.10		
4	5.00	0.36	1.00E-10	108.17	491.68	673.214276	6.63
5	5.00	0.44	1.00E-10	119.59	543.57		
6	5.00	0.40	7.00E-11	136.28	619.46	6798.037451	66.93
7	5.00	0.40	1.30E-10	100.00	454.56		

E[FS] = 518.276764 E[ln FS] = 6.231951 Total 10157.359771 100.00
Var[FS]= 10157.359771
σ[FS]= 100.783728 σ[ln FS]= 0.192658 **β =** 32.347277
V(FS) = 0.194459 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Elev. 13.0 **Head =** 3.64 **Horizontal Gradient (Ix) =** 0.140

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	5.00	0.40	1.00E-10	114.02	814.43		
2	4.50	0.40	1.00E-10	102.62	732.99	6633.042314	26.44
3	5.50	0.40	1.00E-10	125.42	895.88		
4	5.00	0.36	1.00E-10	108.17	772.64	1662.427089	6.63
5	5.00	0.44	1.00E-10	119.59	854.19		
6	5.00	0.40	7.00E-11	136.28	973.44	16786.990441	66.93
7	5.00	0.40	1.30E-10	100.00	714.31		

E[FS] = 814.434915 E[ln FS] = 6.683936 Total 25082.459843 100.00
Var[FS]= 25082.459843
σ[FS]= 158.374429 σ[ln FS]= 0.192658 **β =** 34.693331
V(FS) = 0.194459 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

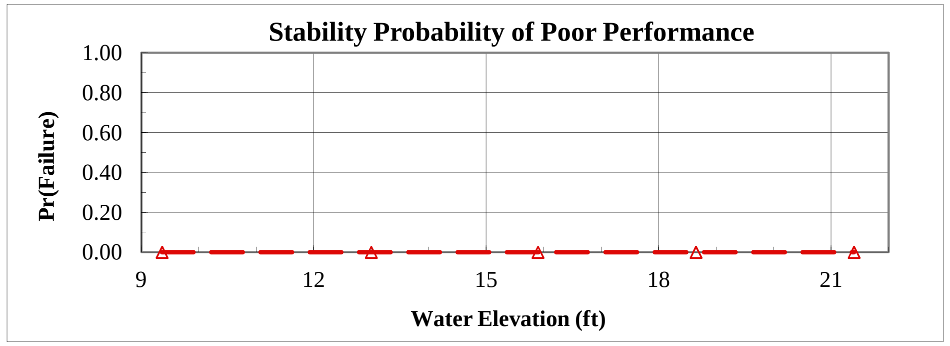
Project: Lower San Joaquin
Study Area: Left Bank French Camp Slough
River Section: Index Point FL1

Levee Mile: STA 1049+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 21.40
L/S Toe Elev.: 9.36
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	28	4	13.00
Levee Cohesion	100	40	40.00
Levee γ	120	8	7.00
Foundation Φ	30	4	13.00
Foundation Cohesion	100	40	40.00



Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	9.36	0.0000
Elev. 13.0	3.64	13.00	0.000000
200 year	6.54	15.90	0.000000
Elev. 18.65	9.29	18.65	0.000000
Crest	12.04	21.40	0.000000

Crest	**Head =**	12.04	**Pr(f)=0**	NO

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	28	100	120	30	100	2.28		
2	24	100	120	30	100	2.20	0.005852	6.83
3	32	100	120	30	100	2.35		
4	28	60	120	30	100	2.21	0.004356	5.09
5	28	140	120	30	100	2.34		
6	28	100	112	30	100	2.32	0.001722	2.01
7	28	100	128	30	100	2.24		
8	28	100	120	26	100	2.09	0.036290	42.36
9	28	100	120	34	100	2.47		
10	28	100	120	30	60	2.07	0.037442	43.71
11	28	100	120	30	140	2.46		

E[FS] = 2.280000   E[ln FS] = 0.816003   Total 0.085663 100.00
Var[FS]= 0.085663
σ[FS]= 0.292682   σ[ln FS]= 0.127845   **β =** 6.382739
V(FS) = 0.128369   **F(z) =** 0.000000
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Elev. 18.65	**Head =**	9.29	**Pr(f)=0**	YES

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	28	100	120	30	100	2.41		
2	24	100	120	30	100			
3	32	100	120	30	100			
4	28	60	120	30	100			
5	28	140	120	30	100			
6	28	100	112	30	100			
7	28	100	128	30	100			
8	28	100	120	26	100			
9	28	100	120	34	100			
10	28	100	120	30	60			
11	28	100	120	30	140			

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   **β =**
V(FS) =   **F(z) =**
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

200 year	**Head =**	6.54	**Pr(f)=0**	YES

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	28	100	120	30	100	2.50		
2	24	100	120	30	100			
3	32	100	120	30	100			
4	28	60	120	30	100			
5	28	140	120	30	100			
6	28	100	112	30	100			
7	28	100	128	30	100			
8	28	100	120	26	100			
9	28	100	120	34	100			
10	28	100	120	30	60			
11	28	100	120	30	140			

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   **β =**
V(FS) =   **F(z) =**
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Elev. 13.0	**Head =**	3.64	**Pr(f)=0**	YES

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	28	100	120	30	100	2.58		
2	24	100	120	30	100			
3	32	100	120	30	100			
4	28	60	120	30	100			
5	28	140	120	30	100			
6	28	100	112	30	100			
7	28	100	128	30	100			
8	28	100	120	26	100			
9	28	100	120	34	100			
10	28	100	120	30	60			
11	28	100	120	30	140			

E[FS] =   E[ln FS] =   Total
Var[FS]=
σ[FS]=   σ[ln FS]=   **β =**
V(FS) =   **F(z) =**
FS req'd = 1.00   ln(FS req'd) = 0.000000   **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

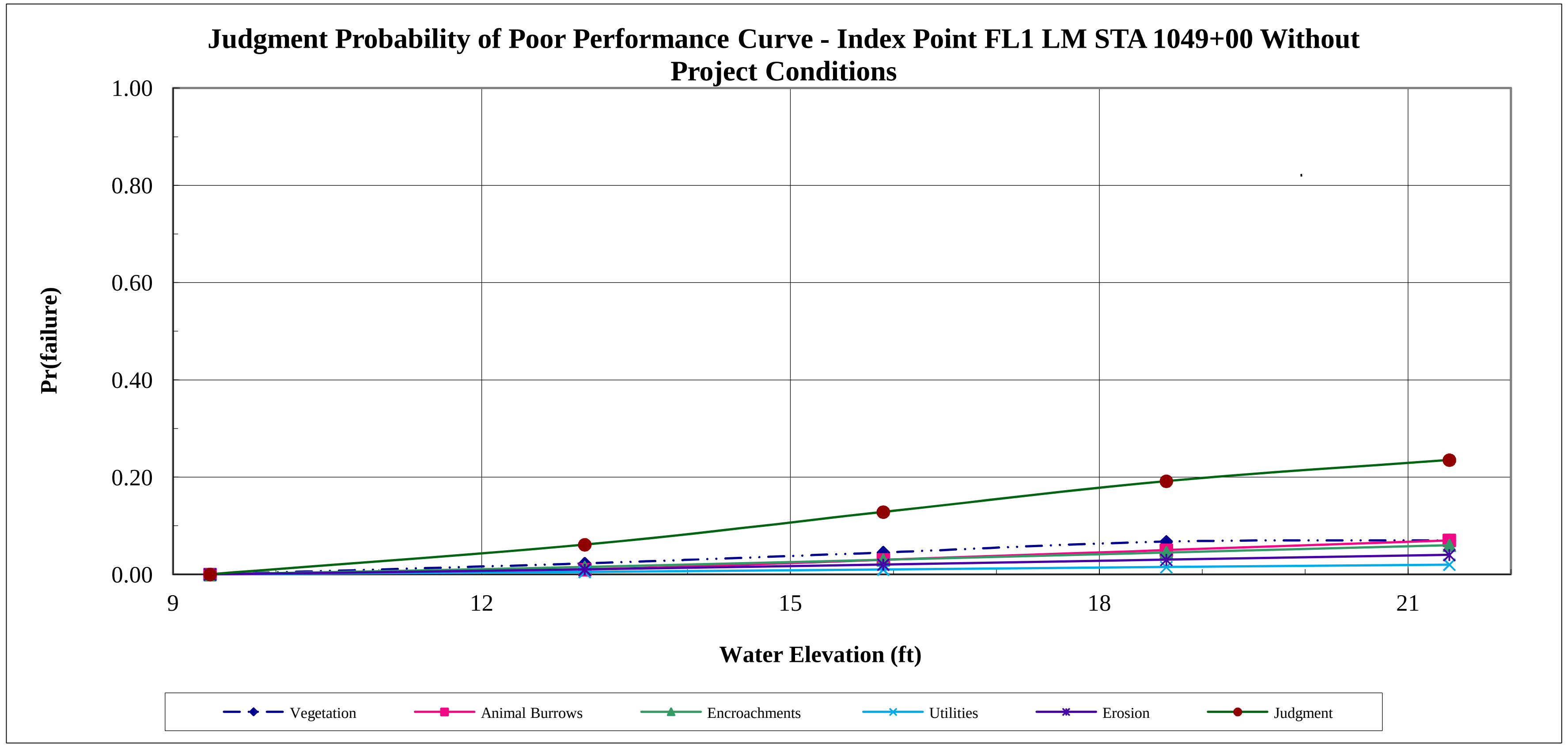
Project: Lower San Joaquin
Study Area: Left Bank French Camp Slough
River Section: Index Point FL1

Levee Mile: STA 1049+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 21.40
L/S Toe Elev.: 9.36
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
9.36	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
13.00	0.0225	0.9775	0.0100	0.9900	0.0150	0.9850	0.0050	0.9950	0.0100	0.9900	0.0610	0.9390
15.90	0.0450	0.9550	0.0300	0.9700	0.0300	0.9700	0.0100	0.9900	0.0200	0.9800	0.1282	0.8718
18.65	0.0675	0.9325	0.0500	0.9500	0.0450	0.9550	0.0150	0.9850	0.0300	0.9700	0.1917	0.8083
21.40	0.0700	0.9300	0.0700	0.9300	0.0600	0.9400	0.0200	0.9800	0.0400	0.9600	0.2351	0.7649


Judgment Probability of Poor Performance Curve - Index Point FL1 LM STA 1049+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

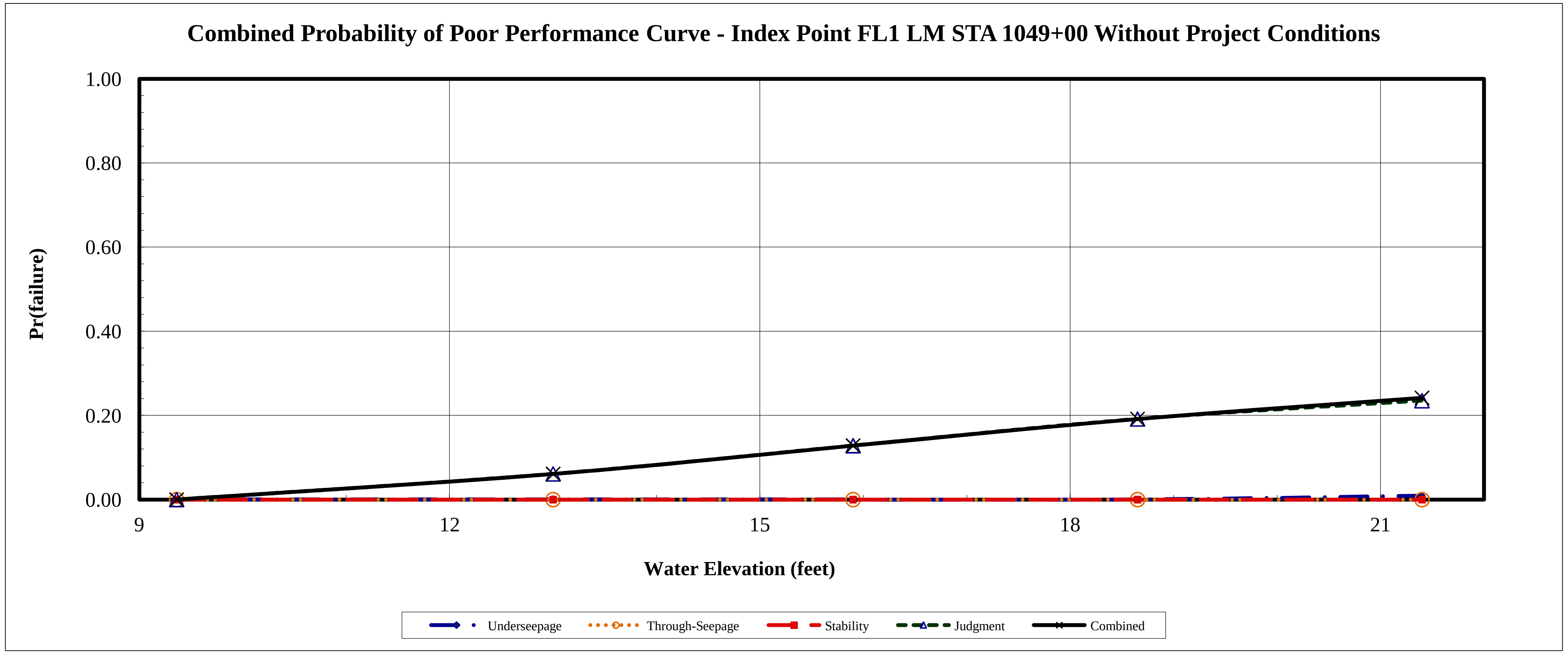
Project: Lower San Joaquin
Study Area: Left Bank French Camp Slough
River Section: Index Point FL1

Levee Mile: STA 1049+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 21.40
L/S Toe Elev.: 9.36
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
9.36	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
13.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0610	0.9390	0.0610	0.9390
15.90	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1282	0.8718	0.1282	0.8718
18.65	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1917	0.8083	0.1917	0.8083
21.40	0.0087	0.9913	0.0000	1.0000	0.0000	1.0000	0.2351	0.7649	0.2418	0.7582



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: Right Bank French Camp Slough
Basin and Reach: Index Point FR1

Levee Mile: STA 1164+20
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 21.77
L/S Toe Elev.: 8.14
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/12/2012
Date: 12/10/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket		Aquifer Material		Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
											Material	Kb (ft/day)	Material	Kf (ft/day)					
WR0404_075C	7	7	2	13	29	10	8	2	19	25	SC	0.007	SM	0.28	40	367	800	391556	98
WR0404_042B	3.5					9					SC	0.007	SM	0.28	40				
WR0404_041B	9					9.5					ML	0.007	SP-SM	14	2000				
1-CPT-43	6.5					5					ML	0.007	SM	0.28	40				
WR0404_043C	8					6.5					CL	0.0007	ML	0.028	40				
WR0404_046B	5					7					CL	0.0007	ML	0.028	40				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WR0404_075C	SC	7	0.007				7	SM	10	0.28							0.28
WR0404_042B	SC	3.5	0.007				3.5	SM	9	0.28							0.28
WR0404_041B	ML	9	0.007				9	SP-SM	9.5	14							14
1-CPT-43	ML	6.5	0.007				6.5	SM	5	0.28							0.28
WR0404_043C	CL	8	0.0007				8	ML	6.5	0.028							0.028
WR0404_046B	CL	5	0.0007				5	ML	7	0.028							0.028

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: Right Bank French Camp Slough
River Section: Index Point FR1

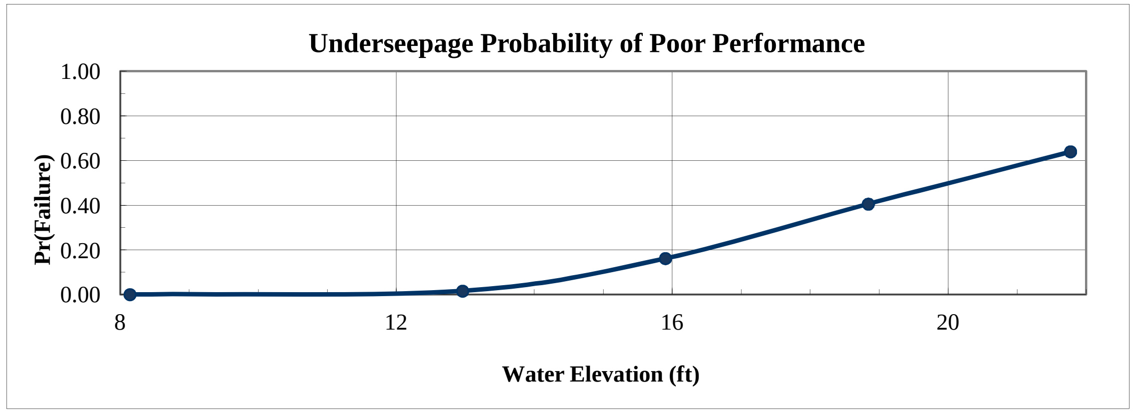
Levee Mile: STA 1164+20
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 21.77
L/S Toe Elev.: 8.14
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/12/2012
Date: 12/10/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permaebility Ratio	367	360	98
Blanket Thickness (z)	7	2	29
Aquifer Thickness (d)	20	2	25

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	150	78	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	8.14	0.0000
E.ev. 12.96	4.82	12.96	0.0157
200 yr	7.76	15.90	0.1615
Elev. 18.84	10.70	18.84	0.4054
Crest	13.63	21.77	0.6396

Crest		**Rh**
Head =	13.63	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	367	7.00	20.00	131.36	226.67	0.0459	7.09	1.01		
2	727	7.00	20.00	139.84	318.96	0.0373	8.10	1.16	0.129600	68.20
3	7	7.00	20.00	32.05	32.06	0.1407	3.07	0.44		
4	367	9.00	20.00	135.01	257.02	0.0426	7.45	0.83	0.060025	31.59
5	367	5.00	20.00	125.37	191.57	0.0506	6.61	1.32		
6	367	7.00	22.00	132.82	237.74	0.0490	7.22	1.03	0.000400	0.21
7	367	7.00	18.00	129.63	215.04	0.0426	6.93	0.99		
								Total	0.190025	100.00

E[I] = 1.010000
Var[I]= 0.190025
σ[I]= 0.435919
V(I) = 0.431603
Ic= 0.80

E[ln I] = -0.075461
σ [ln I] = 0.413307
ln(I crit) = -0.223144

β = -0.182579
F(z) = 0.360427
Pr(f) % = 63.957331

Elev. 18.84		**Rh**
Head =	10.70	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	367	7.00	20.00	131.36	226.67	0.0459	5.56	0.79		
2	727	7.00	20.00	139.84	318.96	0.0373	6.36	0.91	0.081225	67.98
3	7	7.00	20.00	32.05	32.06	0.1407	2.41	0.34		
4	367	9.00	20.00	135.01	257.02	0.0426	5.85	0.65	0.038025	31.83
5	367	5.00	20.00	125.37	191.57	0.0506	5.19	1.04		
6	367	7.00	22.00	132.82	237.74	0.0490	5.67	0.81	0.000225	0.19
7	367	7.00	18.00	129.63	215.04	0.0426	5.44	0.78		
								Total	0.119475	100.00

E[I] = 0.790000
Var[I]= 0.119475
σ[I]= 0.345652
V(I) = 0.437534
Ic= 0.80

E[ln I] = -0.323302
σ [ln I] = 0.418520
ln(I crit) = -0.223144

β = -0.772488
F(z) = 0.594569
Pr(f) % = 40.543051

200 yr		**Rh**
Head =	7.76	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	367	7.00	20.00	131.36	226.67	0.0459	4.03	0.58		
2	727	7.00	20.00	139.84	318.96	0.0373	4.61	0.66	0.042025	67.95
3	7	7.00	20.00	32.05	32.06	0.1407	1.75	0.25		
4	367	9.00	20.00	135.01	257.02	0.0426	4.24	0.47	0.019600	31.69
5	367	5.00	20.00	125.37	191.57	0.0506	3.76	0.75		
6	367	7.00	22.00	132.82	237.74	0.0490	4.11	0.59	0.000225	0.36
7	367	7.00	18.00	129.63	215.04	0.0426	3.95	0.56		
								Total	0.061850	100.00

E[I] = 0.580000
Var[I]= 0.061850
σ[I]= 0.248697
V(I) = 0.428787
Ic= 0.80

E[ln I] = -0.629117
σ [ln I] = 0.410827
ln(I crit) = -0.223144

β = -1.531341
F(z) = 0.838469
Pr(f) % = 16.153113

E.ev. 12.96		**Rh**
Head =	4.82	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	367	7.00	20.00	131.36	226.67	0.0459	2.51	0.36		
2	727	7.00	20.00	139.84	318.96	0.0373	2.86	0.41	0.015625	65.79
3	7	7.00	20.00	32.05	32.06	0.1407	1.09	0.16		
4	367	9.00	20.00	135.01	257.02	0.0426	2.64	0.29	0.008100	34.11
5	367	5.00	20.00	125.37	191.57	0.0506	2.34	0.47		
6	367	7.00	22.00	132.82	237.74	0.0490	2.55	0.36	0.000025	0.11
7	367	7.00	18.00	129.63	215.04	0.0426	2.45	0.35		
								Total	0.023750	100.00

E[I] = 0.360000
Var[I]= 0.023750
σ[I]= 0.154110
V(I) = 0.428084
Ic= 0.80

E[ln I] = -1.105786
σ [ln I] = 0.410207
ln(I crit) = -0.223144

β = -2.695676
F(z) = 0.984289
Pr(f) % = 1.571054

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

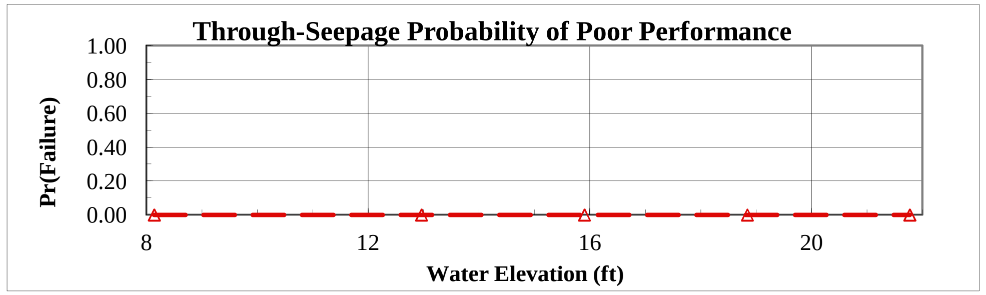
Project: Lower San Joaquin
Study Area: Right Bank French Camp Slough
River Section: Index Point FR1

Levee Mile: STA 1164+20
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 21.77
L/S Toe Elev.: 8.14
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/12/2012
Date: 12/10/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	5	0.5	10.00
Initial Porosity (n)	0.4	0.04	10.00
Initial Permeability (Ko)	1.00E-10	3.00E-11	30.00


Through-Seepage Probability of Poor Performance

Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	8.14	0.0000
E.ev. 12.96	4.82	12.96	0.000000
200 yr	7.76	15.90	0.000000
Elev. 18.84	10.70	18.84	0.000000
Crest	13.63	21.77	0.000000

Pr(f)=0
NO

Crest | **Head =** 13.63 | **Horizontal Gradient (Ix) =** 0.520

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	5.00	0.40	1.00E-10	114.02	219.27		
2	4.50	0.40	1.00E-10	102.62	197.34	480.797446	26.44
3	5.50	0.40	1.00E-10	125.42	241.20		
4	5.00	0.36	1.00E-10	108.17	208.02	120.501372	6.63
5	5.00	0.44	1.00E-10	119.59	229.97		
6	5.00	0.40	7.00E-11	136.28	262.08	1216.808479	66.93
7	5.00	0.40	1.30E-10	100.00	192.31		

E[FS] = 219.270939 | E[ln FS] = 5.371750 | Total 1818.107296 | 100.00
Var[FS]= 1818.107296
σ[FS]= 42.639269 | σ[ln FS]= 0.192658 | β = 27.882356
V(FS) = 0.194459 | F(z) = 0.000000
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Elev. 18.84 | **Head =** 10.70 | **Horizontal Gradient (Ix) =** 0.440

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	5.00	0.40	1.00E-10	114.02	259.14		
2	4.50	0.40	1.00E-10	102.62	233.22	671.527011	26.44
3	5.50	0.40	1.00E-10	125.42	285.05		
4	5.00	0.36	1.00E-10	108.17	245.84	168.303569	6.63
5	5.00	0.44	1.00E-10	119.59	271.79		
6	5.00	0.40	7.00E-11	136.28	309.73	1699.509363	66.93
7	5.00	0.40	1.30E-10	100.00	227.28		

E[FS] = 259.138382 | E[ln FS] = 5.538804 | Total 2539.339943 | 100.00
Var[FS]= 2539.339943
σ[FS]= 50.391864 | σ[ln FS]= 0.192658 | β = 28.749460
V(FS) = 0.194459 | F(z) = 0.000000
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

200 yr | **Head =** 7.76 | **Horizontal Gradient (Ix) =** 0.350

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	5.00	0.40	1.00E-10	114.02	325.77		
2	4.50	0.40	1.00E-10	102.62	293.20	1061.286770	26.44
3	5.50	0.40	1.00E-10	125.42	358.35		
4	5.00	0.36	1.00E-10	108.17	309.06	265.988334	6.63
5	5.00	0.44	1.00E-10	119.59	341.67		
6	5.00	0.40	7.00E-11	136.28	389.37	2685.918471	66.93
7	5.00	0.40	1.30E-10	100.00	285.72		

E[FS] = 325.773966 | E[ln FS] = 5.767645 | Total 4013.193575 | 100.00
Var[FS]= 4013.193575
σ[FS]= 63.349772 | σ[ln FS]= 0.192658 | β = 29.937274
V(FS) = 0.194459 | F(z) = 0.000000
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

E.ev. 12.96 | **Head =** 4.82 | **Horizontal Gradient (Ix) =** 0.240

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	5.00	0.40	1.00E-10	114.02	475.09		
2	4.50	0.40	1.00E-10	102.62	427.58	2257.076899	26.44
3	5.50	0.40	1.00E-10	125.42	522.60		
4	5.00	0.36	1.00E-10	108.17	450.71	565.686995	6.63
5	5.00	0.44	1.00E-10	119.59	498.28		
6	5.00	0.40	7.00E-11	136.28	567.84	5712.239803	66.93
7	5.00	0.40	1.30E-10	100.00	416.68		

E[FS] = 475.087034 | E[ln FS] = 6.144940 | Total 8535.003697 | 100.00
Var[FS]= 8535.003697
σ[FS]= 92.385084 | σ[ln FS]= 0.192658 | β = 31.895640
V(FS) = 0.194459 | F(z) = 0.000000
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

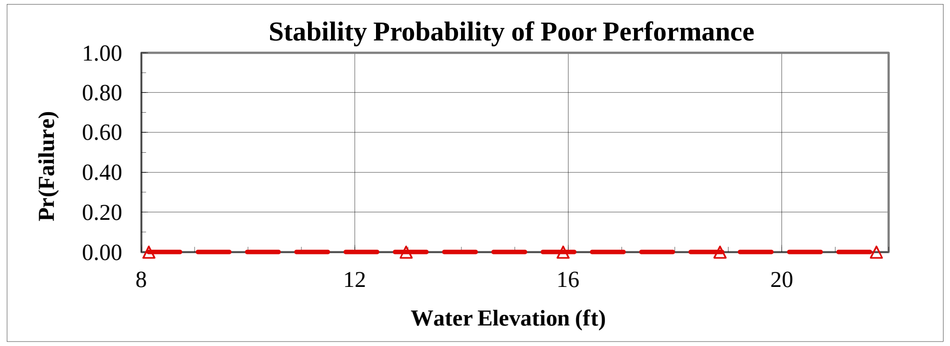
Project: Lower San Joaquin
Study Area: Right Bank French Camp Slough
River Section: Index Point FR1

Levee Mile: STA 1164+20
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 21.77
L/S Toe Elev.: 8.14
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/12/2012
Date: 12/10/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	28	4	13.00
Levee Cohesion	100	40	40.00
Levee γ	120	8	7.00
Foundation Φ	28	4	13.00
Foundation Cohesion	50	20	40.00



Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	8.14	0.0000
E.ev. 12.96	4.82	12.96	0.000000
200 yr	7.76	15.90	0.000000
Elev. 18.84	10.70	18.84	0.000000
Crest	13.63	21.77	0.000000

Crest	**Head =**	13.63	**Pr(f)=0**	NO

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	28	100	120	28	50	1.52		
2	24	100	120	28	50	1.46	0.002209	24.29
3	32	100	120	28	50	1.56		
4	28	60	120	28	50	1.49	0.000729	8.02
5	28	140	120	28	50	1.54		
6	28	100	112	28	50	1.52	0.000081	0.89
7	28	100	128	28	50	1.50		
8	28	100	120	24	50	1.44	0.005476	60.21
9	28	100	120	32	50	1.59		
10	28	100	120	28	30	1.49	0.000600	6.60
11	28	100	120	28	70	1.54		

E[FS] = 1.520000
Var[FS]= 0.009095
σ[FS]= 0.095369
V(FS) = 0.062743
FS req'd = 1.00

E[ln FS] = 0.416746
σ[ln FS]= 0.062681
ln(FS req'd) = 0.000000

Total 0.009095 100.00

β = 6.648663
F(z) = 0.000000
Pr(f) % = 0.000000

Elev. 18.84	**Head =**	10.70	**Pr(f)=0**	YES

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	28	100	120	28	50	1.65		
2	24	100	120	28	50			
3	32	100	120	28	50			
4	28	60	120	28	50			
5	28	140	120	28	50			
6	28	100	112	28	50			
7	28	100	128	28	50			
8	28	100	120	24	50			
9	28	100	120	32	50			
10	28	100	120	28	30			
11	28	100	120	28	70			

E[FS] =
Var[FS]=
σ[FS]=
V(FS) =
FS req'd = 1.00

E[ln FS] =
σ[ln FS]=
ln(FS req'd) = 0.000000

Total

β =
F(z) =
Pr(f) % = 0.000000

200 yr	**Head =**	7.76	**Pr(f)=0**	YES

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	28	100	120	28	50	1.76		
2	24	100	120	28	50			
3	32	100	120	28	50			
4	28	60	120	28	50			
5	28	140	120	28	50			
6	28	100	112	28	50			
7	28	100	128	28	50			
8	28	100	120	24	50			
9	28	100	120	32	50			
10	28	100	120	28	30			
11	28	100	120	28	70			

E[FS] =
Var[FS]=
σ[FS]=
V(FS) =
FS req'd = 1.00

E[ln FS] =
σ[ln FS]=
ln(FS req'd) = 0.000000

Total

β =
F(z) =
Pr(f) % = 0.000000

E.ev. 12.96	**Head =**	4.82	**Pr(f)=0**	YES

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	28	100	120	28	50	1.88		
2	24	100	120	28	50			
3	32	100	120	28	50			
4	28	60	120	28	50			
5	28	140	120	28	50			
6	28	100	112	28	50			
7	28	100	128	28	50			
8	28	100	120	24	50			
9	28	100	120	32	50			
10	28	100	120	28	30			
11	28	100	120	28	70			

E[FS] =
Var[FS]=
σ[FS]=
V(FS) =
FS req'd = 1.00

E[ln FS] =
σ[ln FS]=
ln(FS req'd) = 0.000000

Total

β =
F(z) =
Pr(f) % = 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

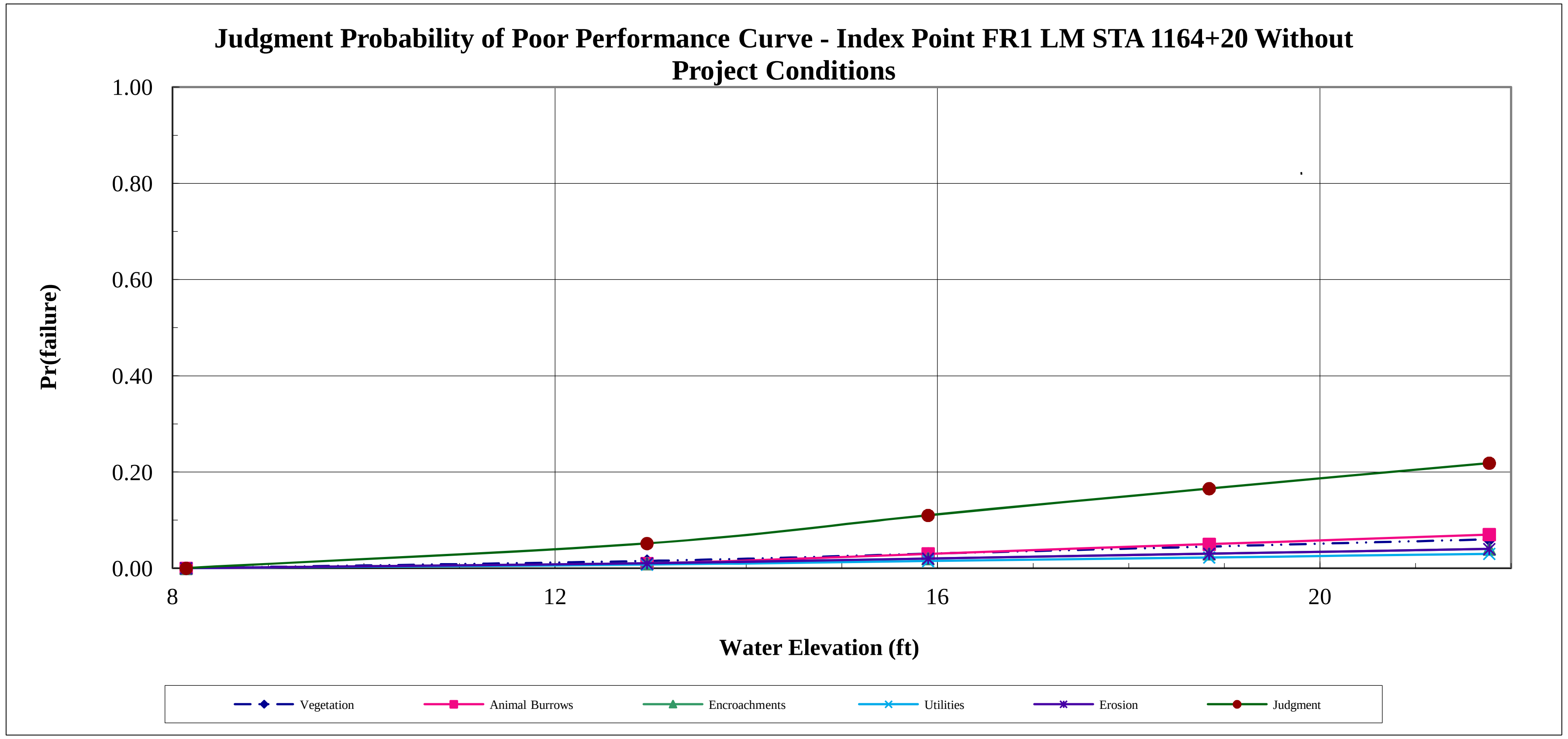
Project: Lower San Joaquin
Study Area: Right Bank French Camp Slough
River Section: Index Point FR1

Levee Mile: STA 1164+20
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 21.77
L/S Toe Elev.: 8.14
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/12/2012
Date: 12/10/2012

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
8.14	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
12.96	0.0150	0.9850	0.0100	0.9900	0.0100	0.9900	0.0075	0.9925	0.0100	0.9900	0.0514	0.9486
15.90	0.0300	0.9700	0.0300	0.9700	0.0200	0.9800	0.0150	0.9850	0.0200	0.9800	0.1099	0.8901
18.84	0.0450	0.9550	0.0500	0.9500	0.0300	0.9700	0.0225	0.9775	0.0300	0.9700	0.1656	0.8344
21.77	0.0600	0.9400	0.0700	0.9300	0.0400	0.9600	0.0300	0.9700	0.0400	0.9600	0.2185	0.7815



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

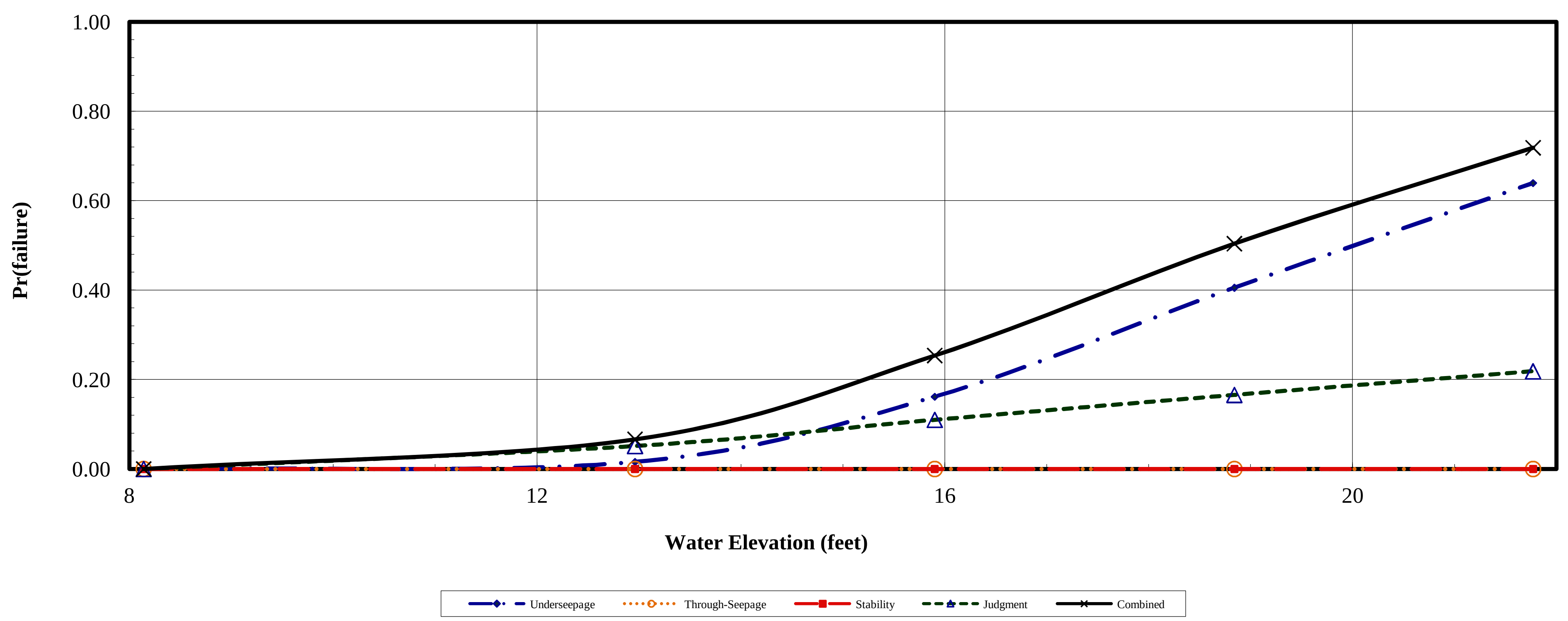
Project: Lower San Joaquin
Study Area: Right Bank French Camp Slough
River Section: Index Point FR1

Levee Mile: STA 1164+20
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 21.77
L/S Toe Elev.: 8.14
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/12/2012
Date: 12/10/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
8.14	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
12.96	0.0157	0.9843	0.0000	1.0000	0.0000	1.0000	0.0514	0.9486	0.0663	0.9337
15.90	0.1615	0.8385	0.0000	1.0000	0.0000	1.0000	0.1099	0.8901	0.2537	0.7463
18.84	0.4054	0.5946	0.0000	1.0000	0.0000	1.0000	0.1656	0.8344	0.5039	0.4961
21.77	0.6396	0.3604	0.0000	1.0000	0.0000	1.0000	0.2185	0.7815	0.7183	0.2817


Combined Probability of Poor Performance Curve - Index Point FR1 LM STA 1164+20 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: Left Bank Stockton Diverting Canal
Basin and Reach: Index Point SL-1
Coordinates: State Plane (ft), N 2183207, E 6340943

Levee Mile: STA 846+68
River Mile: XX.XX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 39.16
L/S Toe Elev.: 25.00
W/S Toe Elev.: 25.00

Analysis By: J. Hogan
Checked By: M. Perlea, G. Johnson
Date: 9/27/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material Material	Aquifer Material Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
WCSBDC_001B	9	10	5	38	50	6	17	11	152	65	CL/ML	0.007	SM	0.28	40	194	192	33493	98
WCSBDC_002B	6					38					CL	0.007	SP-SM	2.8	400				
WCSBDC_003B	16.7					8					CH/ML	0.007	SM	0.28	40				
WCSBDC_004B	6					20					CL	0.007	SM	0.28	40				
WCSBDC_008C	10.8					12					CL/ML	0.007	SP-SM	2.8	400				
WCSBDC_009C	6.4					11					CL/ML	0.007	SP-SM	2.8	400				
WCSBDC_005B	16					24					CL	0.007	ML	0.28	40				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WCSBDC_001B	CL	5	0.007	ML	4	0.007	9	SM	6	0.28							0.28
WCSBDC_002B	CL	6	0.007				6	SP-SM	38	2.8							2.8
WCSBDC_003B	CH	16	0.007	ML	7	0.07	16.7	SM	8	0.28							0.28
WCSBDC_004B	CL	6	0.007				6	SM	20	0.28							0.28
WCSBDC_008C	CL	8	0.007	ML	28	0.07	10.8	SP-SM	12	2.8							2.8
WCSBDC_009C	CL	4	0.007	ML	24	0.07	6.4	SP-SM	11	2.8							2.8
WCSBDC_005B	CL	16	0.007				16	ML	24	0.28							0.28

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: Left Bank Stockton Diverting Canal
River Section: Index Point SL-1
Coordinates: State Plane (ft), N 2183207, E 6340943

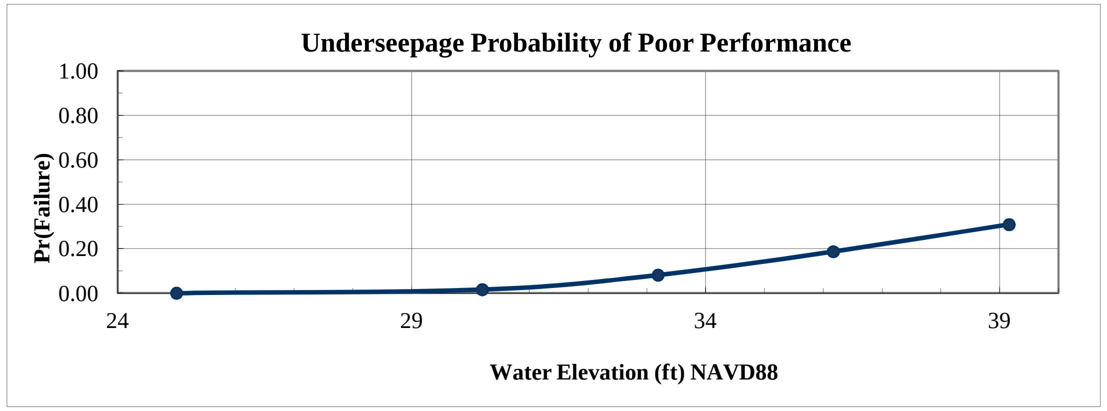
Levee Mile: STA 846+68
River Mile: XX.XX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 39.16
L/S Toe Elev.: 25.00
W/S Toe Elev.: 25.00

Analysis By: J. Hogan
Checked By: M. Perlea, G. Johnson
Date: 9/27/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permeability Ratio	194	192	98
Blanket Thickness (z)	10	5	50
Aquifer Thickness (d)	17	11	65

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	115	77	∞	112



Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	25.00	0.0000
200yr	5.20	30.20	0.0160
200yr + 3ft	8.19	33.19	0.0813
Crest-3ft	11.17	36.17	0.1869
Crest	14.16	39.16	0.3087

Crest	**Rh**
Head = 14.16	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	194	10.00	17.00	101.75	181.60	0.0472	7.14	0.71		
2	386	10.00	17.00	107.85	256.16	0.0385	8.22	0.82	0.087025	39.27
3	2	10.00	17.00	18.44	18.44	0.1493	2.29	0.23		
4	194	15.00	17.00	105.74	222.42	0.0420	7.77	0.52	0.122500	55.27
5	194	5.00	17.00	91.70	128.41	0.0572	6.12	1.22		
6	194	10.00	28.00	106.49	233.07	0.0672	7.92	0.79	0.012100	5.46
7	194	10.00	6.00	85.01	107.89	0.0222	5.66	0.57		
								Total	0.221625	100.00

E[I] = 0.710000
Var[I]= 0.221625
σ[I]= 0.470771
V(I) = 0.663057
E[ln I] = -0.524689
σ [ln I] = 0.603653
Ic= 0.80
ln(I crit) = -0.223144
β = -0.869190
F(z) = 0.691298
Pr(f) % = 30.870162

Crest-3ft	**Rh**
Head = 11.17	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	194	10.00	17.00	101.75	181.60	0.0472	5.63	0.56		
2	386	10.00	17.00	107.85	256.16	0.0385	6.49	0.65	0.055225	38.97
3	2	10.00	17.00	18.44	18.44	0.1493	1.81	0.18		
4	194	15.00	17.00	105.74	222.42	0.0420	6.13	0.41	0.078400	55.32
5	194	5.00	17.00	91.70	128.41	0.0572	4.83	0.97		
6	194	10.00	28.00	106.49	233.07	0.0672	6.25	0.63	0.008100	5.72
7	194	10.00	6.00	85.01	107.89	0.0222	4.47	0.45		
								Total	0.141725	100.00

E[I] = 0.560000
Var[I]= 0.141725
σ[I]= 0.376464
V(I) = 0.672257
E[ln I] = -0.766265
σ [ln I] = 0.610650
Ic= 0.80
ln(I crit) = -0.223144
β = -1.254836
F(z) = 0.813110
Pr(f) % = 18.688985

200yr + 3ft	**Rh**
Head = 8.19	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	194	10.00	17.00	101.75	181.60	0.0472	4.13	0.41		
2	386	10.00	17.00	107.85	256.16	0.0385	4.76	0.48	0.030625	39.84
3	2	10.00	17.00	18.44	18.44	0.1493	1.33	0.13		
4	194	15.00	17.00	105.74	222.42	0.0420	4.50	0.30	0.042025	54.67
5	194	5.00	17.00	91.70	128.41	0.0572	3.54	0.71		
6	194	10.00	28.00	106.49	233.07	0.0672	4.58	0.46	0.004225	5.50
7	194	10.00	6.00	85.01	107.89	0.0222	3.27	0.33		
								Total	0.076875	100.00

E[I] = 0.410000
Var[I]= 0.076875
σ[I]= 0.277263
V(I) = 0.676252
E[ln I] = -1.079897
σ [ln I] = 0.613675
Ic= 0.80
ln(I crit) = -0.223144
β = -1.759721
F(z) = 0.918658
Pr(f) % = 8.134187

200yr	**Rh**
Head = 5.20	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	194	10.00	17.00	101.75	181.60	0.0472	2.62	0.26		
2	386	10.00	17.00	107.85	256.16	0.0385	3.02	0.30	0.012100	39.54
3	2	10.00	17.00	18.44	18.44	0.1493	0.84	0.08		
4	194	15.00	17.00	105.74	222.42	0.0420	2.85	0.19	0.016900	55.23
5	194	5.00	17.00	91.70	128.41	0.0572	2.25	0.45		
6	194	10.00	28.00	106.49	233.07	0.0672	2.91	0.29	0.001600	5.23
7	194	10.00	6.00	85.01	107.89	0.0222	2.08	0.21		
								Total	0.030600	100.00

E[I] = 0.260000
Var[I]= 0.030600
σ[I]= 0.174929
V(I) = 0.672802
E[ln I] = -1.533773
σ [ln I] = 0.611063
Ic= 0.80
ln(I crit) = -0.223144
β = -2.510007
F(z) = 0.984017
Pr(f) % = 1.598306

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Lower San Joaquin
Study Area: Left Bank Stockton Diverting Canal
River Section: Index Point SL-1
Coordinates: State Plane (ft), N 2183207, E 6340943

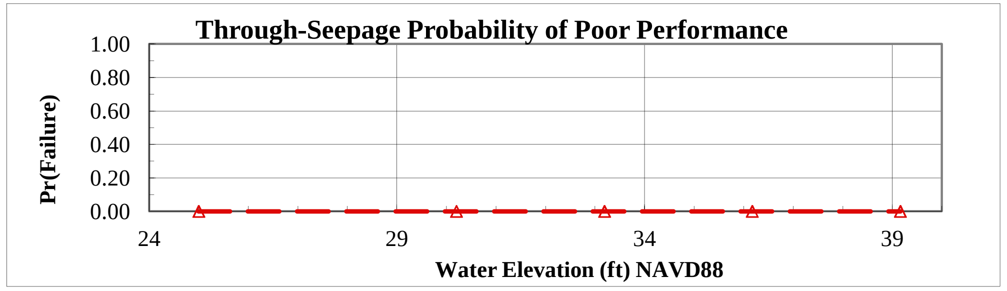
Levee Mile: STA 846+68
River Mile: XX.XX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 39.16
L/S Toe Elev.: 25.00
W/S Toe Elev.: 25.00

Analysis By: J. Hogan
Checked By: M. Perlea, G. Johnson
Date: 9/27/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	50	5.0	10.00
Initial Porosity (n)	0.7	0.07	10.00
Initial Permeability (Ko)	1.00E-10	3.00E-11	30.00

Pr(f)=0
NO



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	25.00	0.0000
200yr	5.20	30.20	0.000000
200yr + 3ft	8.19	33.19	0.000000
Crest-3ft	11.17	36.17	0.000000
Crest	14.16	39.16	0.000000

Crest | **Head =** 14.16 | **Horizontal Gradient (Ix) =** 0.480

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	50.00	0.70	1.00E-10	1508.35	3142.41		
2	45.00	0.70	1.00E-10	1357.52	2828.16	98747.114311	26.44
3	55.00	0.70	1.00E-10	1659.19	3456.65		
4	50.00	0.63	1.00E-10	1430.95	2981.15	24748.806051	6.63
5	50.00	0.77	1.00E-10	1581.98	3295.78		
6	50.00	0.70	7.00E-11	1802.83	3755.89	249910.491369	66.93
7	50.00	0.70	1.30E-10	1322.91	2756.07		

E[FS] = 3142.405358 | E[ln FS] = 8.034185 | Total 373406.411731 | 100.00
Var[FS]= 373406.411731
σ[FS]= 611.069891 | σ[ln FS]= 0.192658 | β = 41.701873
V(FS) = 0.194459 | F(z) = 0.000000
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Crest-3ft | **Head =** 11.17 | **Horizontal Gradient (Ix) =** 0.450

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	50.00	0.70	1.00E-10	1508.35	3351.90		
2	45.00	0.70	1.00E-10	1357.52	3016.71	112352.272282	26.44
3	55.00	0.70	1.00E-10	1659.19	3687.09		
4	50.00	0.63	1.00E-10	1430.95	3179.89	28158.641551	6.63
5	50.00	0.77	1.00E-10	1581.98	3515.50		
6	50.00	0.70	7.00E-11	1802.83	4006.29	284342.603513	66.93
7	50.00	0.70	1.30E-10	1322.91	2939.81		

E[FS] = 3351.899048 | E[ln FS] = 8.098724 | Total 424853.517347 | 100.00
Var[FS]= 424853.517347
σ[FS]= 651.807884 | σ[ln FS]= 0.192658 | β = 42.036863
V(FS) = 0.194459 | F(z) = 0.000000
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

200yr + 3ft | **Head =** 8.19 | **Horizontal Gradient (Ix) =** 0.320

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	50.00	0.70	1.00E-10	1508.35	4713.61		
2	45.00	0.70	1.00E-10	1357.52	4242.25	222181.007199	26.44
3	55.00	0.70	1.00E-10	1659.19	5184.97		
4	50.00	0.63	1.00E-10	1430.95	4471.72	55684.813615	6.63
5	50.00	0.77	1.00E-10	1581.98	4943.67		
6	50.00	0.70	7.00E-11	1802.83	5633.84	562298.605580	66.93
7	50.00	0.70	1.30E-10	1322.91	4134.11		

E[FS] = 4713.608036 | E[ln FS] = 8.439650 | Total 840164.426394 | 100.00
Var[FS]= 840164.426394
σ[FS]= 916.604837 | σ[ln FS]= 0.192658 | β = 43.806461
V(FS) = 0.194459 | F(z) = 0.000000
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

200yr | **Head =** 5.20 | **Horizontal Gradient (Ix) =** 0.290

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	50.00	0.70	1.00E-10	1508.35	5201.22		
2	45.00	0.70	1.00E-10	1357.52	4681.10	270527.171667	26.44
3	55.00	0.70	1.00E-10	1659.19	5721.34		
4	50.00	0.63	1.00E-10	1430.95	4934.31	67801.723117	6.63
5	50.00	0.77	1.00E-10	1581.98	5455.09		
6	50.00	0.70	7.00E-11	1802.83	6216.65	684653.712383	66.93
7	50.00	0.70	1.30E-10	1322.91	4561.77		

E[FS] = 5201.222661 | E[ln FS] = 8.538091 | Total 1022982.607167 | 100.00
Var[FS]= 1022982.607167
σ[FS]= 1011.426027 | σ[ln FS]= 0.192658 | β = 44.317420
V(FS) = 0.194459 | F(z) = 0.000000
FS req'd = 1.00 | ln(FS req'd) = 0.000000 | **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

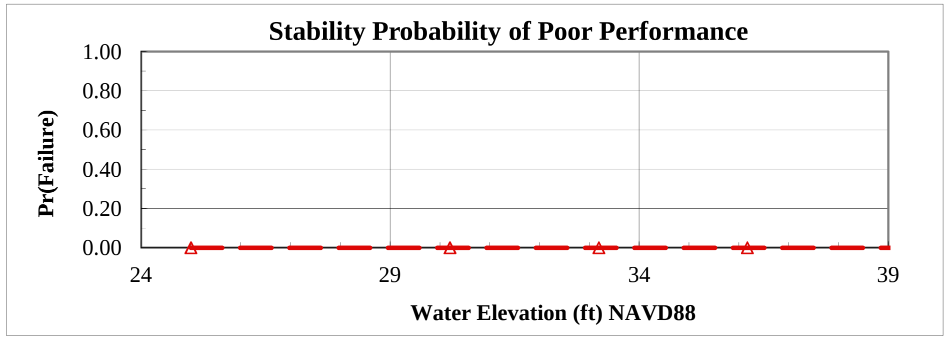
Project: Lower San Joaquin
Study Area: Left Bank Stockton Diverting Canal
River Section: Index Point SL-1
Coordinates: State Plane (ft), N 2183207, E 6340943

Levee Mile: STA 846+68
River Mile: XX.XX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 39.16
L/S Toe Elev.: 25.00
W/S Toe Elev.: 25.00

Analysis By: J. Hogan
Checked By: M. Perlea, G. Johnson
Date: 9/27/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	34	4	13.00
Levee Cohesion	100	40	40.00
Levee γ	115	8	7.00
Foundation Φ	31	4	13.00
Foundation Cohesion	150	60	40.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	25.00	0.0000
200yr	5.20	30.20	0.000000
200yr + 3ft	8.19	33.19	0.000000
Crest-3ft	11.17	36.17	0.000000
Crest	14.16	39.16	0.000000

Crest **Head =** 14.16 **Pr(f)=0** NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	115	31	150	1.40		
2	30	100	115	31	150	1.32	0.002401	52.46
3	38	100	115	31	150	1.41		
4	34	60	115	31	150	1.39	0.000030	0.66
5	34	140	115	31	150	1.40		
6	34	100	107	31	150	1.40	0.000529	11.56
7	34	100	123	31	150	1.35		
8	34	100	115	27	150	1.34	0.001560	34.09
9	34	100	115	35	150	1.42		
10	34	100	115	31	90	1.39	0.000056	1.23
11	34	100	115	31	210	1.41		

E[FS] = 1.397000 E[ln FS] = 0.333156 Total 0.004577 100.00
Var[FS]= 0.004577
σ[FS]= 0.067652 σ[ln FS]= 0.048398 **β =** 6.883664
V(FS) = 0.048426 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Crest-3ft **Head =** 11.17 **Pr(f)=0** YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	115	31	150	1.13		
2	30	100	115	31	150	1.02	0.008372	28.40
3	38	100	115	31	150	1.21		
4	34	60	115	31	150	1.08	0.004225	14.33
5	34	140	115	31	150	1.21		
6	34	100	107	31	150	1.13	0.002704	9.17
7	34	100	123	31	150	1.23		
8	34	100	115	27	150	1.10	0.008556	29.02
9	34	100	115	35	150	1.28		
10	34	100	115	31	90	1.08	0.005625	19.08
11	34	100	115	31	210	1.23		

E[FS] = E[ln FS] = Total 0.029483 100.00
Var[FS]= 0.029483
σ[FS]= 0.171705 σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

200yr + 3ft **Head =** 8.19 **Pr(f)=0** YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	115	31	150	1.27		
2	30	100	115	31	150	1.12	0.012769	79.39
3	38	100	115	31	150	1.34		
4	34	60	115	31	150	1.19	0.001156	7.19
5	34	140	115	31	150	1.26		
6	34	100	107	31	150	1.22	0.000016	0.10
7	34	100	123	31	150	1.23		
8	34	100	115	27	150	1.20	0.001722	10.71
9	34	100	115	35	150	1.29		
10	34	100	115	31	90	1.21	0.000420	2.61
11	34	100	115	31	210	1.25		

E[FS] = E[ln FS] = Total 0.016084 100.00
Var[FS]= 0.016084
σ[FS]= 0.126821 σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

200yr **Head =** 5.20 **Pr(f)=0** YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	115	31	150	1.30		
2	30	100	115	31	150			
3	38	100	115	31	150			
4	34	60	115	31	150			
5	34	140	115	31	150			
6	34	100	107	31	150			
7	34	100	123	31	150			
8	34	100	115	27	150			
9	34	100	115	35	150			
10	34	100	115	31	90			
11	34	100	115	31	210			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

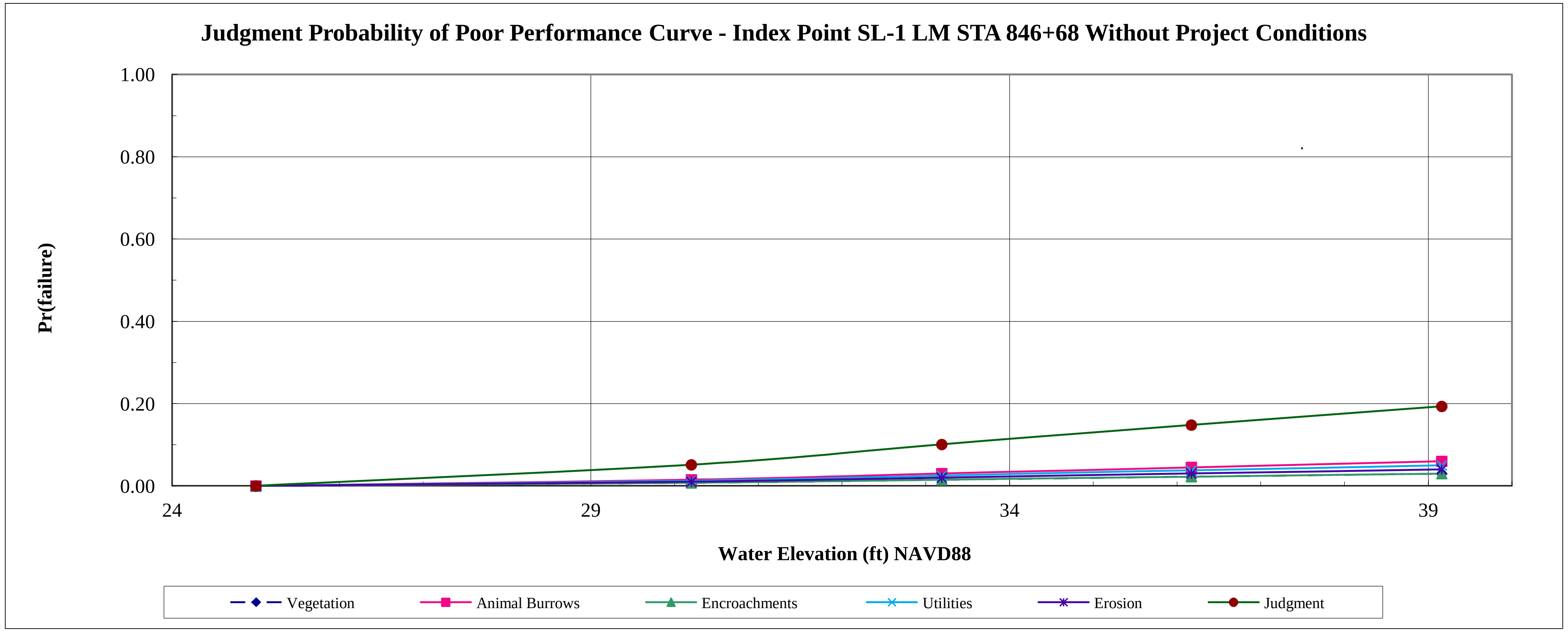
Project: Lower San Joaquin
Study Area: Left Bank Stockton Diverting Canal
River Section: Index Point SL-1
Coordinates: State Plane (ft), N 2183207, E 6340943

Levee Mile: STA 846+68
River Mile: XX.XX
Analysis Case: Without Project Conditions

Datum: NAVD 88
Crest Elev.: 39.16
L/S Toe Elev.: 25.00
W/S Toe Elev.: 25.00

Analysis By: J. Hogan
Checked By: M. Perlea, G. John
Date: 9/27/2012

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
25.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
30.20	0.0075	0.9925	0.0150	0.9850	0.0075	0.9925	0.0125	0.9875	0.0100	0.9900	0.0514	0.9486
33.19	0.0150	0.9850	0.0300	0.9700	0.0150	0.9850	0.0250	0.9750	0.0200	0.9800	0.1008	0.8992
36.17	0.0225	0.9775	0.0450	0.9550	0.0225	0.9775	0.0375	0.9625	0.0300	0.9700	0.1481	0.8519
39.16	0.0300	0.9700	0.0600	0.9400	0.0300	0.9700	0.0500	0.9500	0.0400	0.9600	0.1934	0.8066


Judgment Probability of Poor Performance Curve - Index Point SL-1 LM STA 846+68 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

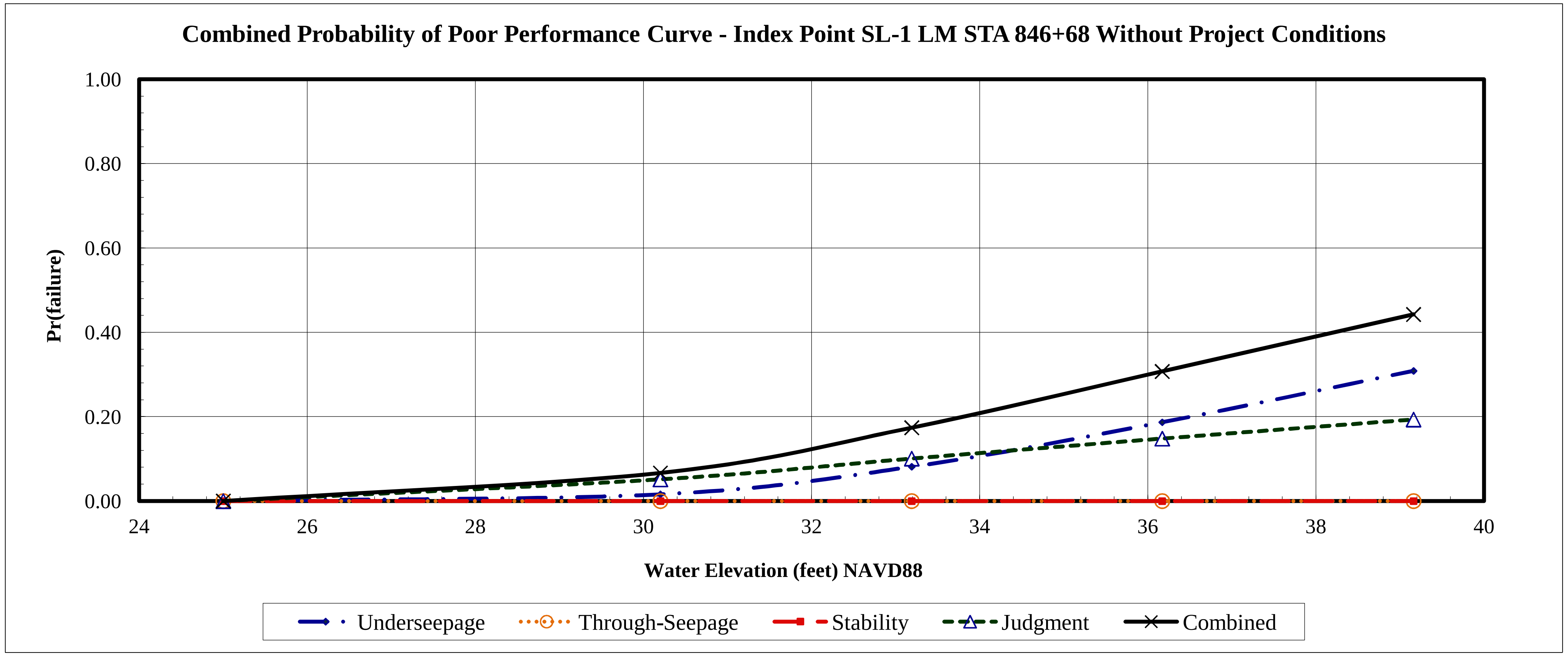
Project: Lower San Joaquin
Study Area: Left Bank Stockton Diverting Canal
River Section: Index Point SL-1
Coordinates: State Plane (ft), N 2183207, E 6340943

Levee Mile: STA 846+68
River Mile: XX.XX
Analysis Case: Without Project Conditions

Datum: NAVD 88
Crest Elev.: 39.16
L/S Toe Elev.: 25.00
W/S Toe Elev.: 25.00

Analysis By: J. Hogan
Checked By: M. Perlea, G. Joh
Date: 9/27/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
25.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
30.20	0.0160	0.9840	0.0000	1.0000	0.0000	1.0000	0.0514	0.9486	0.0666	0.9334
33.19	0.0813	0.9187	0.0000	1.0000	0.0000	1.0000	0.1008	0.8992	0.1739	0.8261
36.17	0.1869	0.8131	0.0000	1.0000	0.0000	1.0000	0.1481	0.8519	0.3073	0.6927
39.16	0.3087	0.6913	0.0000	1.0000	0.0000	1.0000	0.1934	0.8066	0.4424	0.5576



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: Left Bank Stockton Diverting Canal
Basin and Reach: Index Point SL-2
Coordinates: State Plane (ft), N 2176913, E 6352470

Levee Mile: STA 976+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 44.56
L/S Toe Elev.: 34.30
W/S Toe Elev.: 34.79

Analysis By: J. Hogan
Checked By: M. Perlea, G. Johnson
Date: 9/27/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket		Aquifer Material		Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
											Material	Kb (ft/day)	Material	Kf (ft/day)					
WCSBDC_007B	6	7	2	15	29	16	10	6	48	60	CL	0.007	SM	1.4	200	267	103	24889	39
WCSBDC_008B	10					19					CL	0.007	SP-SM	2.8	400				
WCSBDC_009B	4.6					3					CL/ML	0.007	SM	1.4	200				
WCSBDC_011B	5					4					CL	0.007	SM	1.4	200				
WCSBDC_012B	8					8					CL	0.0007	SC	0.28	400				
WCSBDC_025C	8					8					CL	0.007	SM	1.4	200				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WCSBDC_007B	CL	6	0.007				6	SM	16	1.4							1.4
WCSBDC_008B	CL	10	0.007				10	SP-SM	19	2.8							2.8
WCSBDC_009B	CL	4	0.007	ML	6	0.07	4.6	SM	3	1.4							1.4
WCSBDC_011B	CL	5	0.007				5	SM	4	1.4							1.4
WCSBDC_012B	CL	8	0.0007				8	SC	8	0.28							0.28
WCSBDC_025C	CL	8	0.007				8	SM	8	1.4							1.4

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: Left Bank Stockton Diverting Canal
River Section: Index Point SL-2
Coordinates: State Plane (ft), N 2176913, E 6352470

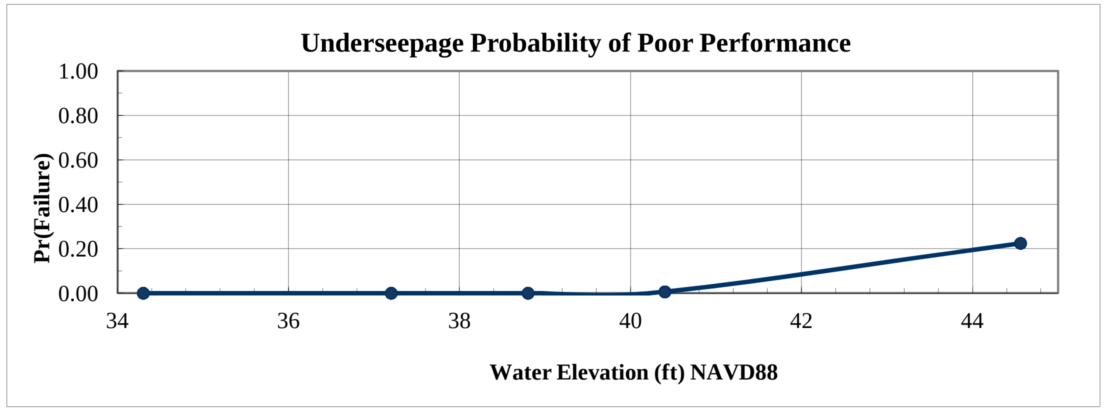
Levee Mile: STA 976+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 44.56
L/S Toe Elev.: 34.30
W/S Toe Elev.: 34.79

Analysis By: J. Hogan
Checked By: M. Perlea, G. Johnson
Date: 9/27/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permaebility Ratio	267	103	39
Blanket Thickness (z)	7	2	29
Aquifer Thickness (d)	10	6	60

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	97	77	∞	112


Underseepage Probability of Poor Performance

Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	34.30	0.0000
200yr	2.90	37.20	0.0000
200yr + 3ft	4.50	38.80	0.0002
Crest-3ft	6.10	40.40	0.0062
Crest	10.26	44.56	0.2245

Crest		**Rh**
Head =	10.26	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	267	7.00	10.00	83.45	136.71	0.0337	4.72	0.67		
2	370	7.00	10.00	86.74	160.93	0.0308	5.09	0.73	0.004225	10.76
3	164	7.00	10.00	77.02	107.14	0.0383	4.21	0.60		
4	267	9.00	10.00	86.05	155.02	0.0314	5.00	0.56	0.024025	61.17
5	267	5.00	10.00	79.21	115.54	0.0368	4.36	0.87		
6	267	7.00	16.00	87.96	172.93	0.0474	5.25	0.75	0.011025	28.07
7	267	7.00	4.00	69.88	86.46	0.0171	3.80	0.54		
								Total	0.039275	100.00

E[I] = 0.670000
Var[I]= 0.039275
σ[I]= 0.198179
V(I) = 0.295790
E[ln I] = -0.442414
σ [ln I] = 0.289610
ln(I crit) = -0.223144

Ic=	0.80

β =	-1.527623
F(z) =	0.775513
Pr(f) % =	22.448734

Crest-3ft		**Rh**
Head =	6.10	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	267	7.00	10.00	83.45	136.71	0.0337	2.81	0.40		
2	370	7.00	10.00	86.74	160.93	0.0308	3.02	0.43	0.001225	8.46
3	164	7.00	10.00	77.02	107.14	0.0383	2.50	0.36		
4	267	9.00	10.00	86.05	155.02	0.0314	2.97	0.33	0.009025	62.35
5	267	5.00	10.00	79.21	115.54	0.0368	2.59	0.52		
6	267	7.00	16.00	87.96	172.93	0.0474	3.12	0.45	0.004225	29.19
7	267	7.00	4.00	69.88	86.46	0.0171	2.26	0.32		
								Total	0.014475	100.00

E[I] = 0.400000
Var[I]= 0.014475
σ[I]= 0.120312
V(I) = 0.300780
E[ln I] = -0.959595
σ [ln I] = 0.294292
ln(I crit) = -0.223144

Ic=	0.80

β =	-3.260691
F(z) =	0.993833
Pr(f) % =	0.616682

200yr + 3ft		**Rh**
Head =	4.50	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	267	7.00	10.00	83.45	136.71	0.0337	2.07	0.30		
2	370	7.00	10.00	86.74	160.93	0.0308	2.23	0.32	0.000900	11.50
3	164	7.00	10.00	77.02	107.14	0.0383	1.85	0.26		
4	267	9.00	10.00	86.05	155.02	0.0314	2.19	0.24	0.004900	62.62
5	267	5.00	10.00	79.21	115.54	0.0368	1.91	0.38		
6	267	7.00	16.00	87.96	172.93	0.0474	2.30	0.33	0.002025	25.88
7	267	7.00	4.00	69.88	86.46	0.0171	1.67	0.24		
								Total	0.007825	100.00

E[I] = 0.300000
Var[I]= 0.007825
σ[I]= 0.088459
V(I) = 0.294863
E[ln I] = -1.245658
σ [ln I] = 0.288739
ln(I crit) = -0.223144

Ic=	0.80

β =	-4.314124
F(z) =	0.999801
Pr(f) % =	0.019908

200yr		**Rh**
Head =	2.90	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	267	7.00	10.00	83.45	136.71	0.0337	1.33	0.19		
2	370	7.00	10.00	86.74	160.93	0.0308	1.44	0.21	0.000400	12.03
3	164	7.00	10.00	77.02	107.14	0.0383	1.19	0.17		
4	267	9.00	10.00	86.05	155.02	0.0314	1.41	0.16	0.002025	60.90
5	267	5.00	10.00	79.21	115.54	0.0368	1.23	0.25		
6	267	7.00	16.00	87.96	172.93	0.0474	1.48	0.21	0.000900	27.07
7	267	7.00	4.00	69.88	86.46	0.0171	1.07	0.15		
								Total	0.003325	100.00

E[I] = 0.190000
Var[I]= 0.003325
σ[I]= 0.057663
V(I) = 0.303488
E[ln I] = -1.704785
σ [ln I] = 0.296829
ln(I crit) = -0.223144

Ic=	0.80

β =	-5.743329
F(z) =	1.000000
Pr(f) % =	0.000030

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Lower San Joaquin
Study Area: Left Bank Stockton Diverting Canal
River Section: Index Point SL-2
Coordinates: State Plane (ft), N 2176913, E 6352470

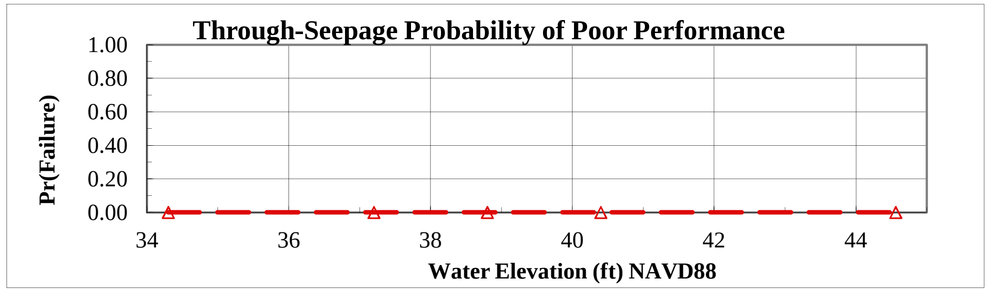
Levee Mile: STA 976+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 44.56
L/S Toe Elev.: 34.30
W/S Toe Elev.: 34.79

Analysis By: J. Hogan
Checked By: M. Perlea, G. Johnson
Date: 9/27/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	50	5.0	10.00
Initial Porosity (n)	0.35	0.04	10.00
Initial Permeability (Ko)	5.00E-10	1.50E-10	30.00

Pr(f)=0
NO



Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	34.30	0.0000
200yr	2.90	37.20	0.000000
200yr + 3ft	4.50	38.80	0.000000
Crest-3ft	6.10	40.40	0.000000
Crest	10.26	44.56	0.000000

Crest	**Head =**	10.26	**Horizontal Gradient (Ix) =**	0.470

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	50.00	0.35	5.00E-10	476.98	1014.86		
2	45.00	0.35	5.00E-10	429.29	913.37	10299.382135	26.44
3	55.00	0.35	5.00E-10	524.68	1116.34		
4	50.00	0.32	5.00E-10	452.51	962.78	2581.315036	6.63
5	50.00	0.39	5.00E-10	500.26	1064.39		
6	50.00	0.35	3.50E-10	570.10	1212.99	26065.811323	66.93
7	50.00	0.35	6.50E-10	418.34	890.09		

E[FS] = 1014.858716 E[ln FS] = 6.903946 Total 38946.508494 100.00
Var[FS]= 38946.508494
σ[FS]= 197.348698 σ[ln FS]= 0.192658 **β =** 35.835305
V(FS) = 0.194459 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Crest-3ft	**Head =**	6.10	**Horizontal Gradient (Ix) =**	0.470

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	50.00	0.35	5.00E-10	476.98	1014.86		
2	45.00	0.35	5.00E-10	429.29	913.37	10299.382135	26.44
3	55.00	0.35	5.00E-10	524.68	1116.34		
4	50.00	0.32	5.00E-10	452.51	962.78	2581.315036	6.63
5	50.00	0.39	5.00E-10	500.26	1064.39		
6	50.00	0.35	3.50E-10	570.10	1212.99	26065.811323	66.93
7	50.00	0.35	6.50E-10	418.34	890.09		

E[FS] = 1014.858716 E[ln FS] = 6.903946 Total 38946.508494 100.00
Var[FS]= 38946.508494
σ[FS]= 197.348698 σ[ln FS]= 0.192658 **β =** 35.835305
V(FS) = 0.194459 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

200yr + 3ft	**Head =**	4.50	**Horizontal Gradient (Ix) =**	0.380

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	50.00	0.35	5.00E-10	476.98	1255.22		
2	45.00	0.35	5.00E-10	429.29	1129.70	15755.772256	26.44
3	55.00	0.35	5.00E-10	524.68	1380.74		
4	50.00	0.32	5.00E-10	452.51	1190.81	3948.839968	6.63
5	50.00	0.39	5.00E-10	500.26	1316.49		
6	50.00	0.35	3.50E-10	570.10	1500.27	39874.914966	66.93
7	50.00	0.35	6.50E-10	418.34	1100.90		

E[FS] = 1255.219991 E[ln FS] = 7.116508 Total 59579.527190 100.00
Var[FS]= 59579.527190
σ[FS]= 244.089179 σ[ln FS]= 0.192658 **β =** 36.938617
V(FS) = 0.194459 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

200yr	**Head =**	2.90	**Horizontal Gradient (Ix) =**	0.320

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	50.00	0.35	5.00E-10	476.98	1490.57		
2	45.00	0.35	5.00E-10	429.29	1341.52	22218.100720	26.44
3	55.00	0.35	5.00E-10	524.68	1639.63		
4	50.00	0.32	5.00E-10	452.51	1414.08	5568.481361	6.63
5	50.00	0.39	5.00E-10	500.26	1563.33		
6	50.00	0.35	3.50E-10	570.10	1781.58	56229.860558	66.93
7	50.00	0.35	6.50E-10	418.34	1307.32		

E[FS] = 1490.573739 E[ln FS] = 7.288358 Total 84016.442639 100.00
Var[FS]= 84016.442639
σ[FS]= 289.855900 σ[ln FS]= 0.192658 **β =** 37.830615
V(FS) = 0.194459 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

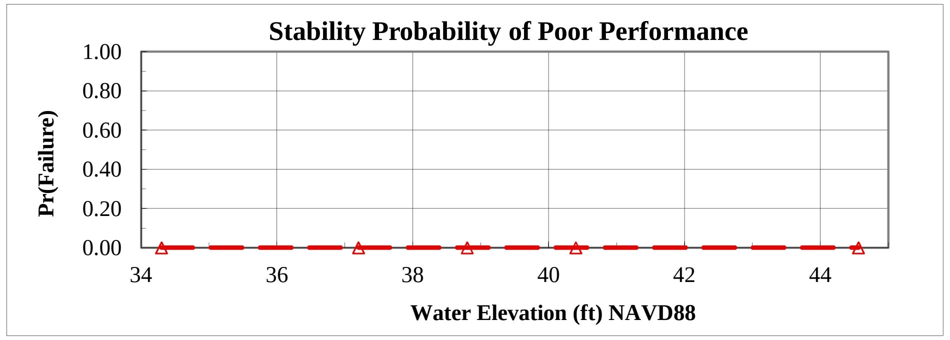
Project: Lower San Joaquin
Study Area: Left Bank Stockton Diverting Canal
River Section: Index Point SL-2
Coordinates: State Plane (ft), N 2176913, E 6352470

Levee Mile: STA 976+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 44.56
L/S Toe Elev.: 34.30
W/S Toe Elev.: 34.79

Analysis By: J. Hogan
Checked By: M. Perlea, G. Johnson
Date: 9/27/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	31	4	13.00
Levee Cohesion	150	60	40.00
Levee γ	115	8	7.00
Foundation Φ	31	4	13.00
Foundation Cohesion	150	60	40.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	34.30	0.0000
200yr	2.90	37.20	0.000000
200yr + 3ft	4.50	38.80	0.000000
Crest-3ft	6.10	40.40	0.000000
Crest	10.26	44.56	0.000000

Crest	**Head =**	10.26	**Pr(f)=0**	NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	31	150	115	31	150	1.68		
2	27	150	115	31	150	1.66	0.000576	4.05
3	35	150	115	31	150	1.71		
4	31	90	115	31	150	1.68	0.000009	0.06
5	31	210	115	31	150	1.69		
6	31	150	107	31	150	1.67	0.000256	1.80
7	31	150	123	31	150	1.70		
8	31	150	115	27	150	1.55	0.013225	93.07
9	31	150	115	35	150	1.78		
10	31	150	115	31	90	1.67	0.000144	1.01
11	31	150	115	31	210	1.69		

E[FS] = 1.682000 E[ln FS] = 0.517478 Total 0.014210 100.00
Var[FS]= 0.014210
σ[FS]= 0.119206 σ[ln FS]= 0.070783 β = 7.310809
V(FS) = 0.070871 F(z) = 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Crest-3ft	**Head =**	6.10	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	31	150	115	31	150	1.13		
2	27	150	115	31	150	1.02	0.008372	28.40
3	35	150	115	31	150	1.21		
4	31	90	115	31	150	1.08	0.004225	14.33
5	31	210	115	31	150	1.21		
6	31	150	107	31	150	1.13	0.002704	9.17
7	31	150	123	31	150	1.23		
8	31	150	115	27	150	1.10	0.008556	29.02
9	31	150	115	35	150	1.28		
10	31	150	115	31	90	1.08	0.005625	19.08
11	31	150	115	31	210	1.23		

E[FS] = E[ln FS] = Total 0.029483 100.00
Var[FS]= 0.029483
σ[FS]= 0.171705 σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

200yr + 3ft	**Head =**	4.50	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	31	150	115	31	150	1.27		
2	27	150	115	31	150	1.12	0.012769	79.39
3	35	150	115	31	150	1.34		
4	31	90	115	31	150	1.19	0.001156	7.19
5	31	210	115	31	150	1.26		
6	31	150	107	31	150	1.22	0.000016	0.10
7	31	150	123	31	150	1.23		
8	31	150	115	27	150	1.20	0.001722	10.71
9	31	150	115	35	150	1.29		
10	31	150	115	31	90	1.21	0.000420	2.61
11	31	150	115	31	210	1.25		

E[FS] = E[ln FS] = Total 0.016084 100.00
Var[FS]= 0.016084
σ[FS]= 0.126821 σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

200yr	**Head =**	2.90	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	31	150	115	31	150	1.30		
2	27	150	115	31	150			
3	35	150	115	31	150			
4	31	90	115	31	150			
5	31	210	115	31	150			
6	31	150	107	31	150			
7	31	150	123	31	150			
8	31	150	115	27	150			
9	31	150	115	35	150			
10	31	150	115	31	90			
11	31	150	115	31	210			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= β =
V(FS) = F(z) =
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

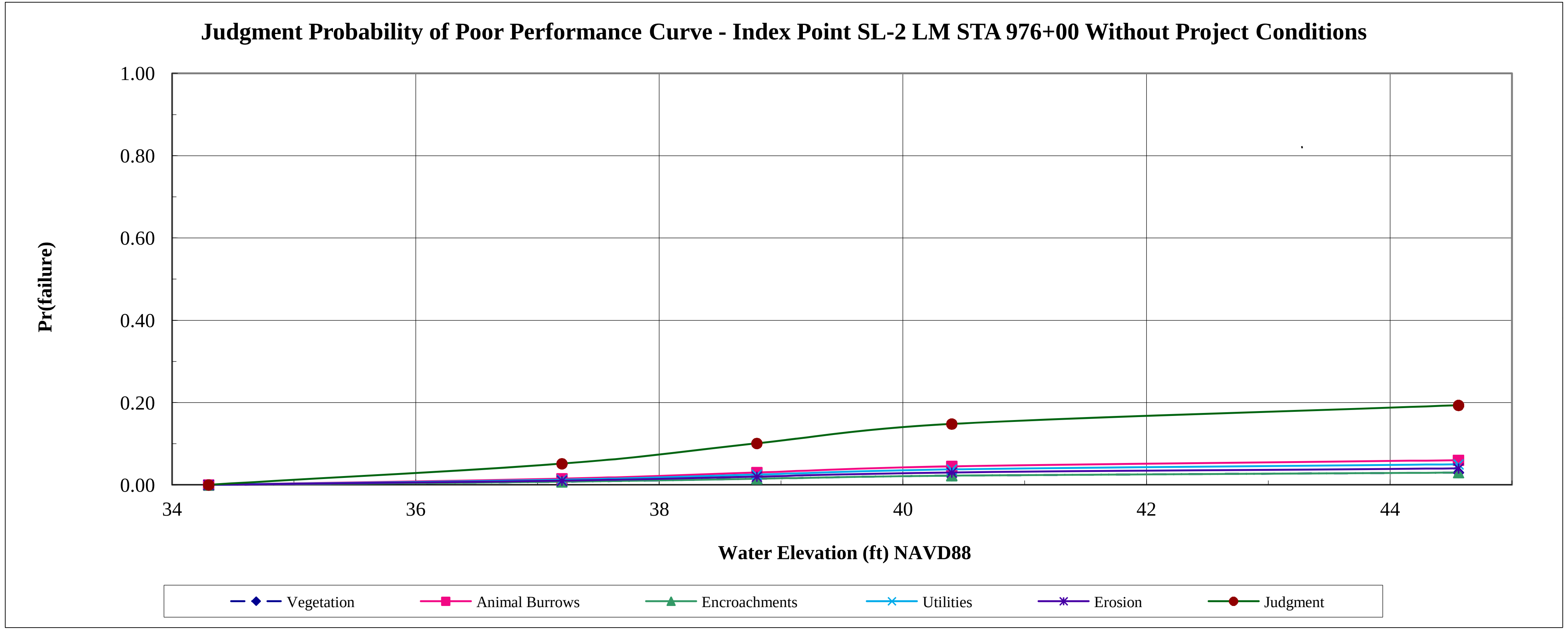
Project: Lower San Joaquin
Study Area: Left Bank Stockton Diverting Canal
River Section: Index Point SL-2
Coordinates: State Plane (ft), N 2176913, E 6352470

Levee Mile: STA 976+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Datum: NAVD 88
Crest Elev.: 44.56
L/S Toe Elev.: 34.30
W/S Toe Elev.: 34.79

Analysis By: J. Hogan
Checked By: M. Perlea, G. John
Date: 9/27/2012

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
34.30	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
37.20	0.0075	0.9925	0.0150	0.9850	0.0075	0.9925	0.0125	0.9875	0.0100	0.9900	0.0514	0.9486
38.80	0.0150	0.9850	0.0300	0.9700	0.0150	0.9850	0.0250	0.9750	0.0200	0.9800	0.1008	0.8992
40.40	0.0225	0.9775	0.0450	0.9550	0.0225	0.9775	0.0375	0.9625	0.0300	0.9700	0.1481	0.8519
44.56	0.0300	0.9700	0.0600	0.9400	0.0300	0.9700	0.0500	0.9500	0.0400	0.9600	0.1934	0.8066



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

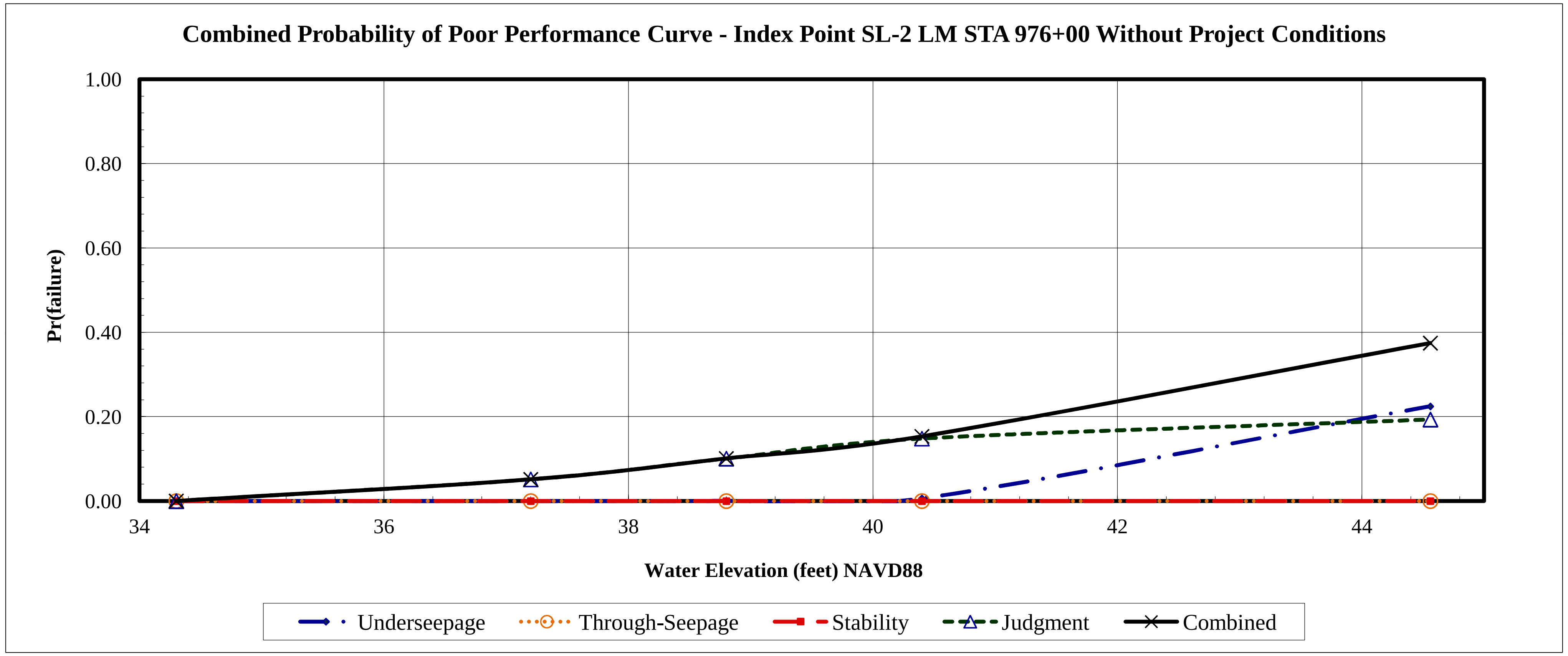
Project: Lower San Joaquin
Study Area: Left Bank Stockton Diverting Canal
River Section: Index Point SL-2
Coordinates: State Plane (ft), N 2176913, E 6352470

Levee Mile: STA 976+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Datum: NAVD 88
Crest Elev.: 44.56
L/S Toe Elev.: 34.30
W/S Toe Elev.: 34.79

Analysis By: J. Hogan
Checked By: M. Perlea, G. Joh
Date: 9/27/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
34.30	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
37.20	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0514	0.9486	0.0514	0.9486
38.80	0.0002	0.9998	0.0000	1.0000	0.0000	1.0000	0.1008	0.8992	0.1009	0.8991
40.40	0.0062	0.9938	0.0000	1.0000	0.0000	1.0000	0.1481	0.8519	0.1533	0.8467
44.56	0.2245	0.7755	0.0000	1.0000	0.0000	1.0000	0.1934	0.8066	0.3745	0.6255


Combined Probability of Poor Performance Curve - Index Point SL-2 LM STA 976+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: Left Bank Calaveras River
Basin and Reach: CL1

Levee Mile: STA 6757+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 31.43
L/S Toe Elev.: 21.00
W/S Toe Elev.: 26.94

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/24/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket		Aquifer Material		Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
											Material	Kb (ft/day)	Material	Kf (ft/day)					
WR1614_014C	11	19	8	119	42	7	15	11	137	73	ML	0.07	SM	2.8	40	40	0	373	0
WCSBCR_001B	10					6.5					CL	0.007	ML	0.28	40				
WCSBCR_003B	29					26					ML	0.07	SP-SM	2.8	40				
WCSBCR_003A	26					24					ML	0.07	SP-SM	2.8	40				
WCSBCR_006C	20					30					ML	0.07	SP-SM	2.8	40				
WCSBCR_008C	13					5					ML	0.07	SP-SM	2.8	40				
WCSBCR_004B	22					4					ML	0.007	OH	0.28	40				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WR1614_014C	ML	11	0.07				11	SM	7	2.8							2.8
WCSBCR_001B	CL	10	0.007				10	ML	6.5	0.28							0.28
WCSBCR_003B	ML	29	0.07				29	SP-SM	26	2.8							2.8
WCSBCR_003A	ML	26	0.07				26	SP-SM	24	2.8							2.8
WCSBCR_006C	ML	20	0.07				20	SP-SM	30	2.8							2.8
WCSBCR_008C	ML	13	0.07				13	SP-SM	5	2.8							2.8
WCSBCR_004B	ML	22	0.007				22	OH	4	0.28							0.28

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: CL1

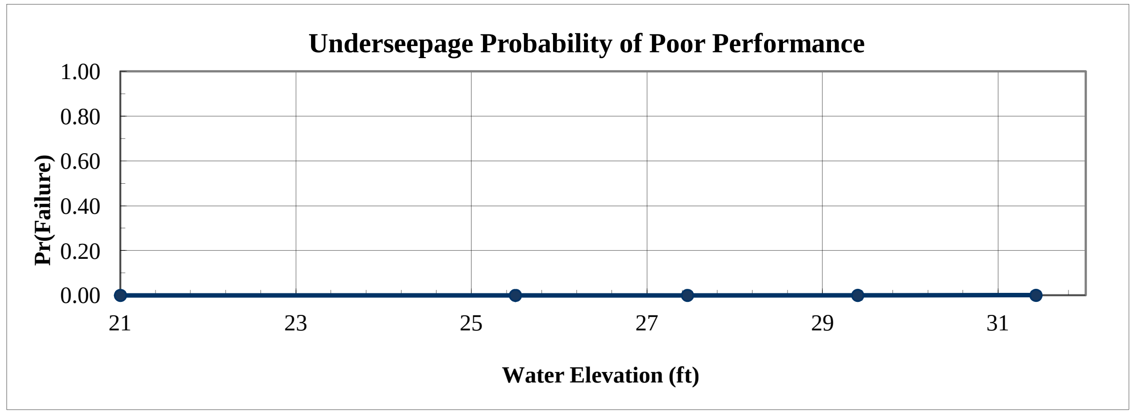
Levee Mile: STA 6757+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 31.43
L/S Toe Elev.: 21.00
W/S Toe Elev.: 26.94

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/24/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permaebility Ratio	40	0	0
Blanket Thickness (z)	19	8	42
Aquifer Thickness (d)	15	11	73

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	158	61	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	21.00	0.0000
200yr	4.50	25.50	0.0000
200yr+2ft	6.46	27.46	0.0000
Crest-2ft	8.40	29.40	0.0001
Crest	10.43	31.43	0.0004

Crest	Rh
Head = 10.43	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	40	19.00	15.00	96.25	106.77	0.0568	4.22	0.22		
2	40	19.00	15.00	96.25	106.77	0.0568	4.22	0.22	0.000000	0.00
3	40	19.00	15.00	96.25	106.77	0.0568	4.22	0.22		
4	40	27.00	15.00	107.66	127.28	0.0507	4.49	0.17	0.008100	90.00
5	40	11.00	15.00	77.98	81.24	0.0681	3.85	0.35		
6	40	19.00	26.00	113.71	140.57	0.0825	4.65	0.24	0.000900	10.00
7	40	19.00	4.00	54.78	55.14	0.0234	3.36	0.18		
								Total	0.009000	100.00

E[I] = 0.220000
Var[I]= 0.009000
σ[I]= 0.094868
V(I) = 0.431220
Ic= 0.80

E[ln I] = -1.599400
σ [ln I] = 0.412970
ln(I crit) = -0.223144

β = -3.872917
F(z) = 0.999570
Pr(f) % = 0.043022

Crest-2ft	Rh
Head = 8.40	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	40	19.00	15.00	96.25	106.77	0.0568	3.40	0.18		
2	40	19.00	15.00	96.25	106.77	0.0568	3.40	0.18	0.000000	0.00
3	40	19.00	15.00	96.25	106.77	0.0568	3.40	0.18		
4	40	27.00	15.00	107.66	127.28	0.0507	3.61	0.13	0.005625	86.21
5	40	11.00	15.00	77.98	81.24	0.0681	3.10	0.28		
6	40	19.00	26.00	113.71	140.57	0.0825	3.75	0.20	0.000900	13.79
7	40	19.00	4.00	54.78	55.14	0.0234	2.71	0.14		
								Total	0.006525	100.00

E[I] = 0.180000
Var[I]= 0.006525
σ[I]= 0.080777
V(I) = 0.448764
Ic= 0.80

E[ln I] = -1.806538
σ [ln I] = 0.428344
ln(I crit) = -0.223144

β = -4.217496
F(z) = 0.999891
Pr(f) % = 0.010927

200yr+2ft	Rh
Head = 6.46	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	40	19.00	15.00	96.25	106.77	0.0568	2.61	0.14		
2	40	19.00	15.00	96.25	106.77	0.0568	2.61	0.14	0.000000	0.00
3	40	19.00	15.00	96.25	106.77	0.0568	2.61	0.14		
4	40	27.00	15.00	107.66	127.28	0.0507	2.78	0.10	0.003600	90.00
5	40	11.00	15.00	77.98	81.24	0.0681	2.38	0.22		
6	40	19.00	26.00	113.71	140.57	0.0825	2.88	0.15	0.000400	10.00
7	40	19.00	4.00	54.78	55.14	0.0234	2.08	0.11		
								Total	0.004000	100.00

E[I] = 0.140000
Var[I]= 0.004000
σ[I]= 0.063246
V(I) = 0.451754
Ic= 0.80

E[ln I] = -2.058971
σ [ln I] = 0.430949
ln(I crit) = -0.223144

β = -4.777760
F(z) = 0.999990
Pr(f) % = 0.001022

200yr	Rh
Head = 4.50	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	40	19.00	15.00	96.25	106.77	0.0568	1.82	0.10		
2	40	19.00	15.00	96.25	106.77	0.0568	1.82	0.10	0.000000	0.00
3	40	19.00	15.00	96.25	106.77	0.0568	1.82	0.10		
4	40	27.00	15.00	107.66	127.28	0.0507	1.94	0.07	0.001600	87.67
5	40	11.00	15.00	77.98	81.24	0.0681	1.66	0.15		
6	40	19.00	26.00	113.71	140.57	0.0825	2.01	0.11	0.000225	12.33
7	40	19.00	4.00	54.78	55.14	0.0234	1.45	0.08		
								Total	0.001825	100.00

E[I] = 0.100000
Var[I]= 0.001825
σ[I]= 0.042720
V(I) = 0.427200
Ic= 0.80

E[ln I] = -2.386401
σ [ln I] = 0.409427
ln(I crit) = -0.223144

β = -5.828628
F(z) = 1.000000
Pr(f) % = 0.000006

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: CL1

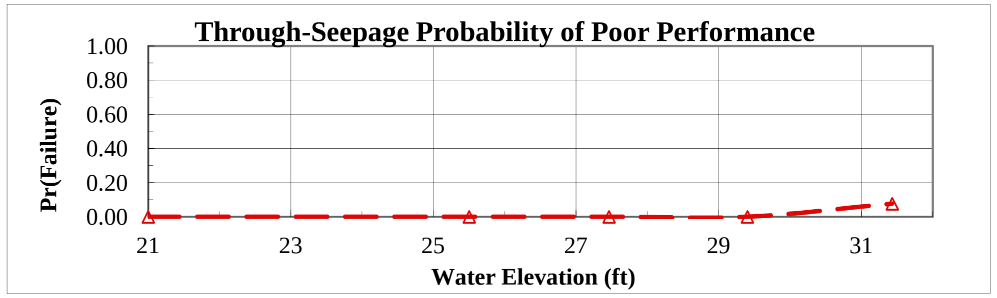
Levee Mile: STA 6757+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 31.43
L/S Toe Elev.: 21.00
W/S Toe Elev.: 26.94

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/24/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	3.2	0.3	10.00
Initial Porosity (n)	0.39	0.04	10.00
Initial Permeability (Ko)	2.00E-06	6.00E-07	30.00

Pr(f)=0
NO



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	21.00	0.0000
200yr	4.50	25.50	0.000000
200yr+2ft	6.46	27.46	0.000003
Crest-2ft	8.40	29.40	0.000010
Crest	10.43	31.43	0.076943

Crest	**Head =**	10.43	**Horizontal Gradient (Ix) =**	0.380

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	3.20	0.39	2.00E-06	0.51	1.34		
2	2.88	0.39	2.00E-06	0.46	1.21	0.017978	26.44
3	3.52	0.39	2.00E-06	0.56	1.47		
4	3.20	0.35	2.00E-06	0.48	1.27	0.004506	6.63
5	3.20	0.43	2.00E-06	0.53	1.41		
6	3.20	0.39	1.40E-06	0.61	1.60	0.045498	66.93
7	3.20	0.39	2.60E-06	0.45	1.18		

E[FS] =	1.340813	E[ln FS] =	0.274717	Total	0.067982		100.00
Var[FS]=	0.067982						
σ[FS]=	0.260733	σ[ln FS]=	0.192658			**β =**	1.425936
V(FS) =	0.194459					**F(z) =**	0.076943
FS req'd =	1.00	ln(FS req'd) =	0.000000			**Pr(f) % =**	7.694347

Crest-2ft	**Head =**	8.40	**Horizontal Gradient (Ix) =**	0.220

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	3.20	0.39	2.00E-06	0.51	2.32		
2	2.88	0.39	2.00E-06	0.46	2.08	0.053636	26.44
3	3.52	0.39	2.00E-06	0.56	2.55		
4	3.20	0.35	2.00E-06	0.48	2.20	0.013443	6.63
5	3.20	0.43	2.00E-06	0.53	2.43		
6	3.20	0.39	1.40E-06	0.61	2.77	0.135743	66.93
7	3.20	0.39	2.60E-06	0.45	2.03		

E[FS] =	2.315949	E[ln FS] =	0.821261	Total	0.202822		100.00
Var[FS]=	0.202822						
σ[FS]=	0.450358	σ[ln FS]=	0.192658			**β =**	4.262800
V(FS) =	0.194459					**F(z) =**	0.000010
FS req'd =	1.00	ln(FS req'd) =	0.000000			**Pr(f) % =**	0.001009

200yr+2ft	**Head =**	6.46	**Horizontal Gradient (Ix) =**	0.210

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	3.20	0.39	2.00E-06	0.51	2.43		
2	2.88	0.39	2.00E-06	0.46	2.18	0.058866	26.44
3	3.52	0.39	2.00E-06	0.56	2.67		
4	3.20	0.35	2.00E-06	0.48	2.30	0.014753	6.63
5	3.20	0.43	2.00E-06	0.53	2.54		
6	3.20	0.39	1.40E-06	0.61	2.90	0.148979	66.93
7	3.20	0.39	2.60E-06	0.45	2.13		

E[FS] =	2.426232	E[ln FS] =	0.867781	Total	0.222598		100.00
Var[FS]=	0.222598						
σ[FS]=	0.471803	σ[ln FS]=	0.192658			**β =**	4.504265
V(FS) =	0.194459					**F(z) =**	0.000003
FS req'd =	1.00	ln(FS req'd) =	0.000000			**Pr(f) % =**	0.000333

200yr	**Head =**	4.50	**Horizontal Gradient (Ix) =**	0.130

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	3.20	0.39	2.00E-06	0.51	3.92		
2	2.88	0.39	2.00E-06	0.46	3.53	0.153609	26.44
3	3.52	0.39	2.00E-06	0.56	4.31		
4	3.20	0.35	2.00E-06	0.48	3.72	0.038499	6.63
5	3.20	0.43	2.00E-06	0.53	4.11		
6	3.20	0.39	1.40E-06	0.61	4.68	0.388756	66.93
7	3.20	0.39	2.60E-06	0.45	3.44		

E[FS] =	3.919299	E[ln FS] =	1.347354	Total	0.580863		100.00
Var[FS]=	0.580863						
σ[FS]=	0.762144	σ[ln FS]=	0.192658			**β =**	6.993515
V(FS) =	0.194459					**F(z) =**	0.000000
FS req'd =	1.00	ln(FS req'd) =	0.000000			**Pr(f) % =**	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

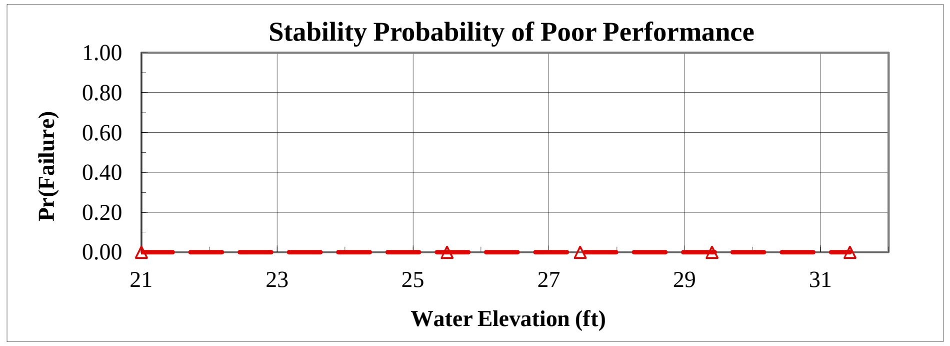
Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: CL1

Levee Mile: STA 6757+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 31.43
L/S Toe Elev.: 21.00
W/S Toe Elev.: 26.94

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/24/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	34	4	13.00
Levee Cohesion	100	40	40.00
Levee γ	115	8	7.00
Foundation Φ	31	4	13.00
Foundation Cohesion	150	60	40.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	21.00	0.0000
200yr	4.50	25.50	0.000000
200yr+2ft	6.46	27.46	0.000000
Crest-2ft	8.40	29.40	0.000000
Crest	10.43	31.43	0.000109

Crest	**Head =**	10.43	**Pr(f)=0**	NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	115	31	150	2.05		
2	30	100	115	31	150	1.88	0.039800	25.96
3	38	100	115	31	150	2.28		
4	34	60	115	31	150	1.70	0.109230	71.25
5	34	140	115	31	150	2.36		
6	34	100	107	31	150	2.05	0.000009	0.01
7	34	100	123	31	150	2.05		
8	34	100	115	27	150	2.10	0.000552	0.36
9	34	100	115	35	150	2.05		
10	34	100	115	31	90	1.93	0.003721	2.43
11	34	100	115	31	210	2.05		

E[FS] = 2.050000 E[ln FS] = 0.699924 Total 0.153313 100.00
Var[FS]= 0.153313
σ[FS]= 0.391552 σ[ln FS]= 0.189292
V(FS) = 0.191001
FS req'd = 1.00 ln(FS req'd) = 0.000000

β =	3.697580
F(z) =	0.000109
Pr(f) % =	0.010883

Crest-2ft	**Head =**	8.40	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	115	31	150	2.28		
2	30	100	115	31	150			
3	38	100	115	31	150			
4	34	60	115	31	150			
5	34	140	115	31	150			
6	34	100	107	31	150			
7	34	100	123	31	150			
8	34	100	115	27	150			
9	34	100	115	35	150			
10	34	100	115	31	90			
11	34	100	115	31	210			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]=
V(FS) =
FS req'd = 1.00 ln(FS req'd) = 0.000000

β =	
F(z) =	
Pr(f) % =	0.000000

200yr+2ft	**Head =**	6.46	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	115	31	150	2.46		
2	30	100	115	31	150			
3	38	100	115	31	150			
4	34	60	115	31	150			
5	34	140	115	31	150			
6	34	100	107	31	150			
7	34	100	123	31	150			
8	34	100	115	27	150			
9	34	100	115	35	150			
10	34	100	115	31	90			
11	34	100	115	31	210			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]=
V(FS) =
FS req'd = 1.00 ln(FS req'd) = 0.000000

β =	
F(z) =	
Pr(f) % =	0.000000

200yr	**Head =**	4.50	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	115	31	150	2.71		
2	30	100	115	31	150			
3	38	100	115	31	150			
4	34	60	115	31	150			
5	34	140	115	31	150			
6	34	100	107	31	150			
7	34	100	123	31	150			
8	34	100	115	27	150			
9	34	100	115	35	150			
10	34	100	115	31	90			
11	34	100	115	31	210			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]=
V(FS) =
FS req'd = 1.00 ln(FS req'd) = 0.000000

β =	
F(z) =	
Pr(f) % =	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

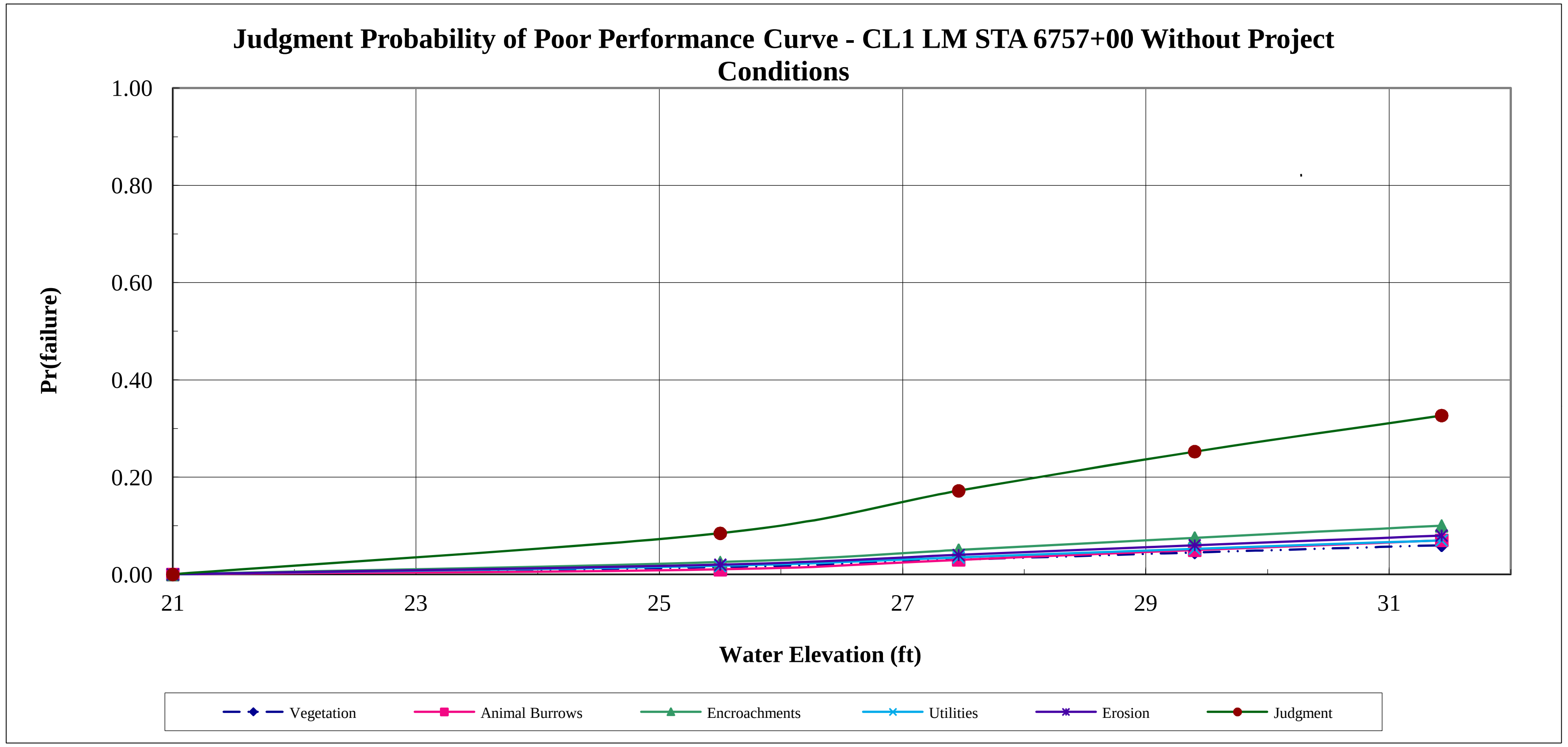
Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: CL1

Levee Mile: STA 6757+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 31.43
L/S Toe Elev.: 21.00
W/S Toe Elev.: 26.94

Analysis By: G. Johnson
Checked By: M. Perlea, J. F
Date: 9/24/2012

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
21.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
25.50	0.0150	0.9850	0.0100	0.9900	0.0250	0.9750	0.0175	0.9825	0.0200	0.9800	0.0845	0.9155
27.46	0.0300	0.9700	0.0300	0.9700	0.0500	0.9500	0.0350	0.9650	0.0400	0.9600	0.1719	0.8281
29.40	0.0450	0.9550	0.0500	0.9500	0.0750	0.9250	0.0525	0.9475	0.0600	0.9400	0.2526	0.7474
31.43	0.0600	0.9400	0.0700	0.9300	0.1000	0.9000	0.0700	0.9300	0.0800	0.9200	0.3268	0.6732



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

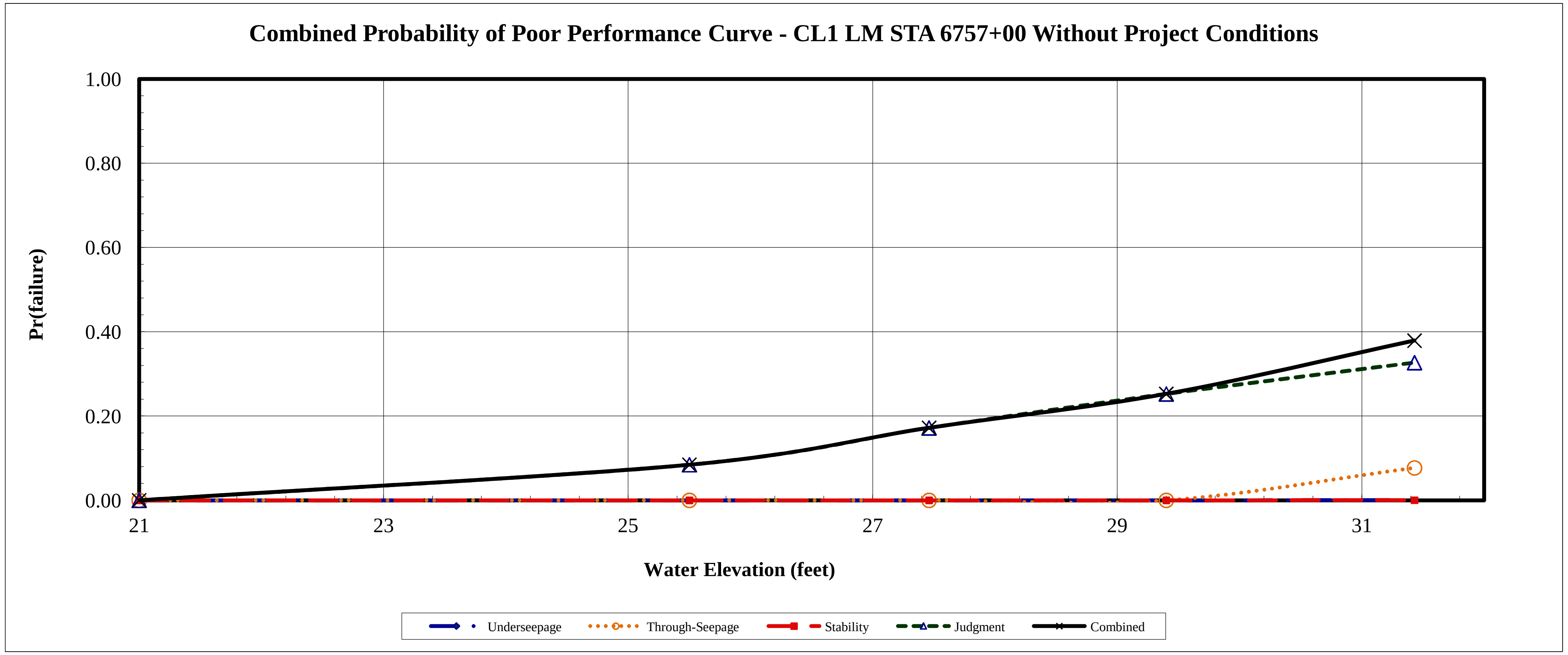
Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: CL1

Levee Mile: STA 6757+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 31.43
L/S Toe Elev.: 21.00
W/S Toe Elev.: 26.94

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hog
Date: 9/24/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
21.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
25.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0845	0.9155	0.0845	0.9155
27.46	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1719	0.8281	0.1719	0.8281
29.40	0.0001	0.9999	0.0000	1.0000	0.0000	1.0000	0.2526	0.7474	0.2527	0.7473
31.43	0.0004	0.9996	0.0769	0.9231	0.0001	0.9999	0.3268	0.6732	0.3790	0.6210



Combined Probability of Poor Performance Curve - CL1 LM STA 6757+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: Right Bank Calaveras River
Basin and Reach: Index Point CR1

Levee Mile: STA 3306+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 29.66
L/S Toe Elev.: 23.80
W/S Toe Elev.: 22.90

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/28/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material Material	Aquifer Material Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
WCNBCR_006A	8	5	2	8	40	26	14	8	86	57	CL	0.007	ML	1.4	200	200	0	11111	0
WCNBCR_007B	4					12					CL	0.007	ML	1.4	200				
WCNBCR_013C	4					14					CL	0.007	ML	1.4	200				
WCNBCR_008B	5					4					CL	0.007	ML	1.4	200				
WCNBCR_010A	2					16					CL	0.007	ML	1.4	200				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WCNBCR_006A	CL	8	0.007				8	ML	26	1.4							1.4
WCNBCR_007B	CL	4	0.007				4	ML	12	1.4							1.4
WCNBCR_013C	CL	4	0.007				4	ML	14	1.4							1.4
WCNBCR_008B	CL	5	0.007				5	ML	4	1.4							1.4
WCNBCR_010A	CL	2	0.007				2	ML	16	1.4							1.4

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point CR1

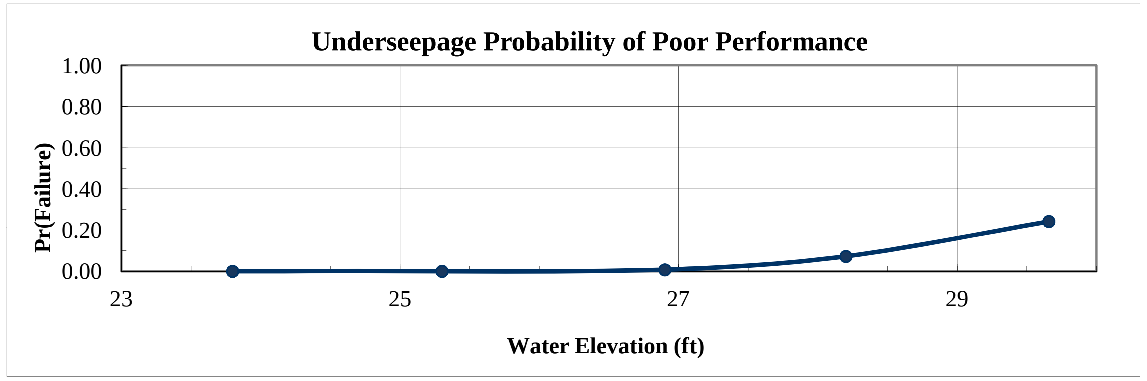
Levee Mile: STA 3306+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 29.66
L/S Toe Elev.: 23.80
W/S Toe Elev.: 22.90

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/28/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permaebility Ratio	200	0	0
Blanket Thickness (z)	5	2	40
Aquifer Thickness (d)	14	8	57

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	37	56	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	23.80	0.0000
Elev. 25.3	1.50	25.30	0.0000
200 yr	3.10	26.90	0.0074
Elev. 28.2	4.40	28.20	0.0727
Crest	5.86	29.66	0.2418

Crest	**Rh**
Head = 5.86	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	200	5.00	14.00	35.84	118.32	0.0666	3.30	0.66		
2	200	5.00	14.00	35.84	118.32	0.0666	3.30	0.66	0.000000	0.00
3	200	5.00	14.00	35.84	118.32	0.0666	3.30	0.66		
4	200	7.00	14.00	36.16	140.00	0.0603	3.53	0.50	0.057600	87.67
5	200	3.00	14.00	35.11	91.65	0.0766	2.94	0.98		
6	200	5.00	22.00	36.25	148.32	0.0914	3.61	0.72	0.008100	12.33
7	200	5.00	6.00	34.42	77.46	0.0357	2.70	0.54		
								Total	0.065700	100.00

E[I] = 0.660000
Var[I]= 0.065700
σ[I]= 0.256320
V(I) = 0.388364

E[ln I] = -0.485756
σ [ln I] = 0.374807

Ic= 0.80
ln(I crit) = -0.223144

β = -1.296015
F(z) = 0.758242
Pr(f) % = 24.175783

Elev. 28.2	**Rh**
Head = 4.40	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	200	5.00	14.00	35.84	118.32	0.0666	2.48	0.50		
2	200	5.00	14.00	35.84	118.32	0.0666	2.48	0.50	0.000000	0.00
3	200	5.00	14.00	35.84	118.32	0.0666	2.48	0.50		
4	200	7.00	14.00	36.16	140.00	0.0603	2.66	0.38	0.032400	88.46
5	200	3.00	14.00	35.11	91.65	0.0766	2.21	0.74		
6	200	5.00	22.00	36.25	148.32	0.0914	2.72	0.54	0.004225	11.54
7	200	5.00	6.00	34.42	77.46	0.0357	2.03	0.41		
								Total	0.036625	100.00

E[I] = 0.500000
Var[I]= 0.036625
σ[I]= 0.191377
V(I) = 0.382753

E[ln I] = -0.761504
σ [ln I] = 0.369748

Ic= 0.80
ln(I crit) = -0.223144

β = -2.059520
F(z) = 0.927306
Pr(f) % = 7.269370

200 yr	**Rh**
Head = 3.10	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	200	5.00	14.00	35.84	118.32	0.0666	1.75	0.35		
2	200	5.00	14.00	35.84	118.32	0.0666	1.75	0.35	0.000000	0.00
3	200	5.00	14.00	35.84	118.32	0.0666	1.75	0.35		
4	200	7.00	14.00	36.16	140.00	0.0603	1.87	0.27	0.015625	88.53
5	200	3.00	14.00	35.11	91.65	0.0766	1.56	0.52		
6	200	5.00	22.00	36.25	148.32	0.0914	1.91	0.38	0.002025	11.47
7	200	5.00	6.00	34.42	77.46	0.0357	1.43	0.29		
								Total	0.017650	100.00

E[I] = 0.350000
Var[I]= 0.017650
σ[I]= 0.132853
V(I) = 0.379581

E[ln I] = -1.117123
σ [ln I] = 0.366882

Ic= 0.80
ln(I crit) = -0.223144

β = -3.044913
F(z) = 0.992589
Pr(f) % = 0.741105

Elev. 25.3	**Rh**
Head = 1.50	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	200	5.00	14.00	35.84	118.32	0.0666	0.85	0.17		
2	200	5.00	14.00	35.84	118.32	0.0666	0.85	0.17	0.000000	0.00
3	200	5.00	14.00	35.84	118.32	0.0666	0.85	0.17		
4	200	7.00	14.00	36.16	140.00	0.0603	0.91	0.13	0.003600	85.21
5	200	3.00	14.00	35.11	91.65	0.0766	0.75	0.25		
6	200	5.00	22.00	36.25	148.32	0.0914	0.93	0.19	0.000625	14.79
7	200	5.00	6.00	34.42	77.46	0.0357	0.69	0.14		
								Total	0.004225	100.00

E[I] = 0.170000
Var[I]= 0.004225
σ[I]= 0.065000
V(I) = 0.382353

E[ln I] = -1.840180
σ [ln I] = 0.369387

Ic= 0.80
ln(I crit) = -0.223144

β = -4.981715
F(z) = 0.999994
Pr(f) % = 0.000600

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point CR1

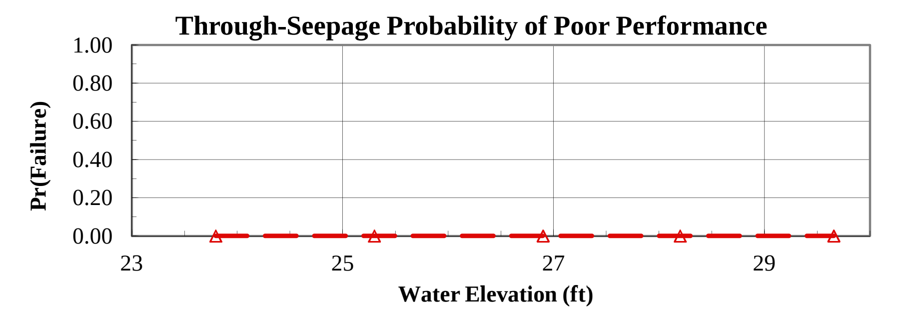
Levee Mile: STA 3306+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 29.66
L/S Toe Elev.: 23.80
W/S Toe Elev.: 22.90

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/28/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	8	0.8	10.00
Initial Porosity (n)	50	5.00	10.00
Initial Permeability (Ko)	2.80E-08	8.40E-09	30.00

Pr(f)=0
NO



Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	23.80	0.0000
Elev. 25.3	1.50	25.30	0.000000
200 yr	3.10	26.90	0.000000
Elev. 28.2	4.40	28.20	0.000000
Crest	5.86	29.66	0.000000

Crest **Head =** 5.86 **Horizontal Gradient (Ix) =** 0.220

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	8.00	50.00	2.80E-08	121.89	554.06		
2	7.20	50.00	2.80E-08	109.70	498.66	3069.837765	26.44
3	8.80	50.00	2.80E-08	134.08	609.47		
4	8.00	45.00	2.80E-08	115.64	525.63	769.387744	6.63
5	8.00	55.00	2.80E-08	127.84	581.10		
6	8.00	50.00	1.96E-08	145.69	662.23	7769.185658	66.93
7	8.00	50.00	3.64E-08	106.91	485.94		

E[FS] =	554.061167	E[ln FS] =	6.298717	Total	11608.411167		100.00
Var[FS]=	11608.411167						
σ[FS]=	107.742337	σ[ln FS]=	0.192658			**β =**	32.693828
V(FS) =	0.194459					**F(z) =**	0.000000
FS req'd =	1.00	ln(FS req'd) =	0.000000			**Pr(f) % =**	0.000000

Elev. 28.2 **Head =** 4.40 **Horizontal Gradient (Ix) =** 0.180

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	8.00	50.00	2.80E-08	121.89	677.19		
2	7.20	50.00	2.80E-08	109.70	609.47	4585.807032	26.44
3	8.80	50.00	2.80E-08	134.08	744.90		
4	8.00	45.00	2.80E-08	115.64	642.43	1149.332308	6.63
5	8.00	55.00	2.80E-08	127.84	710.24		
6	8.00	50.00	1.96E-08	145.69	809.39	11605.820552	66.93
7	8.00	50.00	3.64E-08	106.91	593.93		

E[FS] =	677.185870	E[ln FS] =	6.499387	Total	17340.959892		100.00
Var[FS]=	17340.959892						
σ[FS]=	131.685078	σ[ln FS]=	0.192658			**β =**	33.735420
V(FS) =	0.194459					**F(z) =**	0.000000
FS req'd =	1.00	ln(FS req'd) =	0.000000			**Pr(f) % =**	0.000000

200 yr **Head =** 3.10 **Horizontal Gradient (Ix) =** 0.140

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	8.00	50.00	2.80E-08	121.89	870.67		
2	7.20	50.00	2.80E-08	109.70	783.60	7580.619787	26.44
3	8.80	50.00	2.80E-08	134.08	957.73		
4	8.00	45.00	2.80E-08	115.64	825.99	1899.916673	6.63
5	8.00	55.00	2.80E-08	127.84	913.16		
6	8.00	50.00	1.96E-08	145.69	1040.65	19185.131932	66.93
7	8.00	50.00	3.64E-08	106.91	763.63		

E[FS] =	870.667548	E[ln FS] =	6.750702	Total	28665.668392		100.00
Var[FS]=	28665.668392						
σ[FS]=	169.309387	σ[ln FS]=	0.192658			**β =**	35.039882
V(FS) =	0.194459					**F(z) =**	0.000000
FS req'd =	1.00	ln(FS req'd) =	0.000000			**Pr(f) % =**	0.000000

Elev. 25.3 **Head =** 1.50 **Horizontal Gradient (Ix) =** 0.010

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	8.00	50.00	2.80E-08	121.89	12189.35		
2	7.20	50.00	2.80E-08	109.70	10970.41	1485801.478345	26.44
3	8.80	50.00	2.80E-08	134.08	13408.28		
4	8.00	45.00	2.80E-08	115.64	11563.83	372383.667864	6.63
5	8.00	55.00	2.80E-08	127.84	12784.29		
6	8.00	50.00	1.96E-08	145.69	14569.05	3760285.858706	66.93
7	8.00	50.00	3.64E-08	106.91	10690.76		

E[FS] =	12189.345669	E[ln FS] =	9.389759	Total	5618471.004916		100.00
Var[FS]=	5618471.004916						
σ[FS]=	2370.331412	σ[ln FS]=	0.192658			**β =**	48.738051
V(FS) =	0.194459					**F(z) =**	0.000000
FS req'd =	1.00	ln(FS req'd) =	0.000000			**Pr(f) % =**	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

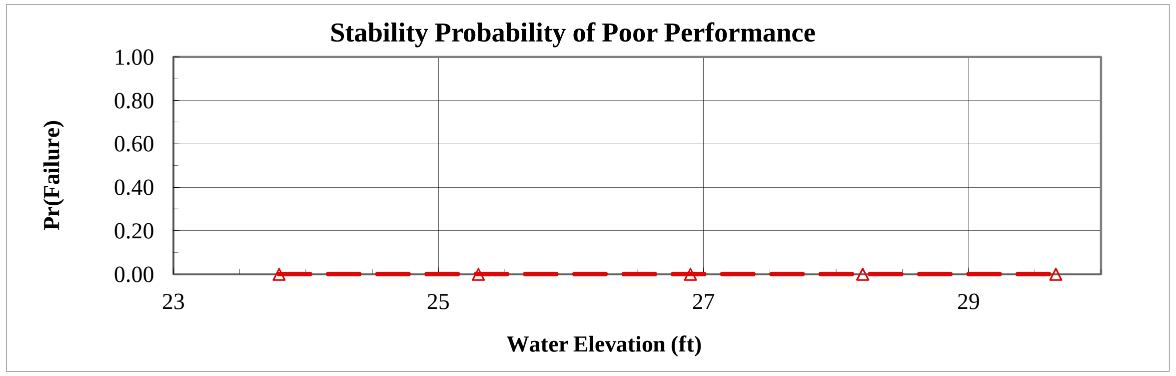
Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point CR1

Levee Mile: STA 3306+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 29.66
L/S Toe Elev.: 23.80
W/S Toe Elev.: 22.90

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/28/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	34	4	13.00
Levee Cohesion	100	40	40.00
Levee γ	120	8	7.00
Foundation Φ	31	4	13.00
Foundation Cohesion	150	60	40.00


Stability Probability of Poor Performance

Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	23.80	0.0000
Elev. 25.3	1.50	25.30	0.000000
200 yr	3.10	26.90	0.000000
Elev. 28.2	4.40	28.20	0.000000
Crest	5.86	29.66	0.000000

Crest	**Head =**	5.86	**Pr(f)=0**	NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	120	31	150	2.91		
2	30	100	120	31	150	2.67	0.022052	8.22
3	38	100	120	31	150	2.97		
4	34	60	120	31	150	2.31	0.189660	70.73
5	34	140	120	31	150	3.18		
6	34	100	112	31	150	2.82	0.000225	0.08
7	34	100	128	31	150	2.85		
8	34	100	120	27	150	2.71	0.008464	3.16
9	34	100	120	35	150	2.89		
10	34	100	120	31	90	2.45	0.047742	17.80
11	34	100	120	31	210	2.89		

E[FS] = 2.910000
Var[FS]= 0.268144
σ[FS]= 0.517826
V(FS) = 0.177947
FS req'd = 1.00

E[ln FS] = 1.052566
σ[ln FS]= 0.176562
ln(FS req'd) = 0.000000

Total 0.268144 100.00

β = 5.961451
F(z) = 0.000000
Pr(f) % = 0.000000

Elev. 28.2	**Head =**	4.40	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	120	31	150	3.13		
2	30	100	120	31	150			
3	38	100	120	31	150			
4	34	60	120	31	150			
5	34	140	120	31	150			
6	34	100	112	31	150			
7	34	100	128	31	150			
8	34	100	120	27	150			
9	34	100	120	35	150			
10	34	100	120	31	90			
11	34	100	120	31	210			

E[FS] =
Var[FS]=
σ[FS]=
V(FS) =
FS req'd = 1.00

E[ln FS] =
σ[ln FS]=
ln(FS req'd) = 0.000000

Total

β =
F(z) =
Pr(f) % = 0.000000

200 yr	**Head =**	3.10	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	120	31	150	3.37		
2	30	100	120	31	150			
3	38	100	120	31	150			
4	34	60	120	31	150			
5	34	140	120	31	150			
6	34	100	112	31	150			
7	34	100	128	31	150			
8	34	100	120	27	150			
9	34	100	120	35	150			
10	34	100	120	31	90			
11	34	100	120	31	210			

E[FS] =
Var[FS]=
σ[FS]=
V(FS) =
FS req'd = 1.00

E[ln FS] =
σ[ln FS]=
ln(FS req'd) = 0.000000

Total

β =
F(z) =
Pr(f) % = 0.000000

Elev. 25.3	**Head =**	1.50	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	120	31	150	3.73		
2	30	100	120	31	150			
3	38	100	120	31	150			
4	34	60	120	31	150			
5	34	140	120	31	150			
6	34	100	112	31	150			
7	34	100	128	31	150			
8	34	100	120	27	150			
9	34	100	120	35	150			
10	34	100	120	31	90			
11	34	100	120	31	210			

E[FS] =
Var[FS]=
σ[FS]=
V(FS) =
FS req'd = 1.00

E[ln FS] =
σ[ln FS]=
ln(FS req'd) = 0.000000

Total

β =
F(z) =
Pr(f) % = 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

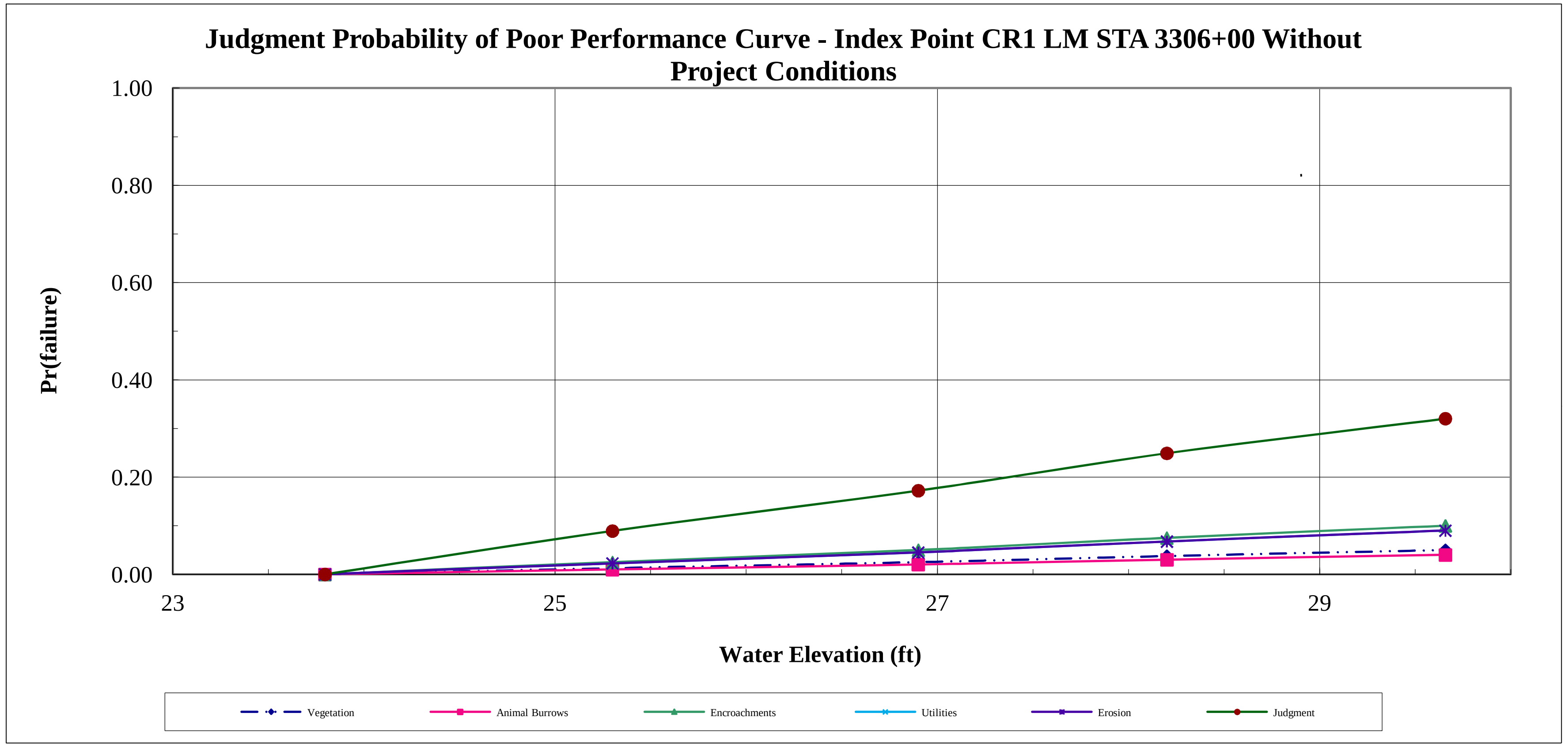
Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point CR1

Levee Mile: STA 3306+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 29.66
L/S Toe Elev.: 23.80
W/S Toe Elev.: 22.90

Analysis By: G. Johnson
Checked By: M. Perlea, J. F
Date: 9/28/2012

Water Surface	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
Elevation	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
23.80	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
25.30	0.0125	0.9875	0.0100	0.9900	0.0250	0.9750	0.0225	0.9775	0.0225	0.9775	0.0892	0.9108
26.90	0.0250	0.9750	0.0200	0.9800	0.0500	0.9500	0.0450	0.9550	0.0450	0.9550	0.1721	0.8279
28.20	0.0375	0.9625	0.0300	0.9700	0.0750	0.9250	0.0675	0.9325	0.0675	0.9325	0.2490	0.7510
29.66	0.0500	0.9500	0.0400	0.9600	0.1000	0.9000	0.0900	0.9100	0.0900	0.9100	0.3203	0.6797



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

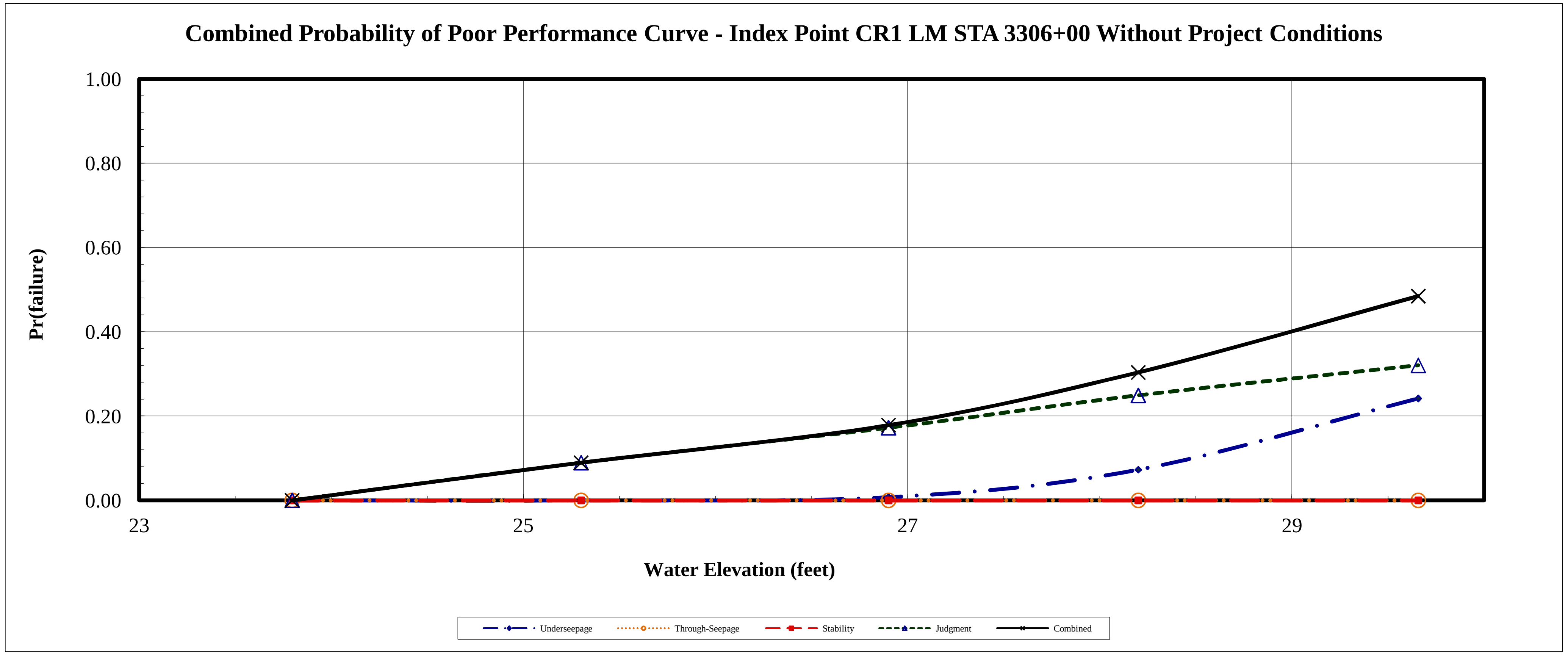
Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point CR1

Levee Mile: STA 3306+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 29.66
L/S Toe Elev.: 23.80
W/S Toe Elev.: 22.90

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hog
Date: 9/28/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
23.80	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
25.30	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0892	0.9108	0.0892	0.9108
26.90	0.0074	0.9926	0.0000	1.0000	0.0000	1.0000	0.1721	0.8279	0.1783	0.8217
28.20	0.0727	0.9273	0.0000	1.0000	0.0000	1.0000	0.2490	0.7510	0.3036	0.6964
29.66	0.2418	0.7582	0.0000	1.0000	0.0000	1.0000	0.3203	0.6797	0.4846	0.5154


Combined Probability of Poor Performance Curve - Index Point CR1 LM STA 3306+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: Right Bank Calaveras River
Basin and Reach: Index Point D4

Levee Mile: STA 3092+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 18.82
L/S Toe Elev.: 5.37
W/S Toe Elev.: 3.18

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/25/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket		Aquifer Material		Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
											Material	Kb (ft/day)	Material	Kf (ft/day)					
WR2074_001B	12	15	7	81	47	33	30	2	158	7	CL/ML	0.0007	SP-SM	19.6	28000	3804	9777	76919955	98
WR2074_002B	19					28					CL/ML	0.007	SP-SM	2.8	400				
WR2074_003B	28					28					CL	0.007	SP-SM	2.8	400				
WR2074_004B	7					28					CL	0.007	SP-SM	2.8	400				
WR2074_005B	18					32					CL/ML	0.007	SP-SM	2.8	400				
WCNBCR_003B	8.5					30					CH/ML	0.007	SP-SM	1.8	257				
WCNBCR_004B	8.3					27					CL/ML	0.007	SP-SM	2	286				
WCNBCR_005B	19					30					CL/ML	0.007	SP-SM	2	286				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WR2074_001B	CL	11	0.0007	ML	10	0.007	12	SP-SM	33	19.6							19.6
WR2074_002B	CL	18	0.007	ML	10	0.07	19	SP-SM	28	2.8							2.8
WR2074_003B	CL	28	0.007				28	SP-SM	28	2.8							2.8
WR2074_004B	CL	7	0.007				7	SP-SM	28	2.8							2.8
WR2074_005B	CL/ML	18	0.007				18	SP-SM	32	2.8							2.8
WCNBCR_003B	CH	8	0.007	ML	5	0.07	8.5	SP-SM	30	1.8							1.8
WCNBCR_004B	CL	8	0.007	ML	3	0.07	8.3	SP-SM	27	2							2
WCNBCR_005B	CL	18	0.007	ML	10	0.07	19	SP-SM	30	2							2

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point D4

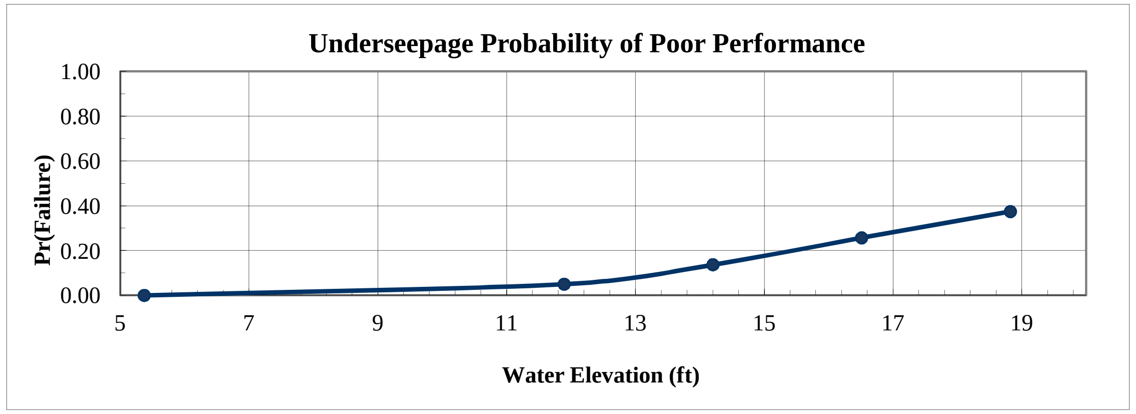
Levee Mile: STA 3092+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 18.82
L/S Toe Elev.: 5.37
W/S Toe Elev.: 3.18

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/25/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permaebility Ratio	3804	3728	98
Blanket Thickness (z)	15	7	47
Aquifer Thickness (d)	30	2	7

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	86	103	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	5.37	0.0000
Elev. 11.89	6.52	11.89	0.0500
200 yr	8.83	14.20	0.1369
Elev. 16.51	11.14	16.51	0.2570
Crest	13.45	18.82	0.3744

Crest	Rh
Head = 13.45	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	3804	15.00	30.00	85.88	1308.36	0.0200	11.75	0.78		
2	7532	15.00	30.00	85.94	1841.02	0.0148	12.20	0.81	0.032400	15.21
3	76	15.00	30.00	80.30	185.03	0.0814	6.76	0.45		
4	3804	22.00	30.00	85.92	1584.50	0.0169	12.02	0.55	0.180625	84.78
5	3804	8.00	30.00	85.77	955.49	0.0262	11.23	1.40		
6	3804	15.00	32.00	85.88	1351.27	0.0208	11.80	0.79	0.000025	0.01
7	3804	15.00	28.00	85.87	1263.99	0.0193	11.70	0.78		
								Total	0.213050	100.00

E[I] = 0.780000
Var[I]= 0.213050
σ[I]= 0.461573
V(I) = 0.591761

E[ln I] = -0.398581
σ [ln I] = 0.547940

Ic= 0.80

ln(I crit) = -0.223144

β =	-0.727416
F(z) =	0.625582
Pr(f) % =	37.441763

Elev. 16.51	Rh
Head = 11.14	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	3804	15.00	30.00	85.88	1308.36	0.0200	9.73	0.65		
2	7532	15.00	30.00	85.94	1841.02	0.0148	10.10	0.67	0.022500	15.15
3	76	15.00	30.00	80.30	185.03	0.0814	5.60	0.37		
4	3804	22.00	30.00	85.92	1584.50	0.0169	9.95	0.45	0.126025	84.85
5	3804	8.00	30.00	85.77	955.49	0.0262	9.30	1.16		
6	3804	15.00	32.00	85.88	1351.27	0.0208	9.77	0.65	0.000000	0.00
7	3804	15.00	28.00	85.87	1263.99	0.0193	9.69	0.65		
								Total	0.148525	100.00

E[I] = 0.650000
Var[I]= 0.148525
σ[I]= 0.385389
V(I) = 0.592907

E[ln I] = -0.581405
σ [ln I] = 0.548857

Ic= 0.80

ln(I crit) = -0.223144

β =	-1.059302
F(z) =	0.743038
Pr(f) % =	25.696167

200 yr	Rh
Head = 8.83	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	3804	15.00	30.00	85.88	1308.36	0.0200	7.72	0.51		
2	7532	15.00	30.00	85.94	1841.02	0.0148	8.01	0.53	0.013225	14.43
3	76	15.00	30.00	80.30	185.03	0.0814	4.44	0.30		
4	3804	22.00	30.00	85.92	1584.50	0.0169	7.89	0.36	0.078400	85.54
5	3804	8.00	30.00	85.77	955.49	0.0262	7.37	0.92		
6	3804	15.00	32.00	85.88	1351.27	0.0208	7.75	0.52	0.000025	0.03
7	3804	15.00	28.00	85.87	1263.99	0.0193	7.68	0.51		
								Total	0.091650	100.00

E[I] = 0.510000
Var[I]= 0.091650
σ[I]= 0.302738
V(I) = 0.593603

E[ln I] = -0.824272
σ [ln I] = 0.549413

Ic= 0.80

ln(I crit) = -0.223144

β =	-1.500278
F(z) =	0.863051
Pr(f) % =	13.694933

Elev. 11.89	Rh
Head = 6.52	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	3804	15.00	30.00	85.88	1308.36	0.0200	5.70	0.38		
2	7532	15.00	30.00	85.94	1841.02	0.0148	5.91	0.39	0.007225	14.67
3	76	15.00	30.00	80.30	185.03	0.0814	3.28	0.22		
4	3804	22.00	30.00	85.92	1584.50	0.0169	5.83	0.27	0.042025	85.33
5	3804	8.00	30.00	85.77	955.49	0.0262	5.44	0.68		
6	3804	15.00	32.00	85.88	1351.27	0.0208	5.72	0.38	0.000000	0.00
7	3804	15.00	28.00	85.87	1263.99	0.0193	5.67	0.38		
								Total	0.049250	100.00

E[I] = 0.380000
Var[I]= 0.049250
σ[I]= 0.221923
V(I) = 0.584009

E[ln I] = -1.114317
σ [ln I] = 0.541724

Ic= 0.80

ln(I crit) = -0.223144

β =	-2.056981
F(z) =	0.950022
Pr(f) % =	4.997793

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

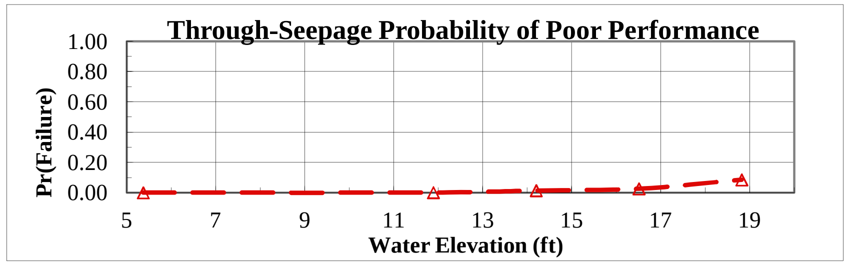
Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point D4

Levee Mile: STA 3092+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 18.82
L/S Toe Elev.: 5.37
W/S Toe Elev.: 3.18

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/25/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	4	0.4	10.00
Initial Porosity (n)	0.39	0.04	10.00
Initial Permeability (Ko)	2.00E-06	6.00E-07	30.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	5.37	0.0000
Elev. 11.89	6.52	11.89	0.001302
200 yr	8.83	14.20	0.014271
Elev. 16.51	11.14	16.51	0.026035
Crest	13.45	18.82	0.085097

Pr(f)=0
NO

Crest	Head =	13.45	Horizontal Gradient (Ix) =	0.480

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	4.00	0.39	2.00E-06	0.64	1.33		
2	3.60	0.39	2.00E-06	0.57	1.19	0.017605	26.44
3	4.40	0.39	2.00E-06	0.70	1.46		
4	4.00	0.35	2.00E-06	0.60	1.26	0.004412	6.63
5	4.00	0.43	2.00E-06	0.67	1.39		
6	4.00	0.39	1.40E-06	0.76	1.59	0.044555	66.93
7	4.00	0.39	2.60E-06	0.56	1.16		

E[FS] =	1.326846	E[ln FS] =	0.264246	Total	0.066573	100.00
Var[FS]=	0.066573					
σ[FS]=	0.258017	σ[ln FS]=	0.192658	β =	1.371584
V(FS) =	0.194459			F(z) =	0.085097
FS req'd =	1.00	ln(FS req'd) =	0.000000	Pr(f) % =	8.509653

Elev. 16.51	Head =	11.14	Horizontal Gradient (Ix) =	0.430

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	4.00	0.39	2.00E-06	0.64	1.48		
2	3.60	0.39	2.00E-06	0.57	1.33	0.021937	26.44
3	4.40	0.39	2.00E-06	0.70	1.63		
4	4.00	0.35	2.00E-06	0.60	1.41	0.005498	6.63
5	4.00	0.43	2.00E-06	0.67	1.55		
6	4.00	0.39	1.40E-06	0.76	1.77	0.055520	66.93
7	4.00	0.39	2.60E-06	0.56	1.30		

E[FS] =	1.481130	E[ln FS] =	0.374247	Total	0.082955	100.00
Var[FS]=	0.082955					
σ[FS]=	0.288020	σ[ln FS]=	0.192658	β =	1.942549	
V(FS) =	0.194459			F(z) =	0.026035	
FS req'd =	1.00	ln(FS req'd) =	0.000000	Pr(f) % =	2.603532	

200 yr	Head =	8.83	Horizontal Gradient (Ix) =	0.410

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	4.00	0.39	2.00E-06	0.64	1.55		
2	3.60	0.39	2.00E-06	0.57	1.40	0.024130	26.44
3	4.40	0.39	2.00E-06	0.70	1.71		
4	4.00	0.35	2.00E-06	0.60	1.47	0.006048	6.63
5	4.00	0.43	2.00E-06	0.67	1.63		
6	4.00	0.39	1.40E-06	0.76	1.86	0.061068	66.93
7	4.00	0.39	2.60E-06	0.56	1.36		

E[FS] =	1.553381	E[ln FS] =	0.421875	Total	0.091246	100.00
Var[FS]=	0.091246					
σ[FS]=	0.302069	σ[ln FS]=	0.192658	β =	2.189765	
V(FS) =	0.194459			F(z) =	0.014271	
FS req'd =	1.00	ln(FS req'd) =	0.000000	Pr(f) % =	1.427063	

Elev. 11.89	Head =	6.52	Horizontal Gradient (Ix) =	0.350

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	4.00	0.39	2.00E-06	0.64	1.82		
2	3.60	0.39	2.00E-06	0.57	1.64	0.033112	26.44
3	4.40	0.39	2.00E-06	0.70	2.00		
4	4.00	0.35	2.00E-06	0.60	1.73	0.008299	6.63
5	4.00	0.43	2.00E-06	0.67	1.91		
6	4.00	0.39	1.40E-06	0.76	2.17	0.083801	66.93
7	4.00	0.39	2.60E-06	0.56	1.60		

E[FS] =	1.819674	E[ln FS] =	0.580099	Total	0.125212	100.00
Var[FS]=	0.125212					
σ[FS]=	0.353853	σ[ln FS]=	0.192658	β =	3.011036	
V(FS) =	0.194459			F(z) =	0.001302	
FS req'd =	1.00	ln(FS req'd) =	0.000000	Pr(f) % =	0.130179	

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

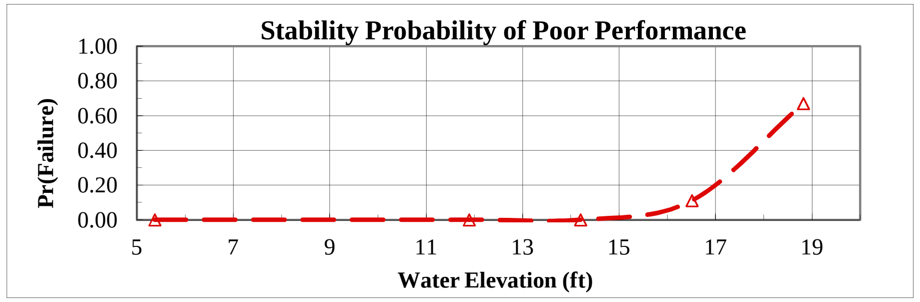
Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point D4

Levee Mile: STA 3092+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 18.82
L/S Toe Elev.: 5.37
W/S Toe Elev.: 3.18

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/25/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	34	4	13.00
Levee Cohesion	100	40	40.00
Levee γ	110	8	7.00
Foundation Φ	27	4	13.00
Foundation Cohesion	50	20	40.00


Stability Probability of Poor Performance

Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	5.37	0.0000
Elev. 11.89	6.52	11.89	0.000000
200 yr	8.83	14.20	0.000044
Elev. 16.51	11.14	16.51	0.110781
Crest	13.45	18.82	0.669813

Crest **Head =** 13.45 **Pr(f)=0** NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	110	27	50	0.95		
2	30	100	110	27	50	0.93	0.000380	2.17
3	38	100	110	27	50	0.97		
4	34	60	110	27	50	0.87	0.005550	31.71
5	34	140	110	27	50	1.02		
6	34	100	102	27	50	0.90	0.001764	10.08
7	34	100	118	27	50	0.98		
8	34	100	110	23	50	0.87	0.005776	33.00
9	34	100	110	31	50	1.03		
10	34	100	110	27	30	0.88	0.004032	23.04
11	34	100	110	27	70	1.01		

E[FS] = 0.950000
Var[FS]= 0.017503
σ[FS]= 0.132298
V(FS) = 0.139261
FS req'd = 1.00

E[ln FS] = -0.060897
σ[ln FS]= 0.138593
ln(FS req'd) = 0.000000

Total 0.017503 100.00

β = -0.439397
F(z) = 0.669813
Pr(f) % = 66.981308

Elev. 16.51 **Head =** 11.14 **Pr(f)=0** NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	110	27	50	1.18		
2	30	100	110	27	50	1.15	0.001024	4.41
3	38	100	110	27	50	1.22		
4	34	60	110	27	50	1.11	0.004761	20.49
5	34	140	110	27	50	1.25		
6	34	100	102	27	50	1.15	0.001764	7.59
7	34	100	118	27	50	1.24		
8	34	100	110	23	50	1.08	0.010712	46.11
9	34	100	110	31	50	1.29		
10	34	100	110	27	30	1.11	0.004970	21.39
11	34	100	110	27	70	1.25		

E[FS] = 1.180000
Var[FS]= 0.023232
σ[FS]= 0.152419
V(FS) = 0.129169
FS req'd = 1.00

E[ln FS] = 0.157241
σ[ln FS]= 0.128635
ln(FS req'd) = 0.000000

Total 0.023232 100.00

β = 1.222386
F(z) = 0.110781
Pr(f) % = 11.078091

200 yr **Head =** 8.83 **Pr(f)=0** NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	110	27	50	1.57		
2	30	100	110	27	50	1.52	0.002704	8.49
3	38	100	110	27	50	1.62		
4	34	60	110	27	50	1.53	0.002401	7.54
5	34	140	110	27	50	1.63		
6	34	100	102	27	50	1.56	0.000156	0.49
7	34	100	118	27	50	1.58		
8	34	100	110	23	50	1.43	0.018225	57.21
9	34	100	110	31	50	1.70		
10	34	100	110	27	30	1.49	0.008372	26.28
11	34	100	110	27	70	1.67		

E[FS] = 1.570000
Var[FS]= 0.031859
σ[FS]= 0.178489
V(FS) = 0.113688
FS req'd = 1.00

E[ln FS] = 0.444655
σ[ln FS]= 0.113323
ln(FS req'd) = 0.000000

Total 0.031859 100.00

β = 3.923788
F(z) = 0.000044
Pr(f) % = 0.004358

Elev. 11.89 **Head =** 6.52 **Pr(f)=0** YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	34	100	110	27	50	1.89		
2	30	100	110	27	50			
3	38	100	110	27	50			
4	34	60	110	27	50			
5	34	140	110	27	50			
6	34	100	102	27	50			
7	34	100	118	27	50			
8	34	100	110	23	50			
9	34	100	110	31	50			
10	34	100	110	27	30			
11	34	100	110	27	70			

E[FS] =
Var[FS]=
σ[FS]=
V(FS) =
FS req'd = 1.00

E[ln FS] =
σ[ln FS]=
ln(FS req'd) = 0.000000

Total

β =
F(z) =
Pr(f) % = 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

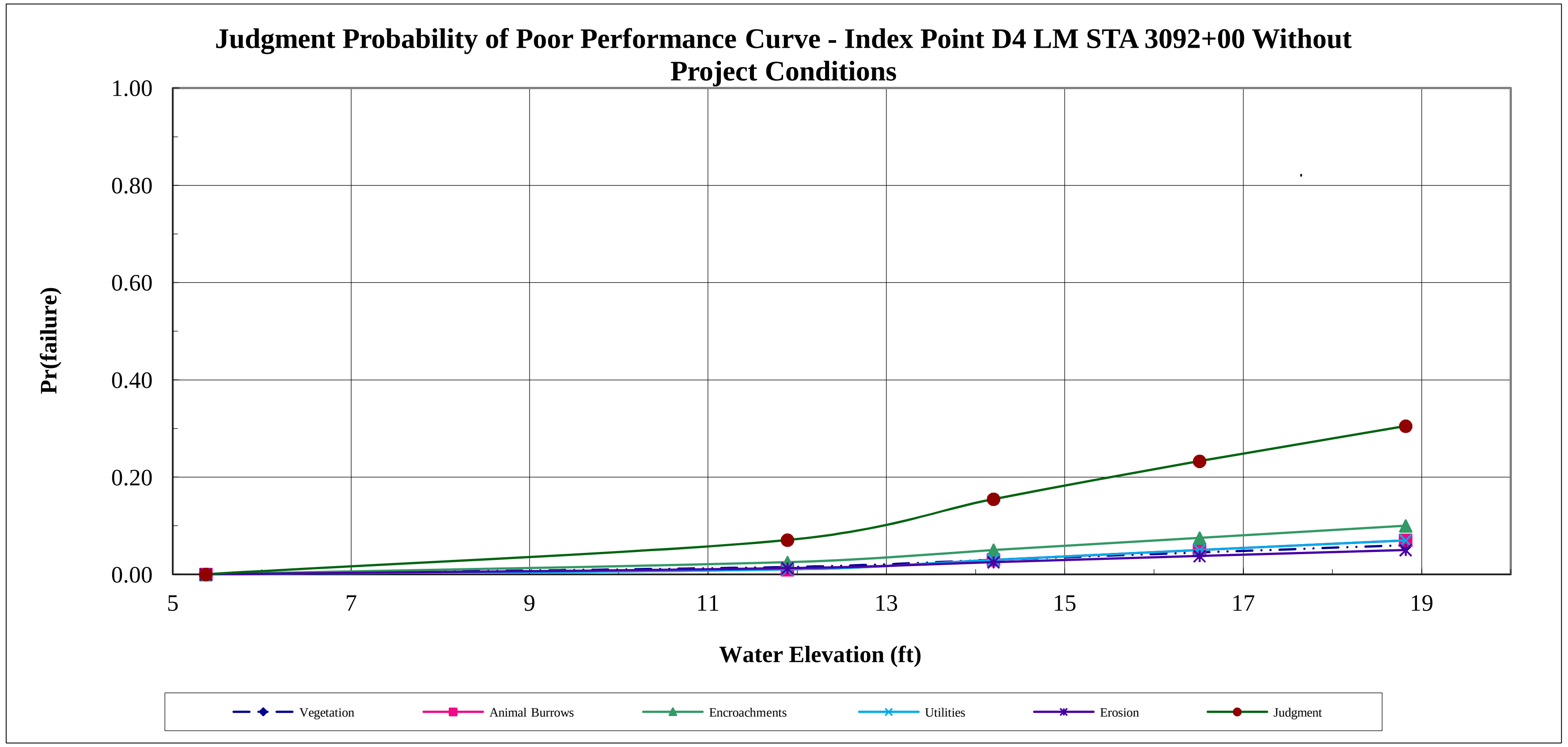
Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point D4

Levee Mile: STA 3092+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 18.82
L/S Toe Elev.: 5.37
W/S Toe Elev.: 3.18

Analysis By: G. Johnson
Checked By: M. Perlea, J. F
Date: 9/25/2012

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
5.37	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
11.89	0.0150	0.9850	0.0100	0.9900	0.0250	0.9750	0.0100	0.9900	0.0125	0.9875	0.0705	0.9295
14.20	0.0300	0.9700	0.0300	0.9700	0.0500	0.9500	0.0300	0.9700	0.0250	0.9750	0.1546	0.8454
16.51	0.0450	0.9550	0.0500	0.9500	0.0750	0.9250	0.0500	0.9500	0.0375	0.9625	0.2327	0.7673
18.82	0.0600	0.9400	0.0700	0.9300	0.1000	0.9000	0.0700	0.9300	0.0500	0.9500	0.3049	0.6951



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point D4

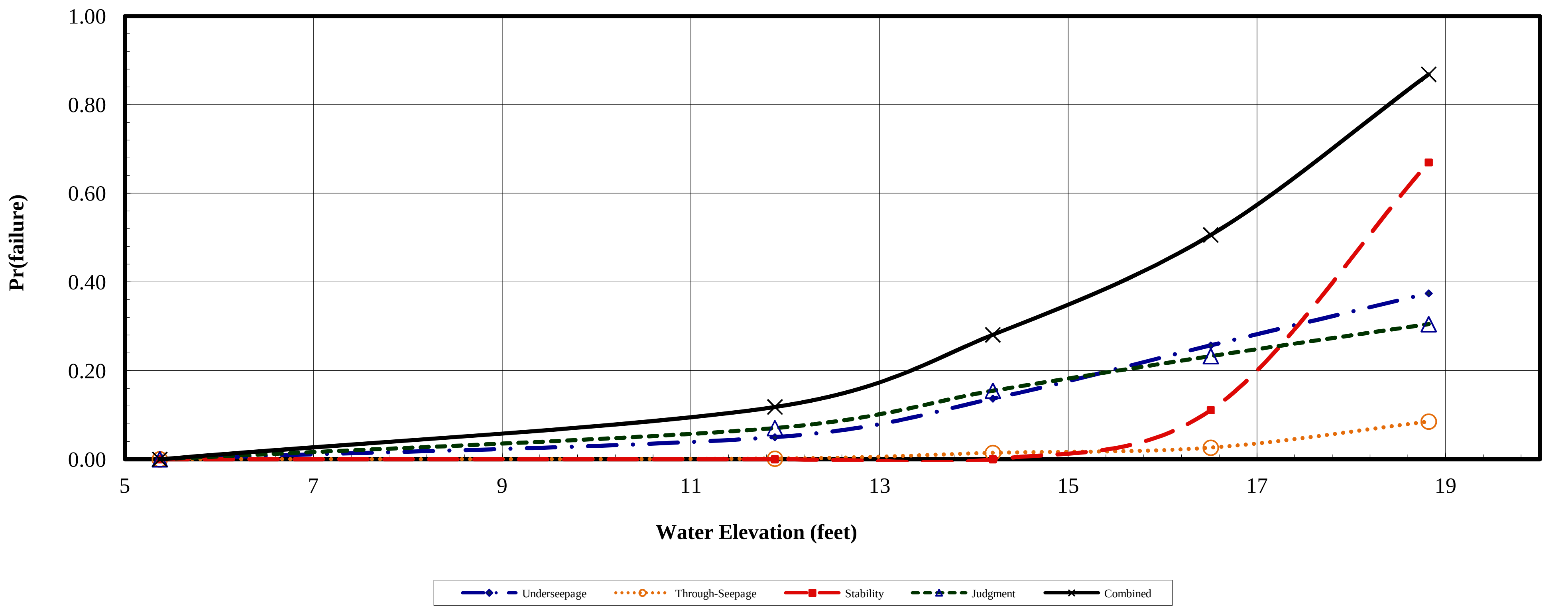
Levee Mile: STA 3092+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 18.82
L/S Toe Elev.: 5.37
W/S Toe Elev.: 3.18

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hog
Date: 9/25/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
5.37	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
11.89	0.0500	0.9500	0.0013	0.9987	0.0000	1.0000	0.0705	0.9295	0.1181	0.8819
14.20	0.1369	0.8631	0.0143	0.9857	0.0000	1.0000	0.1546	0.8454	0.2809	0.7191
16.51	0.2570	0.7430	0.0260	0.9740	0.1108	0.8892	0.2327	0.7673	0.5062	0.4938
18.82	0.3744	0.6256	0.0851	0.9149	0.6698	0.3302	0.3049	0.6951	0.8686	0.1314

Combined Probability of Poor Performance Curve - Index Point D4 LM STA 3092+00 Without Project Conditions



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: Left Bank Calaveras River
Basin and Reach: Index Point D5

Levee Mile: STA 6535+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 17.54
L/S Toe Elev.: 4.10
W/S Toe Elev.: -6.30

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/19/2012

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material Material	Aquifer Material Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
WR1614_003B	15	20	9	133	45	6	15	10	88	67	CL	0.007	SM	0.28	40	44	9	547	20
WR1614_003C	12					23					CL	0.007	SM	0.28	40				
WR1614_004B	21					7					CL	0.007	SM	0.28	40				
WR1614_006B	32					23					ML	0.007	SP-SM	0.4	57				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WR1614_003B	CL	15	0.007				15	SM	6	0.28							0.28
WR1614_003C	CL	12	0.007				12	SM	23	0.28							0.28
WR1614_004B	CL	21	0.007				21	SM	7	0.28							0.28
WR1614_006B	ML	32	0.007				32	SP-SM	23	0.4							0.4

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: Index Point D5

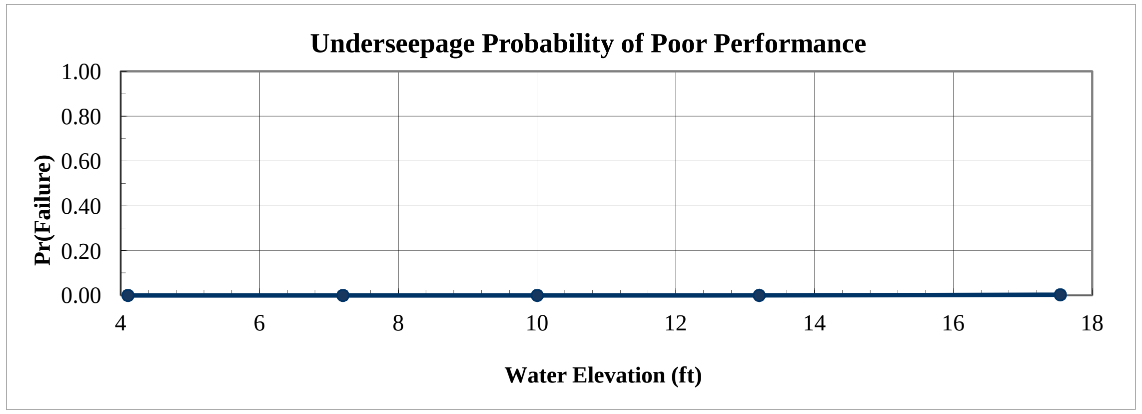
Levee Mile: STA 6535+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 17.54
L/S Toe Elev.: 4.10
W/S Toe Elev.: -6.30

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/19/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permaebility Ratio	44	9	20
Blanket Thickness (z)	20	9	45
Aquifer Thickness (d)	15	10	67

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	120	85	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	4.10	0.0000
Toe+3ft	3.10	7.20	0.0000
Half Height	5.90	10.00	0.0000
200yr	9.10	13.20	0.0001
Crest	13.44	17.54	0.0028

Crest		**Rh**
Head =	13.44	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	44	20.00	15.00	89.57	114.89	0.0518	5.33	0.27		
2	53	20.00	15.00	93.38	126.10	0.0493	5.57	0.28	0.000225	1.57
3	35	20.00	15.00	84.50	102.47	0.0552	5.06	0.25		
4	44	29.00	15.00	96.85	138.35	0.0468	5.81	0.20	0.012100	84.32
5	44	11.00	15.00	75.59	85.21	0.0610	4.66	0.42		
6	44	20.00	25.00	99.24	148.32	0.0752	5.99	0.30	0.002025	14.11
7	44	20.00	5.00	62.87	66.33	0.0233	4.16	0.21		
								Total	0.014350	100.00

E[I] = 0.270000
Var[I]= 0.014350
σ[I]= 0.119791
V(I) = 0.443672

E[ln I] = -1.399178
σ [ln I] = 0.423897

Ic= 0.80

ln(I crit) = -0.223144

β =	-3.300747
F(z) =	0.997234
Pr(f) % =	0.276571

200yr		**Rh**
Head =	9.10	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	44	20.00	15.00	89.57	114.89	0.0518	3.61	0.18		
2	53	20.00	15.00	93.38	126.10	0.0493	3.77	0.19	0.000100	1.51
3	35	20.00	15.00	84.50	102.47	0.0552	3.43	0.17		
4	44	29.00	15.00	96.85	138.35	0.0468	3.93	0.14	0.005625	84.91
5	44	11.00	15.00	75.59	85.21	0.0610	3.15	0.29		
6	44	20.00	25.00	99.24	148.32	0.0752	4.06	0.20	0.000900	13.58
7	44	20.00	5.00	62.87	66.33	0.0233	2.82	0.14		
								Total	0.006625	100.00

E[I] = 0.180000
Var[I]= 0.006625
σ[I]= 0.081394
V(I) = 0.452189

E[ln I] = -1.807820
σ [ln I] = 0.431328

Ic= 0.80

ln(I crit) = -0.223144

β =	-4.191287
F(z) =	0.999881
Pr(f) % =	0.011942

Half Height		**Rh**
Head =	5.90	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	44	20.00	15.00	89.57	114.89	0.0518	2.34	0.12		
2	53	20.00	15.00	93.38	126.10	0.0493	2.44	0.12	0.000025	0.85
3	35	20.00	15.00	84.50	102.47	0.0552	2.22	0.11		
4	44	29.00	15.00	96.85	138.35	0.0468	2.55	0.09	0.002500	85.47
5	44	11.00	15.00	75.59	85.21	0.0610	2.05	0.19		
6	44	20.00	25.00	99.24	148.32	0.0752	2.63	0.13	0.000400	13.68
7	44	20.00	5.00	62.87	66.33	0.0233	1.83	0.09		
								Total	0.002925	100.00

E[I] = 0.120000
Var[I]= 0.002925
σ[I]= 0.054083
V(I) = 0.450694

E[ln I] = -2.212725
σ [ln I] = 0.430026

Ic= 0.80

ln(I crit) = -0.223144

β =	-5.145561
F(z) =	0.999998
Pr(f) % =	0.000186

Toe+3ft		**Rh**
Head =	3.10	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	44	20.00	15.00	89.57	114.89	0.0518	1.23	0.06		
2	53	20.00	15.00	93.38	126.10	0.0493	1.28	0.06	0.000000	0.00
3	35	20.00	15.00	84.50	102.47	0.0552	1.17	0.06		
4	44	29.00	15.00	96.85	138.35	0.0468	1.34	0.05	0.000625	86.21
5	44	11.00	15.00	75.59	85.21	0.0610	1.07	0.10		
6	44	20.00	25.00	99.24	148.32	0.0752	1.38	0.07	0.000100	13.79
7	44	20.00	5.00	62.87	66.33	0.0233	0.96	0.05		
								Total	0.000725	100.00

E[I] = 0.060000
Var[I]= 0.000725
σ[I]= 0.026926
V(I) = 0.448764

E[ln I] = -2.905150
σ [ln I] = 0.428344

Ic= 0.80

ln(I crit) = -0.223144

β =	-6.782287
F(z) =	1.000000
Pr(f) % =	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: Index Point D5

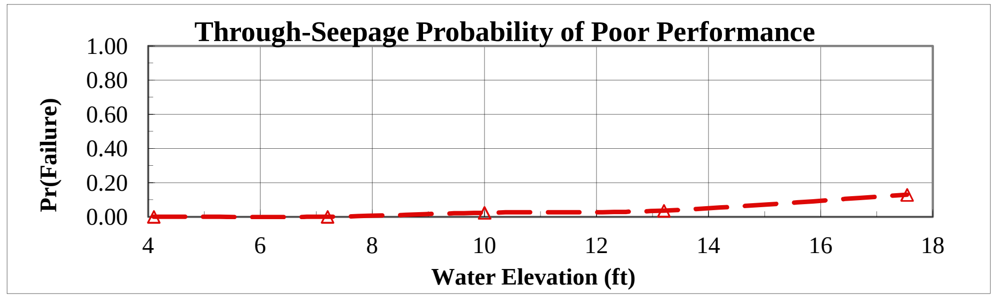
Levee Mile: STA 6535+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 17.54
L/S Toe Elev.: 4.10
W/S Toe Elev.: -6.30

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/19/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	2.9	0.3	10.00
Initial Porosity (n)	0.32	0.03	10.00
Initial Permeability (Ko)	2.00E-06	6.00E-07	30.00

Pr(f)=0
NO



Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	4.10	0.0000
Toe+3ft	3.10	7.20	0.000000
Half Height	5.90	10.00	0.023480
200 yr	9.10	13.20	0.035575
Crest	13.44	17.54	0.128431

Crest	**Head =**	13.44	**Horizontal Gradient (Ix) =**	0.330

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	2.90	0.32	2.00E-06	0.42	1.27		
2	2.61	0.32	2.00E-06	0.38	1.14	0.016064	26.44
3	3.19	0.32	2.00E-06	0.46	1.39		
4	2.90	0.29	2.00E-06	0.40	1.20	0.004026	6.63
5	2.90	0.35	2.00E-06	0.44	1.33		
6	2.90	0.32	1.40E-06	0.50	1.51	0.040655	66.93
7	2.90	0.32	2.60E-06	0.37	1.11		
E[FS] =	1.267443	E[ln FS] =		0.218443	Total	0.060746	100.00
Var[FS]=	0.060746						
σ[FS]=	0.246466	σ[ln FS]=		0.192658		β =	1.133841
V(FS) =	0.194459					F(z) =	0.128431
FS req'd =	1.00	ln(FS req'd) =		0.000000		**Pr(f) % =**	12.843072

Half Height	**Head =**	5.90	**Horizontal Gradient (Ix) =**	0.280

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	2.90	0.32	2.00E-06	0.42	1.49		
2	2.61	0.32	2.00E-06	0.38	1.34	0.022314	26.44
3	3.19	0.32	2.00E-06	0.46	1.64		
4	2.90	0.29	2.00E-06	0.40	1.42	0.005592	6.63
5	2.90	0.35	2.00E-06	0.44	1.57		
6	2.90	0.32	1.40E-06	0.50	1.79	0.056471	66.93
7	2.90	0.32	2.60E-06	0.37	1.31		
E[FS] =	1.493772	E[ln FS] =		0.382746	Total	0.084377	100.00
Var[FS]=	0.084377						
σ[FS]=	0.290478	σ[ln FS]=		0.192658		β =	1.986664
V(FS) =	0.194459					F(z) =	0.023480
FS req'd =	1.00	ln(FS req'd) =		0.000000		**Pr(f) % =**	2.347980

200 yr	**Head =**	9.10	**Horizontal Gradient (Ix) =**	0.290

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	2.90	0.32	2.00E-06	0.42	1.44		
2	2.61	0.32	2.00E-06	0.38	1.30	0.020801	26.44
3	3.19	0.32	2.00E-06	0.46	1.59		
4	2.90	0.29	2.00E-06	0.40	1.37	0.005213	6.63
5	2.90	0.35	2.00E-06	0.44	1.51		
6	2.90	0.32	1.40E-06	0.50	1.72	0.052644	66.93
7	2.90	0.32	2.60E-06	0.37	1.26		
E[FS] =	1.442263	E[ln FS] =		0.347655	Total	0.078659	100.00
Var[FS]=	0.078659						
σ[FS]=	0.280461	σ[ln FS]=		0.192658		β =	1.804521
V(FS) =	0.194459					F(z) =	0.035575
FS req'd =	1.00	ln(FS req'd) =		0.000000		**Pr(f) % =**	3.557483

Toe+3ft	**Head =**	3.10	**Horizontal Gradient (Ix) =**	0.090

Run	**Tractive Stress (Tc)**	**Initial Porosity (n)**	**Initial Permeability (Ko)**	**Critical Gradient (Ic)**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	2.90	0.32	2.00E-06	0.42	4.65		
2	2.61	0.32	2.00E-06	0.38	4.18	0.215973	26.44
3	3.19	0.32	2.00E-06	0.46	5.11		
4	2.90	0.29	2.00E-06	0.40	4.41	0.054129	6.63
5	2.90	0.35	2.00E-06	0.44	4.87		
6	2.90	0.32	1.40E-06	0.50	5.55	0.546588	66.93
7	2.90	0.32	2.60E-06	0.37	4.08		
E[FS] =	4.647291	E[ln FS] =		1.517726	Total	0.816690	100.00
Var[FS]=	0.816690						
σ[FS]=	0.903709	σ[ln FS]=		0.192658		β =	7.877839
V(FS) =	0.194459					F(z) =	0.000000
FS req'd =	1.00	ln(FS req'd) =		0.000000		**Pr(f) % =**	0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

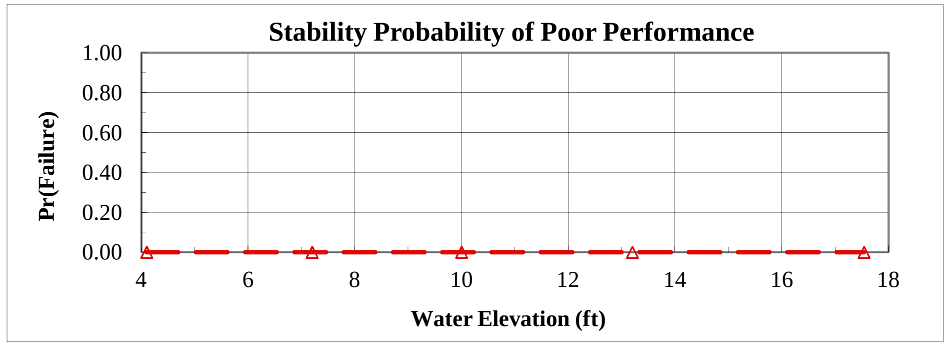
Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: Index Point D5

Levee Mile: STA 6535+00
River Mile: XX.XX
Analysis Case Without Project Conditions

Crest Elev.: 17.54
L/S Toe Elev.: 4.10
W/S Toe Elev.: -6.30

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hogan
Date: 9/19/2012

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	31	4	13.00
Levee Cohesion	150	60	40.00
Levee γ	115	8	7.00
Foundation Φ	31	4	13.00
Foundation Cohesion	150	60	40.00


Stability Probability of Poor Performance

Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	4.10	0.0000
Toe+3ft	3.10	7.20	0.000000
Half Height	5.90	10.00	0.000000
Crest-3ft	9.10	13.20	0.000000
Crest	13.44	17.54	0.000011

Crest	**Head =**	13.44	**Pr(f)=0**	NO

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	31	150	115	31	150	1.86		
2	27	150	115	31	150	1.84	0.000625	0.86
3	35	150	115	31	150	1.89		
4	31	90	115	31	150	1.82	0.001764	2.43
5	31	210	115	31	150	1.90		
6	31	150	107	31	150	1.84	0.000650	0.90
7	31	150	123	31	150	1.89		
8	31	150	115	27	150	1.74	0.014762	20.35
9	31	150	115	35	150	1.99		
10	31	150	115	31	90	1.61	0.054756	75.47
11	31	150	115	31	210	2.08		

E[FS] = 1.860000 E[ln FS] = 0.610199 Total 0.072558 100.00
Var[FS]= 0.072558
σ[FS]= 0.269365 σ[ln FS]= 0.144069 **β =** 4.235458
V(FS) = 0.144820 **F(z) =** 0.000011
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.001140

Crest-3ft	**Head =**	9.10	**Pr(f)=0**	YES

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	31	150	115	31	150	2.15		
2	27	150	115	31	150			
3	35	150	115	31	150			
4	31	90	115	31	150			
5	31	210	115	31	150			
6	31	150	107	31	150			
7	31	150	123	31	150			
8	31	150	115	27	150			
9	31	150	115	35	150			
10	31	150	115	31	90			
11	31	150	115	31	210			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Half Height	**Head =**	5.90	**Pr(f)=0**	YES

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	31	150	115	31	150	2.38		
2	27	150	115	31	150			
3	35	150	115	31	150			
4	31	90	115	31	150			
5	31	210	115	31	150			
6	31	150	107	31	150			
7	31	150	123	31	150			
8	31	150	115	27	150			
9	31	150	115	35	150			
10	31	150	115	31	90			
11	31	150	115	31	210			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Toe+3ft	**Head =**	3.10	**Pr(f)=0**	YES

Run	**Levee Φ**	**Levee Cohesion**	**Levee γ**	**Foundation Φ**	**Foundation Cohesion**	**FS**	**Variance Component**	**% Variance**
1 (Mean)	31	150	115	31	150	2.60		
2	27	150	115	31	150			
3	35	150	115	31	150			
4	31	90	115	31	150			
5	31	210	115	31	150			
6	31	150	107	31	150			
7	31	150	123	31	150			
8	31	150	115	27	150			
9	31	150	115	35	150			
10	31	150	115	31	90			
11	31	150	115	31	210			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

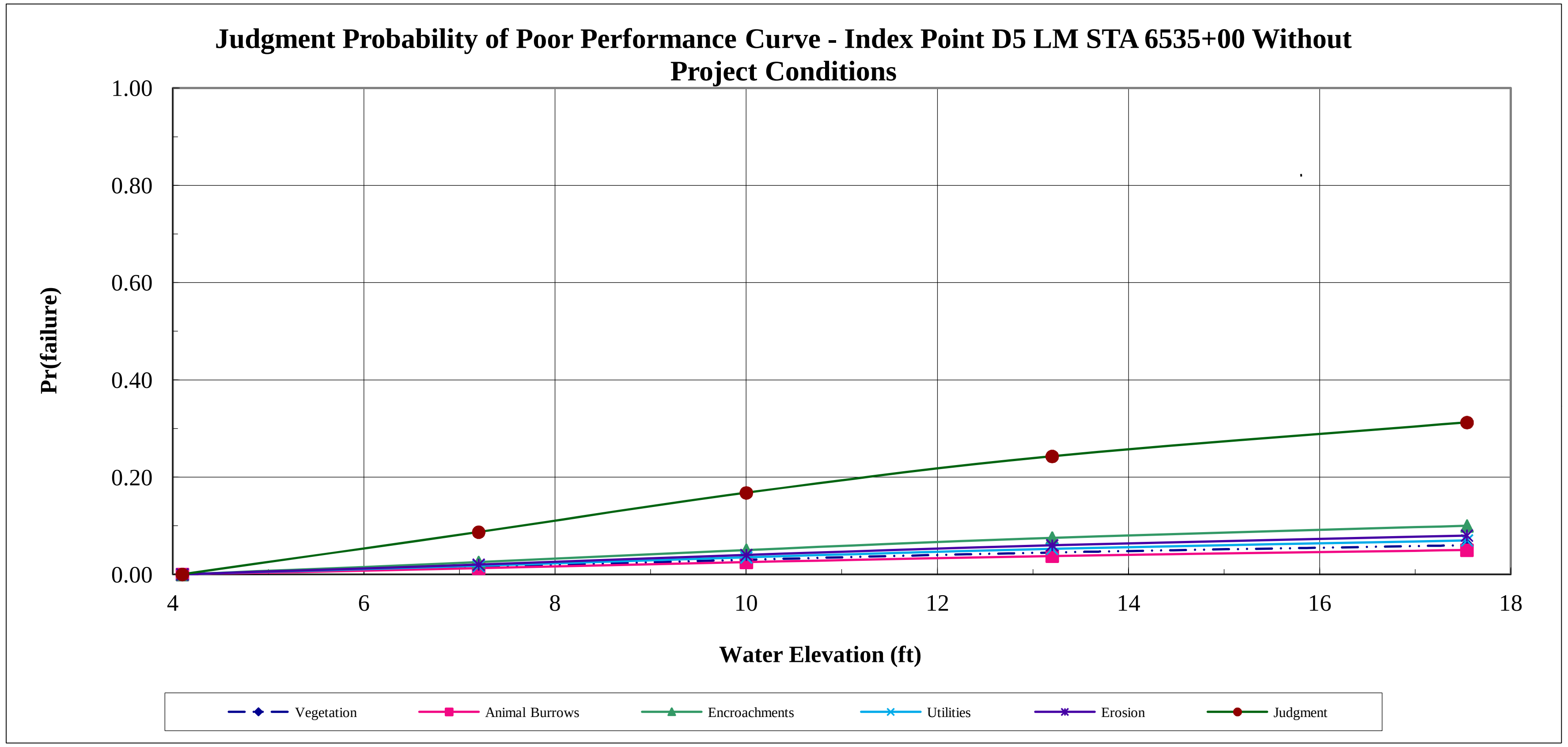
Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: Index Point D5

Levee Mile: STA 6535+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 17.54
L/S Toe Elev.: 4.10
W/S Toe Elev.: -6.30

Analysis By: G. Johnson
Checked By: M. Perlea, J. F
Date: 9/19/2012

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
4.10	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
7.20	0.0150	0.9850	0.0125	0.9875	0.0250	0.9750	0.0175	0.9825	0.0200	0.9800	0.0869	0.9131
10.00	0.0300	0.9700	0.0250	0.9750	0.0500	0.9500	0.0350	0.9650	0.0400	0.9600	0.1677	0.8323
13.20	0.0450	0.9550	0.0375	0.9625	0.0750	0.9250	0.0525	0.9475	0.0600	0.9400	0.2427	0.7573
17.54	0.0600	0.9400	0.0500	0.9500	0.1000	0.9000	0.0700	0.9300	0.0800	0.9200	0.3124	0.6876


Judgment Probability of Poor Performance Curve - Index Point D5 LM STA 6535+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

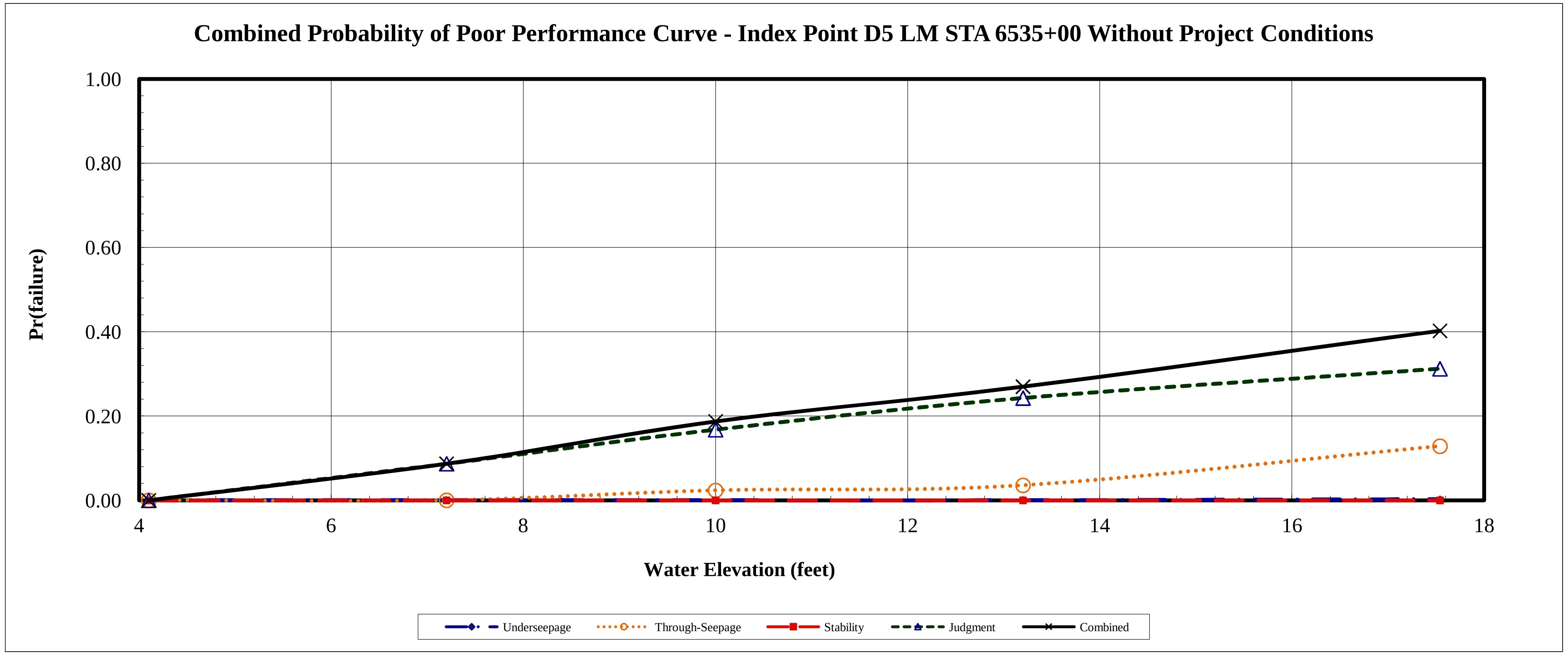
Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: Index Point D5

Levee Mile: STA 6535+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 17.54
L/S Toe Elev.: 4.10
W/S Toe Elev.: -6.30

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hog
Date: 9/19/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
4.10	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
7.20	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0869	0.9131	0.0869	0.9131
10.00	0.0000	1.0000	0.0235	0.9765	0.0000	1.0000	0.1677	0.8323	0.1872	0.8128
13.20	0.0001	0.9999	0.0356	0.9644	0.0000	1.0000	0.2427	0.7573	0.2698	0.7302
17.54	0.0028	0.9972	0.1284	0.8716	0.0000	1.0000	0.3124	0.6876	0.4023	0.5977


Combined Probability of Poor Performance Curve - Index Point D5 LM STA 6535+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: Delta Front Brookside Study Area
Basin and Reach: Index Point D-BS
Coordinates: State Plane (ft), N 2183200, E 6311320

Levee Mile: Sta. 166+50
River Mile: XXXX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 18.00
L/S Toe Elev.: -3.50
W/S Toe Elev.: -7.50

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 3/14/2013

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket Material	Blanket Kb (ft/day)	Aquifer Material Material	Aquifer Material Kf (ft/day)	Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
WR2074_013C	21	18	6	67	33	24	20	9	111	45	CL	0.0028	SP-SM	2.835	1013	607	402	180640	66
WR2074_014C	17					24					CL	0.0028	SP-SM	2.835	1013				
WR2074_011B	24					14					CL-ML	0.0283	SP-SM	2.835	100				
WR2074_015C	9					35					CL	0.0028	SM	1.134	405				
WR2074_016C	8					30					CL	0.0028	SM	1.134	405				
WR2074_008B	19					14					CL-ML	0.0283	SP-SM	2.835	100				
WR2074_018C	24					15					CL	0.0028	SM	1.134	405				
WR2074_012B	23					10					OH-CL	0.0028	SP-SM	2.835	1013				
WR2074_020C	21					10					OH-CL	0.0028	SP-SM	2.835	1013				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WR2074_013C	CL	21	0.0028				21	SP-SM	24	2.835							2.835
WR2074_014C	CL	17	0.0028				17	SP-SM	24	2.835							2.835
WR2074_011B	CL-ML	24	0.0283				24	SP-SM	14	2.835							2.835
WR2074_015C	CL	9	0.0028				9	SM	35	1.134							1.134
WR2074_016C	CL	8	0.0028				8	SM	30	1.134							1.134
WR2074_008B	CL-ML	19	0.0283				19	SP-SM	14	2.835							2.835
WR2074_018C	CL	24	0.0028				24	SM	15	1.134							1.134
WR2074_012B	OH-CL	23	0.0028				23	SP-SM	10	2.835							2.835
WR2074_020C	OH-CL	21	0.0028				21	SP-SM	10	2.835							2.835

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: Delta Front Brookside Study Area
River Section: Index Point D-BS
Coordinates: State Plane (ft), N 2183200, E 6311320

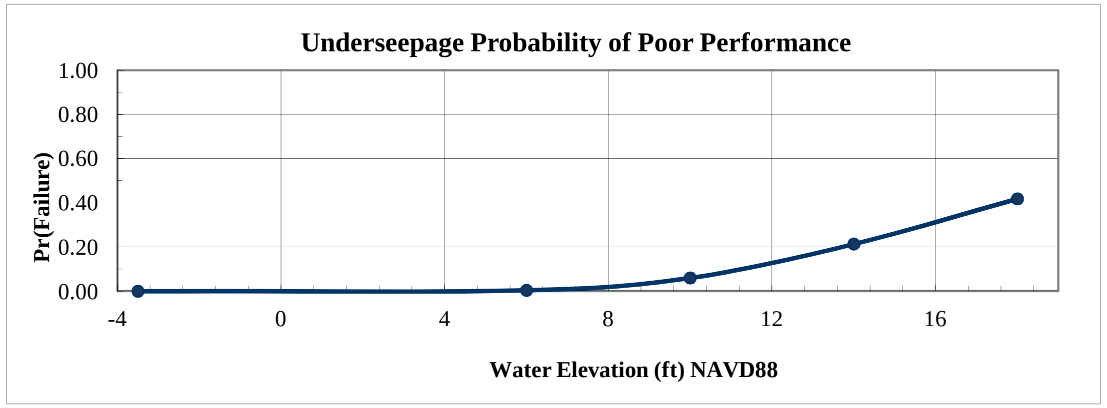
Levee Mile: Sta. 166+50
River Mile: XXXX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 18.00
L/S Toe Elev.: -3.50
W/S Toe Elev.: -7.50

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 3/14/2013

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permaebility Ratio	607	402	66
Blanket Thickness (z)	18	6	33
Aquifer Thickness (d)	20	9	45

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	100	138	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	-3.50	0.0000
Elev. 6.0	9.50	6.00	0.0041
Elev. 10.0	13.50	10.00	0.0600
Elev. 14.0	17.50	14.00	0.2136
Crest	21.50	18.00	0.4180

Crest		**Rh**
Head =	21.50	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	607	18.00	20.00	98.50	467.46	0.0284	14.28	0.79		
2	1009	18.00	20.00	99.09	602.69	0.0238	15.43	0.86	0.012100	15.85
3	205	18.00	20.00	95.72	271.66	0.0396	11.56	0.64		
4	607	24.00	20.00	98.87	539.78	0.0258	14.94	0.62	0.060025	78.62
5	607	12.00	20.00	97.77	381.68	0.0324	13.29	1.11		
6	607	18.00	29.00	98.96	562.90	0.0363	15.13	0.84	0.004225	5.53
7	607	18.00	11.00	97.32	346.68	0.0189	12.81	0.71		
								Total	0.076350	100.00

E[I] = 0.790000
Var[I]= 0.076350
σ[I]= 0.276315
V(I) = 0.349766

E[ln I] = -0.293429
σ [ln I] = 0.339724
ln(I crit) = -0.223144

Ic=	0.80

β =	-0.863726
F(z) =	0.581952
Pr(f) % =	41.804848

Elev. 14.0		**Rh**
Head =	17.50	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	607	18.00	20.00	98.50	467.46	0.0284	11.62	0.65		
2	1009	18.00	20.00	99.09	602.69	0.0238	12.56	0.70	0.008100	16.66
3	205	18.00	20.00	95.72	271.66	0.0396	9.41	0.52		
4	607	24.00	20.00	98.87	539.78	0.0258	12.16	0.51	0.038025	78.20
5	607	12.00	20.00	97.77	381.68	0.0324	10.82	0.90		
6	607	18.00	29.00	98.96	562.90	0.0363	12.32	0.68	0.002500	5.14
7	607	18.00	11.00	97.32	346.68	0.0189	10.42	0.58		
								Total	0.048625	100.00

E[I] = 0.650000
Var[I]= 0.048625
σ[I]= 0.220511
V(I) = 0.339247

E[ln I] = -0.485250
σ [ln I] = 0.330052
ln(I crit) = -0.223144

Ic=	0.80

β =	-1.470225
F(z) =	0.786442
Pr(f) % =	21.355762

Elev. 10.0		**Rh**
Head =	13.50	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	607	18.00	20.00	98.50	467.46	0.0284	8.96	0.50		
2	1009	18.00	20.00	99.09	602.69	0.0238	9.69	0.54	0.004900	16.05
3	205	18.00	20.00	95.72	271.66	0.0396	7.26	0.40		
4	607	24.00	20.00	98.87	539.78	0.0258	9.38	0.39	0.024025	78.71
5	607	12.00	20.00	97.77	381.68	0.0324	8.35	0.70		
6	607	18.00	29.00	98.96	562.90	0.0363	9.50	0.53	0.001600	5.24
7	607	18.00	11.00	97.32	346.68	0.0189	8.04	0.45		
								Total	0.030525	100.00

E[I] = 0.500000
Var[I]= 0.030525
σ[I]= 0.174714
V(I) = 0.349428

E[ln I] = -0.750748
σ [ln I] = 0.339414
ln(I crit) = -0.223144

Ic=	0.80

β =	-2.211894
F(z) =	0.939962
Pr(f) % =	6.003773

Elev. 6.0		**Rh**
Head =	9.50	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	607	18.00	20.00	98.50	467.46	0.0284	6.31	0.35		
2	1009	18.00	20.00	99.09	602.69	0.0238	6.82	0.38	0.002500	17.33
3	205	18.00	20.00	95.72	271.66	0.0396	5.11	0.28		
4	607	24.00	20.00	98.87	539.78	0.0258	6.60	0.28	0.011025	76.43
5	607	12.00	20.00	97.77	381.68	0.0324	5.87	0.49		
6	607	18.00	29.00	98.96	562.90	0.0363	6.69	0.37	0.000900	6.24
7	607	18.00	11.00	97.32	346.68	0.0189	5.66	0.31		
								Total	0.014425	100.00

E[I] = 0.350000
Var[I]= 0.014425
σ[I]= 0.120104
V(I) = 0.343155

E[ln I] = -1.105483
σ [ln I] = 0.333650
ln(I crit) = -0.223144

Ic=	0.80

β =	-3.313303
F(z) =	0.995910
Pr(f) % =	0.409050

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Lower San Joaquin
Study Area: Delta Front Brookside Study Area
River Section: Index Point D-BS
Coordinates: State Plane (ft), N 2183200, E 6311320

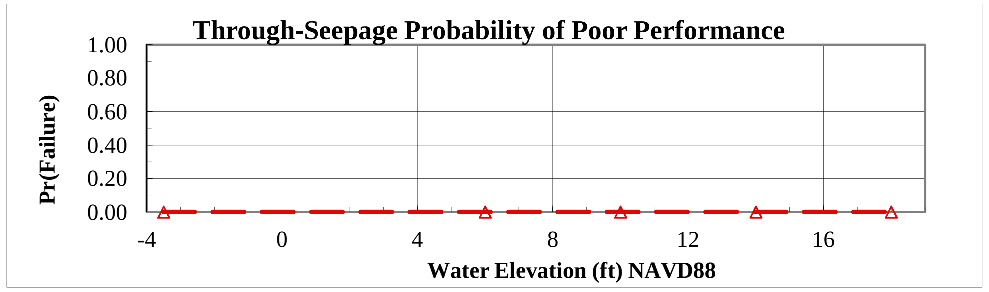
Levee Mile: Sta. 166+50
River Mile: XXXX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 18.00
L/S Toe Elev.: -3.50
W/S Toe Elev.: -7.50

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 3/14/2013

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	25	2.5	10.00
Initial Porosity (n)	0.5	0.05	10.00
Initial Permeability (Ko)	1.00E-10	3.00E-11	30.00

Pr(f)=0
NO



Analysis Case	**Head**	**Elevation**	**Pr(f)**
Toe	0.00	-3.50	0.0000
Elev. 6.0	9.50	6.00	0.000000
Elev. 10.0	13.50	10.00	0.000000
Elev. 14.0	17.50	14.00	0.000000
Crest	21.50	18.00	0.000000

Crest | **Head =** 21.50 | **Horizontal Gradient (Ix) =** 0.410

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	25.00	0.50	1.00E-10	637.40	1554.62		
2	22.50	0.50	1.00E-10	573.66	1399.16	24168.580710	26.44
3	27.50	0.50	1.00E-10	701.14	1710.09		
4	25.00	0.45	1.00E-10	604.69	1474.85	6057.326543	6.63
5	25.00	0.55	1.00E-10	668.51	1630.50		
6	25.00	0.50	7.00E-11	761.83	1858.13	61166.160886	66.93
7	25.00	0.50	1.30E-10	559.03	1363.50		

E[FS] = 1554.624736 E[ln FS] = 7.330431 Total 91392.068138 100.00
Var[FS]= 91392.068138
σ[FS]= 302.311211 σ[ln FS]= 0.192658 **β =** 38.048998
V(FS) = 0.194459 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Elev. 14.0 | **Head =** 17.50 | **Horizontal Gradient (Ix) =** 0.300

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	25.00	0.50	1.00E-10	637.40	2124.65		
2	22.50	0.50	1.00E-10	573.66	1912.19	45141.537971	26.44
3	27.50	0.50	1.00E-10	701.14	2337.12		
4	25.00	0.45	1.00E-10	604.69	2015.62	11313.739909	6.63
5	25.00	0.55	1.00E-10	668.51	2228.36		
6	25.00	0.50	7.00E-11	761.83	2539.45	114244.796054	66.93
7	25.00	0.50	1.30E-10	559.03	1863.44		

E[FS] = 2124.653806 E[ln FS] = 7.642806 Total 170700.073934 100.00
Var[FS]= 170700.073934
σ[FS]= 413.158655 σ[ln FS]= 0.192658 **β =** 39.670395
V(FS) = 0.194459 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Elev. 10.0 | **Head =** 13.50 | **Horizontal Gradient (Ix) =** 0.260

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	25.00	0.50	1.00E-10	637.40	2451.52		
2	22.50	0.50	1.00E-10	573.66	2206.37	60099.680730	26.44
3	27.50	0.50	1.00E-10	701.14	2696.68		
4	25.00	0.45	1.00E-10	604.69	2325.72	15062.671477	6.63
5	25.00	0.55	1.00E-10	668.51	2571.18		
6	25.00	0.50	7.00E-11	761.83	2930.13	152101.059836	66.93
7	25.00	0.50	1.30E-10	559.03	2150.13		

E[FS] = 2451.523623 E[ln FS] = 7.785907 Total 227263.412042 100.00
Var[FS]= 227263.412042
σ[FS]= 476.721525 σ[ln FS]= 0.192658 **β =** 40.413168
V(FS) = 0.194459 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Elev. 6.0 | **Head =** 9.50 | **Horizontal Gradient (Ix) =** 0.210

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	25.00	0.50	1.00E-10	637.40	3035.22		
2	22.50	0.50	1.00E-10	573.66	2731.70	92125.587695	26.44
3	27.50	0.50	1.00E-10	701.14	3338.74		
4	25.00	0.45	1.00E-10	604.69	2879.46	23089.265121	6.63
5	25.00	0.55	1.00E-10	668.51	3183.37		
6	25.00	0.50	7.00E-11	761.83	3627.78	233152.645009	66.93
7	25.00	0.50	1.30E-10	559.03	2662.06		

E[FS] = 3035.219723 E[ln FS] = 7.999481 Total 348367.497825 100.00
Var[FS]= 348367.497825
σ[FS]= 590.226650 σ[ln FS]= 0.192658 **β =** 41.521736
V(FS) = 0.194459 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

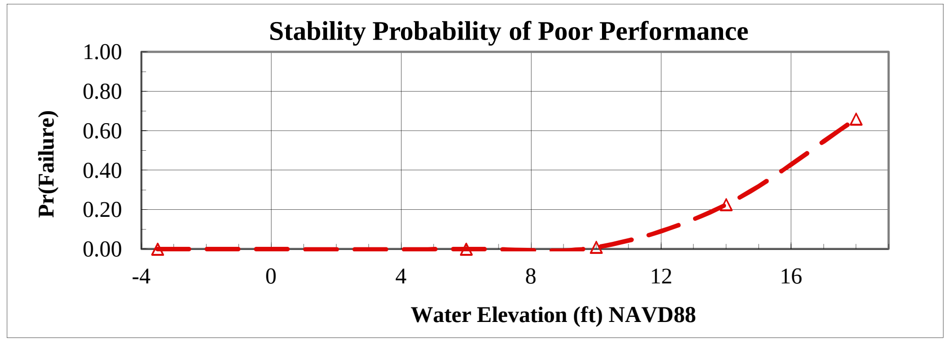
Project: Lower San Joaquin
Study Area: Delta Front Brookside Study Area
River Section: Index Point D-BS
Coordinates: State Plane (ft), N 2183200, E 6311320

Levee Mile: Sta. 166+50
River Mile: XXXX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 18.00
L/S Toe Elev.: -3.50
W/S Toe Elev.: -7.50

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 3/14/2013

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	30	4	13.00
Levee Cohesion	50	20	40.00
Levee γ	120	8	7.00
Foundation Φ	26	3	13.00
Foundation Cohesion	50	20	40.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	-3.50	0.0000
Elev. 6.0	9.50	6.00	0.000000
Elev. 10.0	13.50	10.00	0.009394
Elev. 14.0	17.50	14.00	0.225632
Crest	21.50	18.00	0.659676

Crest **Head =** 21.50 **Pr(f)=0** NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	30	50	120	26	50	0.94		
2	26	50	120	26	50	0.86	0.007832	22.19
3	34	50	120	26	50	1.03		
4	30	30	120	26	50	0.90	0.024964	70.71
5	30	70	120	26	50	1.22		
6	30	50	112	26	50	0.91	0.000420	1.19
7	30	50	128	26	50	0.96		
8	30	50	120	23	50	0.91	0.001156	3.27
9	30	50	120	29	50	0.98		
10	30	50	120	26	30	0.91	0.000930	2.64
11	30	50	120	26	70	0.97		

E[FS] = 0.940000 E[ln FS] = -0.081463 Total 0.035303 100.00
Var[FS]= 0.035303
σ[FS]= 0.187890 σ[ln FS]= 0.197929 **β =** -0.411579
V(FS) = 0.199883 **F(z) =** 0.659676
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 65.967592

Elev. 14.0 **Head =** 17.50 **Pr(f)=0** NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	30	50	120	26	50	1.13		
2	26	50	120	26	50	1.02	0.008372	28.40
3	34	50	120	26	50	1.21		
4	30	30	120	26	50	1.08	0.004225	14.33
5	30	70	120	26	50	1.21		
6	30	50	112	26	50	1.13	0.002704	9.17
7	30	50	128	26	50	1.23		
8	30	50	120	23	50	1.10	0.008556	29.02
9	30	50	120	29	50	1.28		
10	30	50	120	26	30	1.08	0.005625	19.08
11	30	50	120	26	70	1.23		

E[FS] = 1.133000 E[ln FS] = 0.113515 Total 0.029483 100.00
Var[FS]= 0.029483
σ[FS]= 0.171705 σ[ln FS]= 0.150689 **β =** 0.753308
V(FS) = 0.151549 **F(z) =** 0.225632
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 22.563248

Elev. 10.0 **Head =** 13.50 **Pr(f)=0** NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	30	50	120	26	50	1.27		
2	26	50	120	26	50	1.12	0.012769	79.39
3	34	50	120	26	50	1.34		
4	30	30	120	26	50	1.19	0.001156	7.19
5	30	70	120	26	50	1.26		
6	30	50	112	26	50	1.22	0.000016	0.10
7	30	50	128	26	50	1.23		
8	30	50	120	23	50	1.20	0.001722	10.71
9	30	50	120	29	50	1.29		
10	30	50	120	26	30	1.21	0.000420	2.61
11	30	50	120	26	70	1.25		

E[FS] = 1.270000 E[ln FS] = 0.234056 Total 0.016084 100.00
Var[FS]= 0.016084
σ[FS]= 0.126821 σ[ln FS]= 0.099611 **β =** 2.349692
V(FS) = 0.099859 **F(z) =** 0.009394
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.939449

Elev. 6.0 **Head =** 9.50 **Pr(f)=0** YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	30	50	120	26	50	1.30		
2	26	50	120	26	50			
3	34	50	120	26	50			
4	30	30	120	26	50			
5	30	70	120	26	50			
6	30	50	112	26	50			
7	30	50	128	26	50			
8	30	50	120	23	50			
9	30	50	120	29	50			
10	30	50	120	26	30			
11	30	50	120	26	70			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

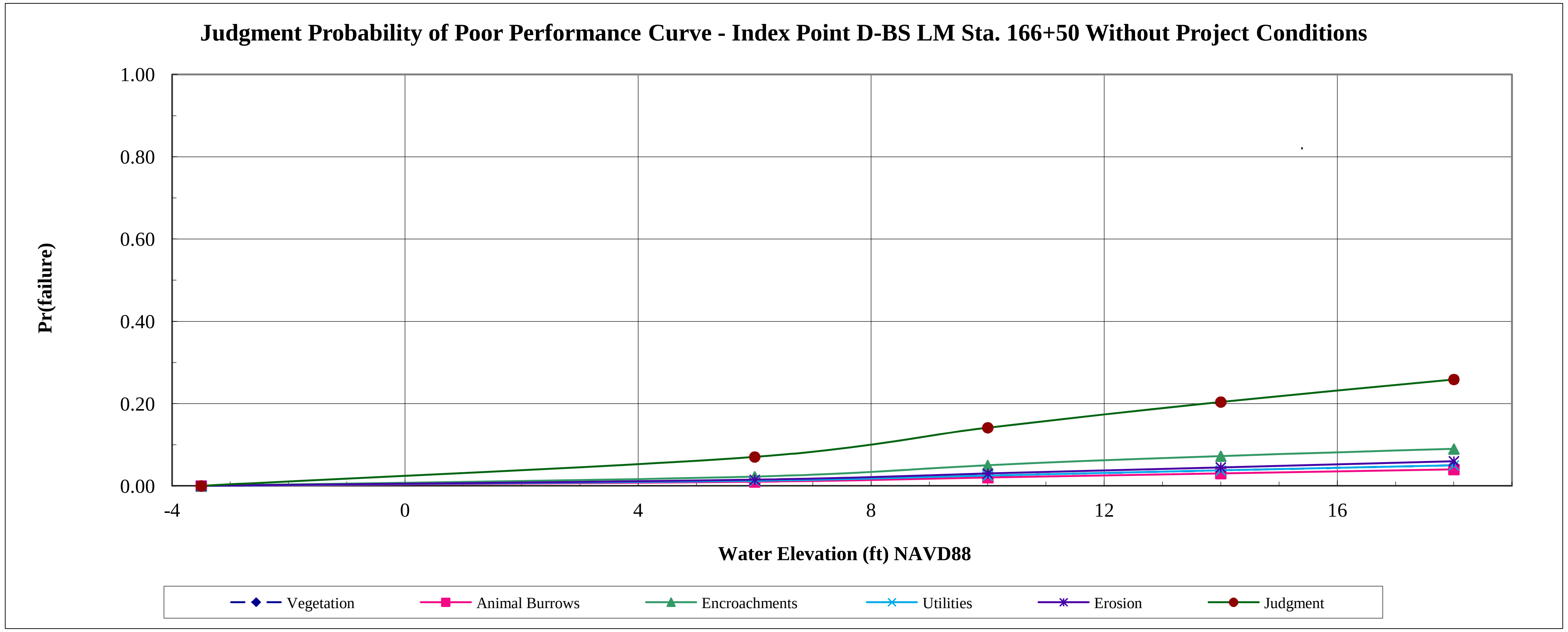
Project: Lower San Joaquin
Study Area: Delta Front Brookside Study Area
River Section: Index Point D-BS
Coordinates: State Plane (ft), N 2183200, E 6311320

Levee Mile: Sta. 166+50
River Mile: XXXX
Analysis Case: Without Project Conditions

Datum: NAVD 88
Crest Elev.: 18.00
L/S Toe Elev.: -3.50
W/S Toe Elev.: -7.50

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perle
Date: 3/14/2013

Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
-3.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
6.00	0.0125	0.9875	0.0100	0.9900	0.0225	0.9775	0.0125	0.9875	0.0150	0.9850	0.0705	0.9295
10.00	0.0250	0.9750	0.0200	0.9800	0.0500	0.9500	0.0250	0.9750	0.0300	0.9700	0.1415	0.8585
14.00	0.0375	0.9625	0.0300	0.9700	0.0725	0.9275	0.0375	0.9625	0.0450	0.9550	0.2040	0.7960
18.00	0.0500	0.9500	0.0400	0.9600	0.0900	0.9100	0.0500	0.9500	0.0600	0.9400	0.2589	0.7411


Judgment Probability of Poor Performance Curve - Index Point D-BS LM Sta. 166+50 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

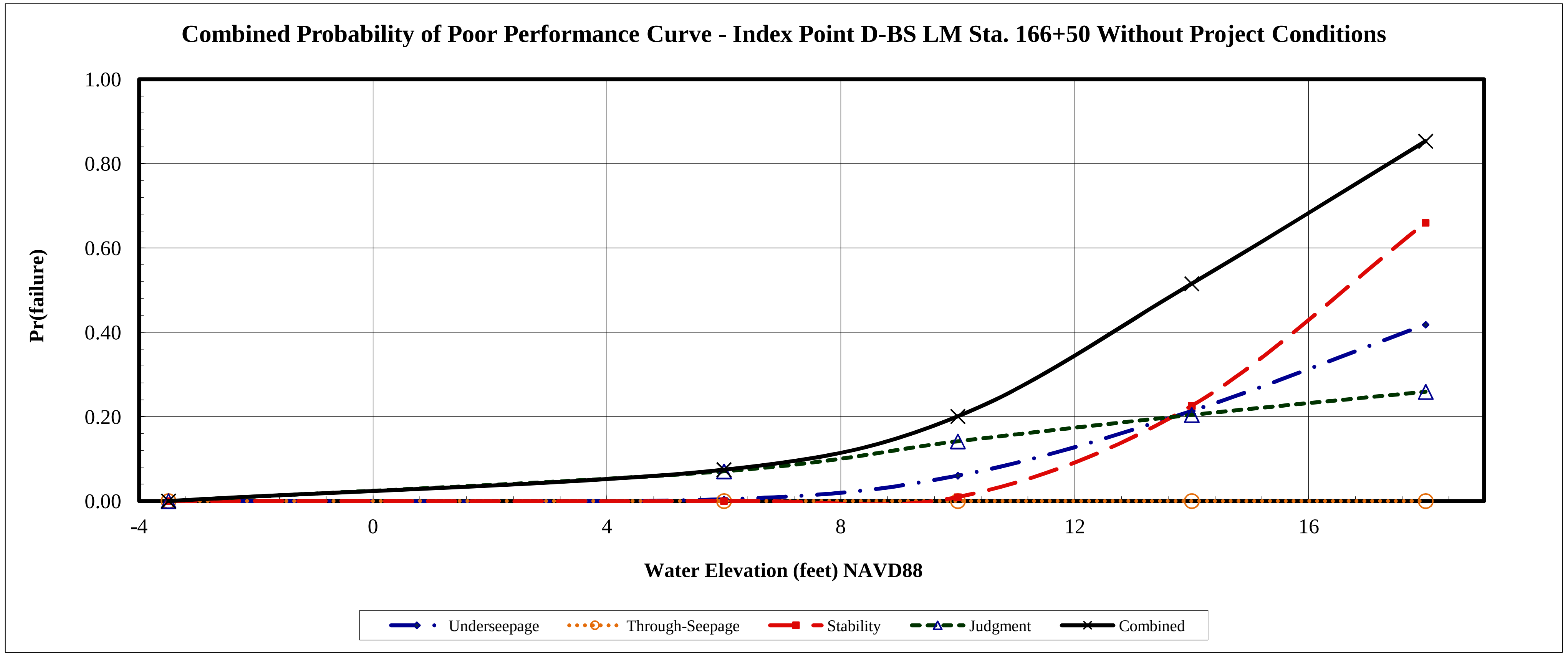
Project: Lower San Joaquin
Study Area: Delta Front Brookside Study Area
River Section: Index Point D-BS
Coordinates: State Plane (ft), N 2183200, E 6311320

Levee Mile: Sta. 166+50
River Mile: XXXX
Analysis Case: Without Project Conditions

Datum: NAVD 88
Crest Elev.: 18.00
L/S Toe Elev.: -3.50
W/S Toe Elev.: -7.50

Analysis By: G. Johnson
Checked By: J. Hogan, M. Per
Date: 3/14/2013

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
-3.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
6.00	0.0041	0.9959	0.0000	1.0000	0.0000	1.0000	0.0705	0.9295	0.0743	0.9257
10.00	0.0600	0.9400	0.0000	1.0000	0.0094	0.9906	0.1415	0.8585	0.2006	0.7994
14.00	0.2136	0.7864	0.0000	1.0000	0.2256	0.7744	0.2040	0.7960	0.5153	0.4847
18.00	0.4180	0.5820	0.0000	1.0000	0.6597	0.3403	0.2589	0.7411	0.8532	0.1468


Combined Probability of Poor Performance Curve - Index Point D-BS LM Sta. 166+50 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Determination of Random Variables For Underseepage Reliability Analysis

Project: Lower San Joaquin
Channel: Delta Front Lincoln Village
Basin and Reach: Index Point D-LV
Coordinates: State Plane (ft), N 2185939, E 6315555

Levee Mile: Sta. 162+50
River Mile: XXXX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 13.20
L/S Toe Elev.: 2.00
W/S Toe Elev.: 3.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 4/9/2013

Boring #	Blanket Thickness Variable (z)					Aquifer Thickness Variable (d)					Hydraulic Conductivity Vairables (Kb and Kf)								
	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Layer Thickness (ft)	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation	Blanket		Aquifer Material		Kf/Kb	Mean (MLV)	Standard Deviation	Variation	Coefficient of Variation
											Material	Kb (ft/day)	Material	Kf (ft/day)					
WR1608_005M	16	12	7	68	58	6	21	9	161	43	OH	0.0284	SM	1.134	40	482	496	218512	98
WR1608_013B	14					12					OH	0.0284	SP-SM	2.835	100				
WR1608_001B	4					28					OH	0.0284	SP-SM	2.835	100				
WR1608_017C	6					26					OH	0.0284	SP-SM	2.835	100				
WR1608_010B	6					32					CL	0.0028	SP-SM	2.835	1013				
WR1608_011B	22					20					CL	0.0028	SP-SM	2.835	1013				
WR1608_018C	18					24					CL	0.0028	SP-SM	2.835	1013				

Boring #	Blanket Material 1 (lowest permeability)			Blanket Material 2			Transformed Blanket Thickness (z)	Aquifer Material 1			Aquifer Material 2			Aquifer Material 3			Transformed Aquifer Horizontal Permeability (kf)
	Material Type	Thickness (z)	Permeability (Kb)	Material Type	Thickness (z)	Permeability (Kb)		Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	Material Type	Thickness (d)	Permeability (Kf)	
WR1608_005M	OH	16	0.0284				16	SM	6	1.134							1.134
WR1608_013B	OH	14	0.0284				14	SP-SM	12	2.835							2.835
WR1608_001B	OH	4	0.0284				4	SP-SM	28	2.835							2.835
WR1608_017C	OH	6	0.0284				6	SP-SM	26	2.835							2.835
WR1608_010B	CL	6	0.0028				6	SP-SM	32	2.835							2.835
WR1608_011B	CL	22	0.0028				22	SP-SM	20	2.835							2.835
WR1608_018C	CL	18	0.0028				18	SP-SM	24	2.835							2.835

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Underseepage Reliability Analysis With Blanket Theory Analysis

Project: Lower San Joaquin
Study Area: Delta Front Lincoln Village
River Section: Index Point D-LV
Coordinates: State Plane (ft), N 2185939, E 6315555

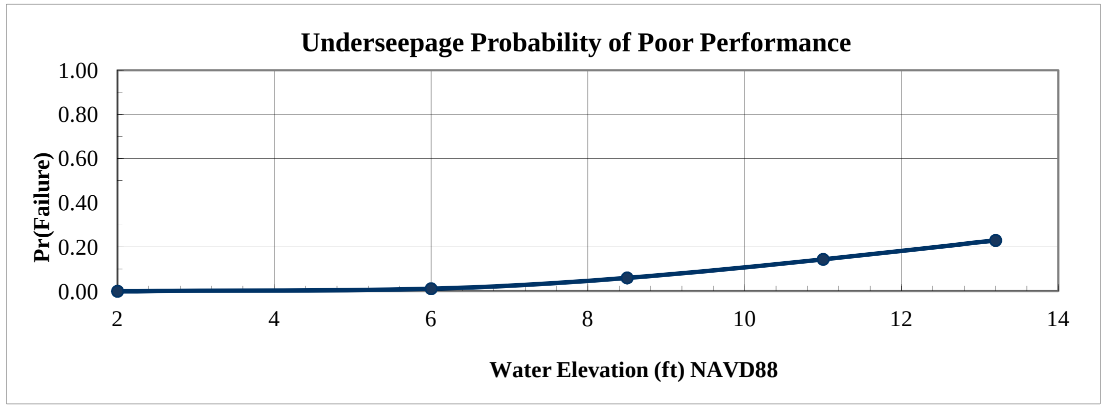
Levee Mile: Sta. 162+50
River Mile: XXXX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 13.20
L/S Toe Elev.: 2.00
W/S Toe Elev.: 3.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 4/9/2013

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Permaebility Ratio	482	472	98
Blanket Thickness (z)	12	7	58
Aquifer Thickness (d)	21	9	43

Blanket Theory Analysis Inputs					
Pr(f)=0	**BTA Case No.**	**L1**	**L2**	**L3**	**γ Blanket**
NO	7A	110	80	∞	112



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	2.00	0.0000
Elev. 6.0	4.00	6.00	0.0115
Elev. 8.5	6.50	8.50	0.0602
Elev. 11.0	9.00	11.00	0.1443
Crest	11.20	13.20	0.2299

Crest		**Rh**
Head =	11.20	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	482	12.00	21.00	106.49	348.52	0.0393	7.30	0.61		
2	954	12.00	21.00	108.19	490.41	0.0309	8.09	0.67	0.042025	19.43
3	10	12.00	21.00	48.16	49.29	0.1183	3.11	0.26		
4	482	19.00	21.00	107.75	438.54	0.0335	7.84	0.41	0.172225	79.63
5	482	5.00	21.00	102.00	224.97	0.0516	6.19	1.24		
6	482	12.00	30.00	107.51	416.56	0.0497	7.72	0.64	0.002025	0.94
7	482	12.00	12.00	104.02	263.45	0.0268	6.59	0.55		
								Total	0.216275	100.00

E[I] = 0.610000
E[ln I] = -0.723397
Var[I]= 0.216275
σ[I]= 0.465054
σ [ln I] = 0.676906
V(I) = 0.762383
β = -1.068682
F(z) = 0.770056
Ic= 0.80
ln(I crit) = -0.223144
Pr(f) % = 22.994448

Elev. 11.0		**Rh**
Head =	9.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	482	12.00	21.00	106.49	348.52	0.0393	5.86	0.49		
2	954	12.00	21.00	108.19	490.41	0.0309	6.50	0.54	0.027225	19.30
3	10	12.00	21.00	48.16	49.29	0.1183	2.50	0.21		
4	482	19.00	21.00	107.75	438.54	0.0335	6.30	0.33	0.112225	79.56
5	482	5.00	21.00	102.00	224.97	0.0516	4.98	1.00		
6	482	12.00	30.00	107.51	416.56	0.0497	6.21	0.52	0.001600	1.13
7	482	12.00	12.00	104.02	263.45	0.0268	5.30	0.44		
								Total	0.141050	100.00

E[I] = 0.490000
E[ln I] = -0.944419
Var[I]= 0.141050
σ[I]= 0.375566
σ [ln I] = 0.679807
V(I) = 0.766462
β = -1.389245
F(z) = 0.855655
Ic= 0.80
ln(I crit) = -0.223144
Pr(f) % = 14.434498

Elev. 8.5		**Rh**
Head =	6.50	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	482	12.00	21.00	106.49	348.52	0.0393	4.23	0.35		
2	954	12.00	21.00	108.19	490.41	0.0309	4.70	0.39	0.014400	19.83
3	10	12.00	21.00	48.16	49.29	0.1183	1.81	0.15		
4	482	19.00	21.00	107.75	438.54	0.0335	4.55	0.24	0.057600	79.31
5	482	5.00	21.00	102.00	224.97	0.0516	3.59	0.72		
6	482	12.00	30.00	107.51	416.56	0.0497	4.48	0.37	0.000625	0.86
7	482	12.00	12.00	104.02	263.45	0.0268	3.83	0.32		
								Total	0.072625	100.00

E[I] = 0.350000
E[ln I] = -1.282587
Var[I]= 0.072625
σ[I]= 0.269490
σ [ln I] = 0.682297
V(I) = 0.769972
β = -1.879807
F(z) = 0.939760
Ic= 0.80
ln(I crit) = -0.223144
Pr(f) % = 6.024031

Elev. 6.0		**Rh**
Head =	4.00	

Run	Kf/Kb	z	d	x1	x3	$	hx	I	Variance Component	% Variance
1 (Mean)	482	12.00	21.00	106.49	348.52	0.0393	2.61	0.22		
2	954	12.00	21.00	108.19	490.41	0.0309	2.89	0.24	0.005625	20.93
3	10	12.00	21.00	48.16	49.29	0.1183	1.11	0.09		
4	482	19.00	21.00	107.75	438.54	0.0335	2.80	0.15	0.021025	78.23
5	482	5.00	21.00	102.00	224.97	0.0516	2.21	0.44		
6	482	12.00	30.00	107.51	416.56	0.0497	2.76	0.23	0.000225	0.84
7	482	12.00	12.00	104.02	263.45	0.0268	2.36	0.20		
								Total	0.026875	100.00

E[I] = 0.220000
E[ln I] = -1.734952
Var[I]= 0.026875
σ[I]= 0.163936
σ [ln I] = 0.664566
V(I) = 0.745163
β = -2.610653
F(z) = 0.988543
Ic= 0.80
ln(I crit) = -0.223144
Pr(f) % = 1.145657

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Through-Seepage Reliability Analysis With Khilar's Extended Model

Project: Lower San Joaquin
Study Area: Delta Front Lincoln Village
River Section: Index Point D-LV
Coordinates: State Plane (ft), N 2185939, E 6315555

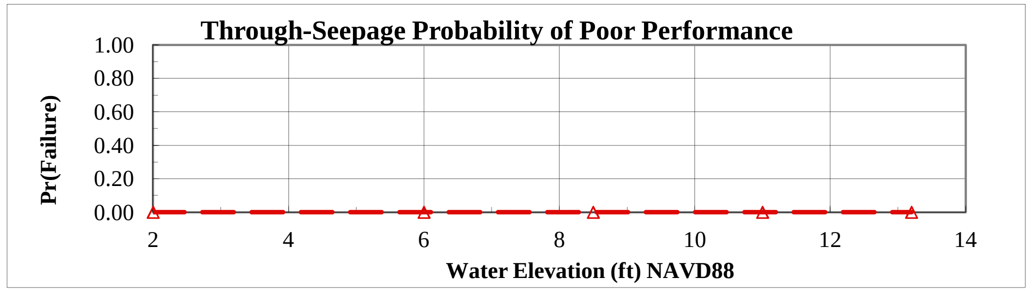
Levee Mile: Sta. 162+50
River Mile: XXXX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 13.20
L/S Toe Elev.: 2.00
W/S Toe Elev.: 3.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 4/9/2013

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Tractive Stress (Tc)	25	2.5	10.00
Initial Porosity (n)	0.5	0.05	10.00
Initial Permeability (Ko)	1.00E-10	3.00E-11	30.00

Pr(f)=0
NO



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	2.00	0.0000
Elev. 6.0	4.00	6.00	0.000000
Elev. 8.5	6.50	8.50	0.000000
Elev. 11.0	9.00	11.00	0.000000
Crest	11.20	13.20	0.000000

Crest	**Head =**	11.20	**Horizontal Gradient (Ix) =**	0.160

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	25.00	0.50	1.00E-10	637.40	3983.73		
2	22.50	0.50	1.00E-10	573.66	3585.35	158700.719428	26.44
3	27.50	0.50	1.00E-10	701.14	4382.10		
4	25.00	0.45	1.00E-10	604.69	3779.29	39774.866868	6.63
5	25.00	0.55	1.00E-10	668.51	4178.17		
6	25.00	0.50	7.00E-11	761.83	4761.46	401641.861129	66.93
7	25.00	0.50	1.30E-10	559.03	3493.96		
E[FS] =	3983.725887		E[ln FS] =	8.271414	Total	600117.447424	100.00

Var[FS]= 600117.447424
σ[FS]= 774.672478
σ[ln FS]= 0.192658
β = 42.933222
V(FS) = 0.194459
F(z) = 0.000000
FS req'd = 1.00
ln(FS req'd) = 0.000000
Pr(f) % = 0.000000

Elev. 11.0	**Head =**	9.00	**Horizontal Gradient (Ix) =**	0.140

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	25.00	0.50	1.00E-10	637.40	4552.83		
2	22.50	0.50	1.00E-10	573.66	4097.55	207282.572314	26.44
3	27.50	0.50	1.00E-10	701.14	5008.11		
4	25.00	0.45	1.00E-10	604.69	4319.19	51950.846521	6.63
5	25.00	0.55	1.00E-10	668.51	4775.05		
6	25.00	0.50	7.00E-11	761.83	5441.67	524593.451270	66.93
7	25.00	0.50	1.30E-10	559.03	3993.10		
E[FS] =	4552.829585		E[ln FS] =	8.404946	Total	783826.870105	100.00

Var[FS]= 783826.870105
σ[FS]= 885.339974
σ[ln FS]= 0.192658
β = 43.626324
V(FS) = 0.194459
F(z) = 0.000000
FS req'd = 1.00
ln(FS req'd) = 0.000000
Pr(f) % = 0.000000

Elev. 8.5	**Head =**	6.50	**Horizontal Gradient (Ix) =**	0.100

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	25.00	0.50	1.00E-10	637.40	6373.96		
2	22.50	0.50	1.00E-10	573.66	5736.57	406273.841735	26.44
3	27.50	0.50	1.00E-10	701.14	7011.36		
4	25.00	0.45	1.00E-10	604.69	6046.87	101823.659182	6.63
5	25.00	0.55	1.00E-10	668.51	6685.07		
6	25.00	0.50	7.00E-11	761.83	7618.34	1028203.164490	66.93
7	25.00	0.50	1.30E-10	559.03	5590.33		
E[FS] =	6373.961419		E[ln FS] =	8.741418	Total	1536300.665407	100.00

Var[FS]= 1536300.665407
σ[FS]= 1239.475964
σ[ln FS]= 0.192658
β = 45.372802
V(FS) = 0.194459
F(z) = 0.000000
FS req'd = 1.00
ln(FS req'd) = 0.000000
Pr(f) % = 0.000000

Elev. 6.0	**Head =**	4.00	**Horizontal Gradient (Ix) =**	0.010

Run	Tractive Stress (Tc)	Initial Porosity (n)	Initial Permeability (Ko)	Critical Gradient (Ic)	FS	Variance Component	% Variance
1 (Mean)	25.00	0.50	1.00E-10	637.40	63739.61		
2	22.50	0.50	1.00E-10	573.66	57365.65	40627384.173499	26.44
3	27.50	0.50	1.00E-10	701.14	70113.58		
4	25.00	0.45	1.00E-10	604.69	60468.71	10182365.918159	6.63
5	25.00	0.55	1.00E-10	668.51	66850.67		
6	25.00	0.50	7.00E-11	761.83	76183.41	102820316.449005	66.93
7	25.00	0.50	1.30E-10	559.03	55903.34		
E[FS] =	63739.614192		E[ln FS] =	11.044003	Total	153630066.540663	100.00

Var[FS]= 153630066.540663
σ[FS]= 12394.759640
σ[ln FS]= 0.192658
β = 57.324494
V(FS) = 0.194459
F(z) = 0.000000
FS req'd = 1.00
ln(FS req'd) = 0.000000
Pr(f) % = 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Landside Long-Term Stability Analysis With UTEXAS4

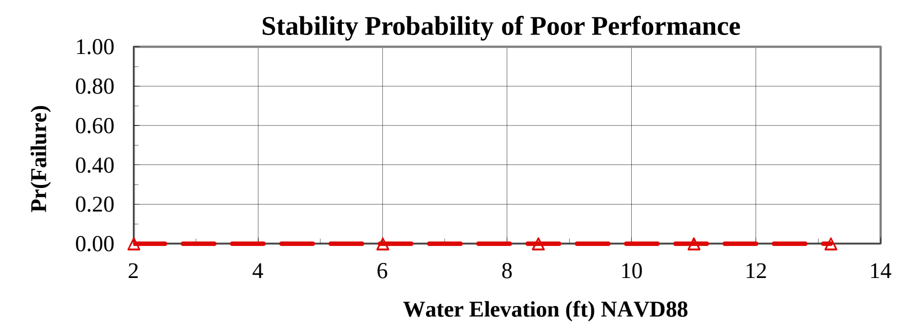
Project: Lower San Joaquin
Study Area: Delta Front Lincoln Village
River Section: Index Point D-LV
Coordinates: State Plane (ft), N 2185939, E 6315555

Levee Mile: Sta. 162+50
River Mile: XXXX
Analysis Case Without Project Conditions

Datum: NAVD 88
Crest Elev.: 13.20
L/S Toe Elev.: 2.00
W/S Toe Elev.: 3.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 4/9/2013

Random Variables			
Parameter	**Expected Value**	**Standard Deviation**	**Coefficient of Variation, %**
Levee Φ	27	4	13.00
Levee Cohesion	50	20	40.00
Levee γ	120	8	7.00
Foundation Φ	28	4	13.00
Foundation Cohesion	25	10	40.00



Analysis Case	Head	Elevation	Pr(f)
Toe	0.00	2.00	0.0000
Elev. 6.0	4.00	6.00	0.000000
Elev. 8.5	6.50	8.50	0.000000
Elev. 11.0	9.00	11.00	0.000000
Crest	11.20	13.20	0.000000

Crest	**Head =**	11.20	**Pr(f)=0**	NO

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	27	50	120	28	25	1.83		
2	23	50	120	28	25	1.74	0.003906	15.00
3	31	50	120	28	25	1.86		
4	27	30	120	28	25	1.79	0.001764	6.77
5	27	70	120	28	25	1.87		
6	27	50	112	28	25	1.87	0.001190	4.57
7	27	50	128	28	25	1.80		
8	27	50	120	24	25	1.62	0.018906	72.61
9	27	50	120	32	25	1.89		
10	27	50	120	28	15	1.81	0.000272	1.05
11	27	50	120	28	35	1.85		

E[FS] = 1.830000 E[ln FS] = 0.600443 Total 0.026039 100.00
Var[FS]= 0.026039
σ[FS]= 0.161366 σ[ln FS]= 0.088007 **β =** 6.822640
V(FS) = 0.088178 **F(z) =** 0.000000
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Elev. 11.0	**Head =**	9.00	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	27	50	120	28	25	1.94		
2	23	50	120	28	25			
3	31	50	120	28	25			
4	27	30	120	28	25			
5	27	70	120	28	25			
6	27	50	112	28	25			
7	27	50	128	28	25			
8	27	50	120	24	25			
9	27	50	120	32	25			
10	27	50	120	28	15			
11	27	50	120	28	35			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Elev. 8.5	**Head =**	6.50	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	27	50	120	28	25	2.04		
2	23	50	120	28	25			
3	31	50	120	28	25			
4	27	30	120	28	25			
5	27	70	120	28	25			
6	27	50	112	28	25			
7	27	50	128	28	25			
8	27	50	120	24	25			
9	27	50	120	32	25			
10	27	50	120	28	15			
11	27	50	120	28	35			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Elev. 6.0	**Head =**	4.00	**Pr(f)=0**	YES

Run	Levee Φ	Levee Cohesion	Levee γ	Foundation Φ	Foundation Cohesion	FS	Variance Component	% Variance
1 (Mean)	27	50	120	28	25	2.13		
2	23	50	120	28	25			
3	31	50	120	28	25			
4	27	30	120	28	25			
5	27	70	120	28	25			
6	27	50	112	28	25			
7	27	50	128	28	25			
8	27	50	120	24	25			
9	27	50	120	32	25			
10	27	50	120	28	15			
11	27	50	120	28	35			

E[FS] = E[ln FS] = Total
Var[FS]=
σ[FS]= σ[ln FS]= **β =**
V(FS) = **F(z) =**
FS req'd = 1.00 ln(FS req'd) = 0.000000 **Pr(f) % =** 0.000000

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Judgment Probability of Poor Performance Curve

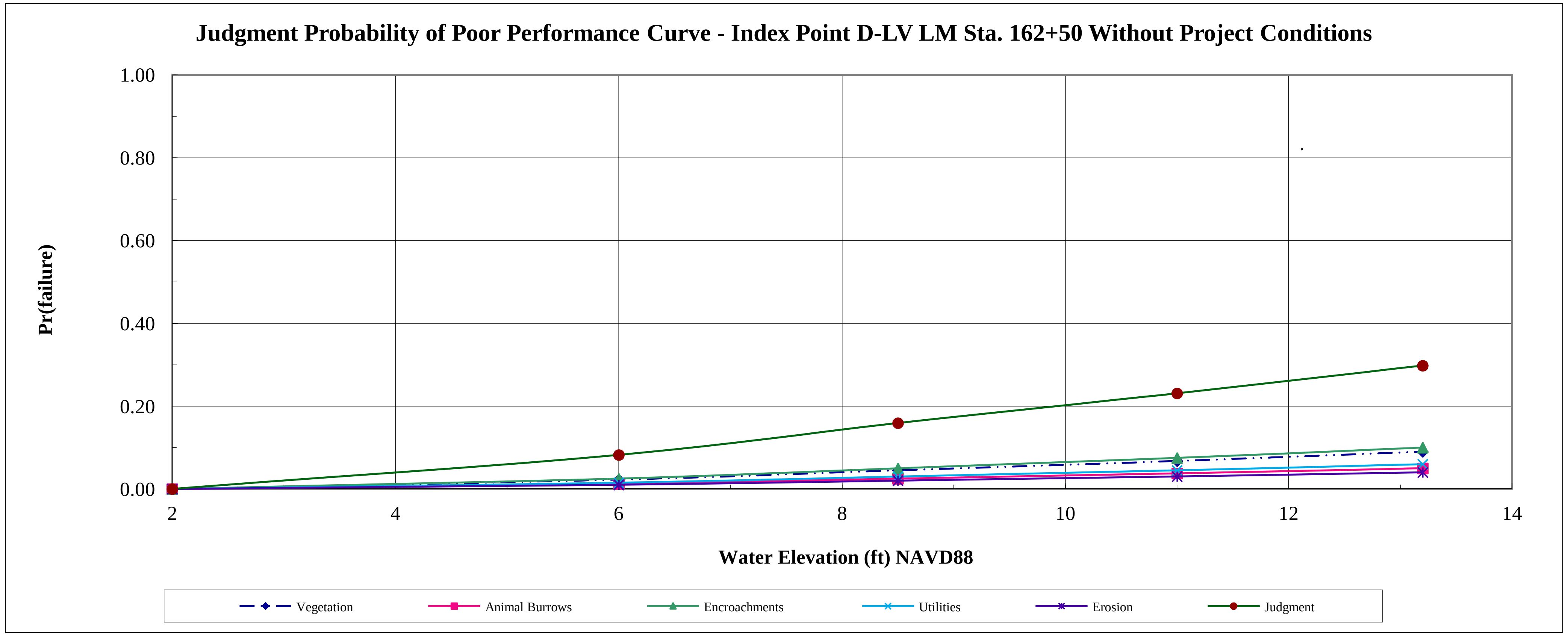
Project: Lower San Joaquin
Study Area: Delta Front Lincoln Village
River Section: Index Point D-LV
Coordinates: State Plane (ft), N 2185939, E 6315555

Levee Mile: Sta. 162+50
River Mile: XXXX
Analysis Case: Without Project Conditions

Datum: NAVD 88
Crest Elev.: 13.20
L/S Toe Elev.: 2.00
W/S Toe Elev.: 3.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perl
Date: 4/9/2013

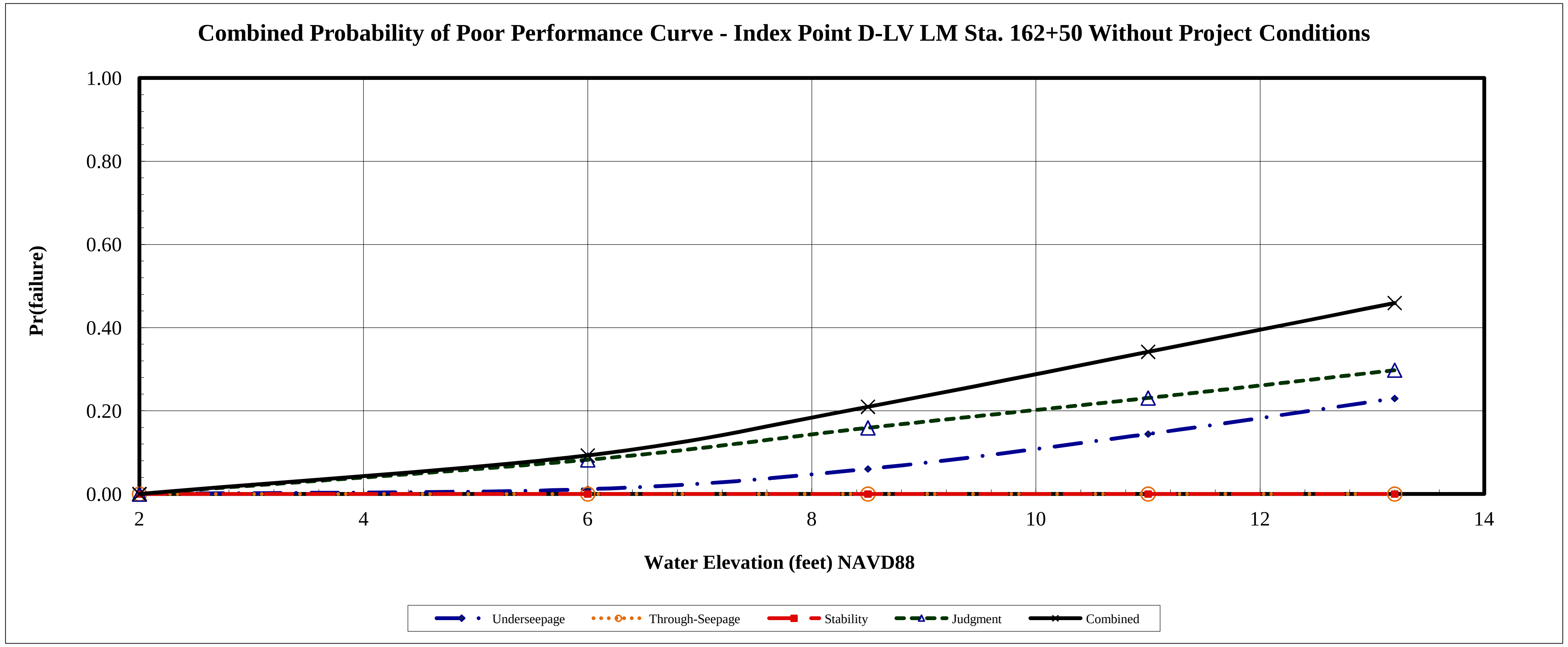
Water Surface Elevation	Vegetation		Animal Burrows		Encroachments		Utilities		Erosion		Judgment	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
2.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
6.00	0.0225	0.9775	0.0125	0.9875	0.0250	0.9750	0.0150	0.9850	0.0100	0.9900	0.0822	0.9178
8.50	0.0450	0.9550	0.0250	0.9750	0.0500	0.9500	0.0300	0.9700	0.0200	0.9800	0.1591	0.8409
11.00	0.0675	0.9325	0.0375	0.9625	0.0750	0.9250	0.0450	0.9550	0.0300	0.9700	0.2309	0.7691
13.20	0.0900	0.9100	0.0500	0.9500	0.1000	0.9000	0.0600	0.9400	0.0400	0.9600	0.2979	0.7021


Judgment Probability of Poor Performance Curve - Index Point D-LV LM Sta. 162+50 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

Project:	Lower San Joaquin			**Datum:**	NAVD 88		
Study Area:	Delta Front Lincoln Village	**Levee Mile:**	Sta. 162+50	**Crest Elev.:**	13.20	**Analysis By:**	G. Johnson
River Section:	Index Point D-LV	**River Mile:**	XXXX	**L/S Toe Elev.:**	2.00	**Checked By:**	J. Hogan, M. Perle
Coordinates:	State Plane (ft), N 2185939, E 6315555	**Analysis Case:**	Without Project Conditions	**W/S Toe Elev.:**	3.00	**Date:**	4/9/2013

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
2.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
6.00	0.0115	0.9885	0.0000	1.0000	0.0000	1.0000	0.0822	0.9178	0.0928	0.9072
8.50	0.0602	0.9398	0.0000	1.0000	0.0000	1.0000	0.1591	0.8409	0.2098	0.7902
11.00	0.1443	0.8557	0.0000	1.0000	0.0000	1.0000	0.2309	0.7691	0.3419	0.6581
13.20	0.2299	0.7701	0.0000	1.0000	0.0000	1.0000	0.2979	0.7021	0.4593	0.5407



LOWER SAN JOAQUIN RIVER
FEASIBILITY STUDY

GEOTECHNICAL REPORT

ENCLOSURE E4
SEISMIC AND LIQUEFACTION ANALYSES

LOWER SAN JOAQUIN LEVEE
Seismic Vulnerability Evaluation

1. Introduction and Scope

The purpose of this study was to assess the vulnerability to seismic action of the levees in the Lower San Joaquin Levee System. Some of the levees in the northern portion of the system are frequently hydraulically loaded and, therefore, their severe damaging due to a strong earthquake in vicinity may induce immediately loss of flood protection capability.

The vulnerability evaluation considered only the significant loss of strength of cohesionless or low plasticity soils through liquefaction due to dynamic loading. The liquefaction and seismic evaluation was focused on examining potential layers that could experience liquefaction and their associated impact to global slope stability of the levee. The computed factors of safety against slope stability refer exclusively to failure surfaces potentially affected by liquefaction; in some cases the static factor of safety can be lower than the computed factor of safety affected by liquefaction. The static stability, which can be controlled by the presence of weak cohesive soils was not within the scope of this analysis, even if the strength of these materials may be affected by the seismic action.

In most of the cases/segments it was determined that liquefaction was primarily isolated to the deeper foundation layers and that it had minimal effect on the global stability of the levee and foundation. In four of the examined cases only, three in RD 17 Unit and one in RD 404 Unit, the liquefiable layer was shallow enough such that it could pose a significant effect on the stability of the levee (list the locations).

Even though global instability resulting from liquefaction does not appear to be a primary concern when the layer is located at greater depths, there could be other seismic performance concerns given the geologic nature of the area and the potential for differential settlement. The foundations for many of the segments consist of numerous geomorphologic channels that run orthogonal to the levee axis. As a result there are variable foundation conditions along the axis of the levee. The variability of the foundation coupled with the potential for transverse cracking due to liquefaction and differential settlement is a concern and should be carefully considered in the alternatives evaluation.

2. Study Area and Sites Seismicity

The main units of the Lower San Joaquin Levee System are presented on Figure 1.1 and will be separately evaluated from the seismic vulnerability point of view:

- RD (River District) 17 – Southern part
- RD 17 – Northern part
- RD 404
- Calaveras River

- Stockton Diverting Canal
- Mormon Slough
- Brookside
- Lincoln Village

The USGS Interactive Deaggregations (Beta) accessible at the following URL address: https://geohazards.usgs.gov/deaggint/2008/ was used for the seismicity assessment at locations along the levee. The following parameters were used as input:

- Location, through latitude and longitude; the coordinates corresponding to each unit were used in evaluations.

- Exceedance probability of the seismic event within a given exposure period of time. The 20% exceedance probability in 50 years was selected, which corresponds approximately to the average return period (ARP) of 224 years. This was considered an appropriate approximation of the 200-year ARP recommended by California Department of Water Resources (DWR) for urban levee seismic evaluation (ULE).

- Spectral period. For liquefaction triggering evaluation the Peak Ground Acceleration (PGA) was the main desired result of the seismicity assessment.

- Shear wave velocity of the upper 30 m of the site (Vs_{30}).

Shear wave velocity measurements were not available; therefore, correlation with N (SPT) was used to estimate the median Vs_{30}; for each unit N_{60} was evaluated based on available deep borings, as shown in Appendix B.

Vs_{30} was evaluated through correlations with N_{60} available in literature, as shown in Figure 2.1 [Figures 23 for large data base of all types of soils and Figure 24 for granular soils, from USACE WES (1987)]. Based on these graphs, the data in Table 2-1 were suggested for use in this study and other evaluations.

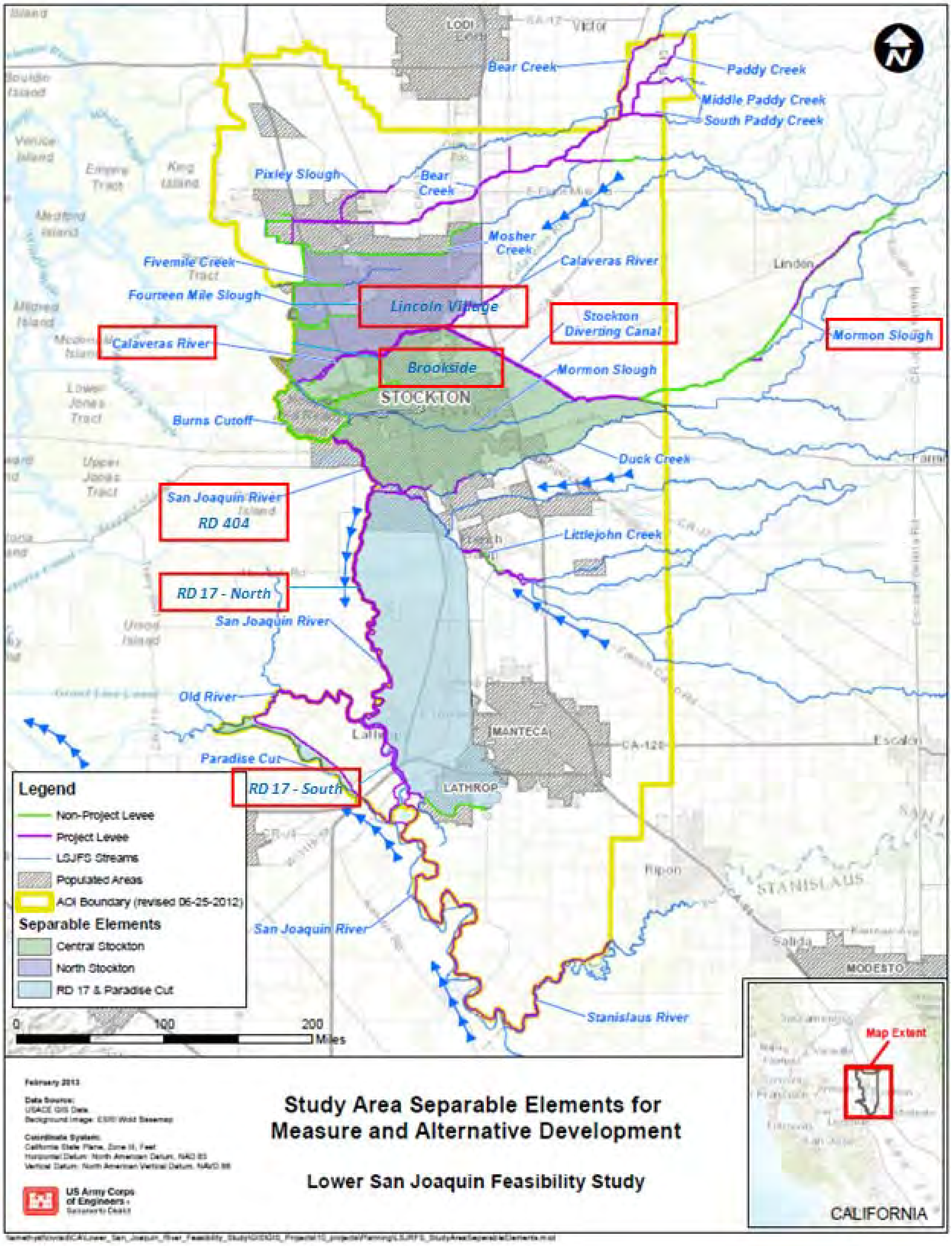


Figure 1.1. Main units of the Lower San Joaquin Levee System.

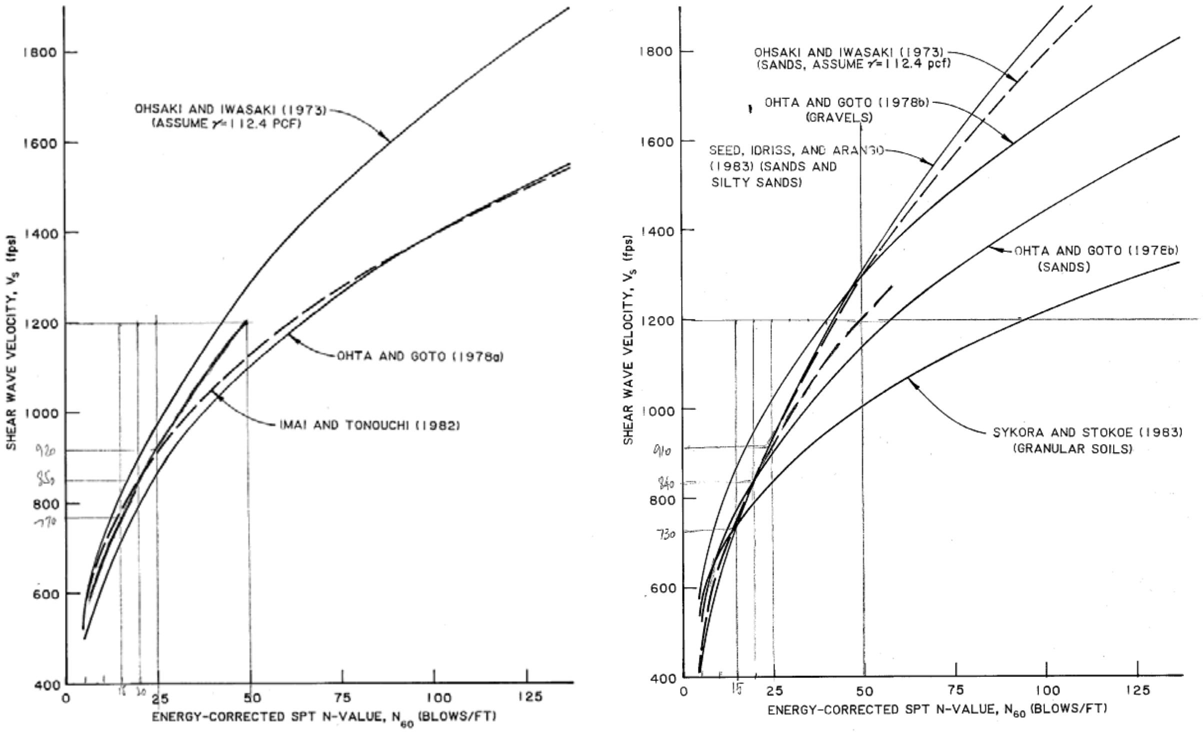


Figure 23. Comparison of results for N versus V_s correlations (proposed by select studies)

Figure 24. Comparison of results for N versus V_s correlations in granular soils (proposed by select studies)

Figure 2.1. Excerpt of USACE WES (1987): Average curves were considered that pass through the point represented by N_{60} = 50 and Vs = 1200 fps, which is the boundary between stiff soil and soft rock in USGS classification.

Table 2-1. Suggested Correlation between Vs and N_{60}

Mean N_{60}	Vs (m/s)	Mean N_{60}	Vs (m/s)	Mean N_{60}	Vs (m/s)	Mean N_{60}	Vs (m/s)
≤ 7	180*	15	230	23	270	32.5	308
8	181	16	235	24	275	35	317
9	189	17	241	25	279	37.5	326
10	197	18	246	26	283	40	334
11	204	19	251	27	287	42.5	342
12	211	20	256	28	291	45	349
13	217	21	261	29	295	50	364
14	224	22	266	30	299	100	474

Note: * The minimum Vs accepted by the USGS 2008 Interactive Deaggregations web program is 180 m/s, which corresponds to the boundary between stiff and soft soils (USGS Site Classes D and E).

In what follows the parameters for each units are listed, as well as the corresponding site seismicity parameters obtained from the USGS web site. Details on parameter evaluation are included in Appendix B.

2.1. **RD 17 – Southern part** (Stations 1480 to 1840)

Mid-point coordinates: latitude 37.809, longitude -121.321
Harmonic mean SPT – N_{60}: 21.6
Evaluated Vs_{30}: 265 m/s (detail in Appendix B)
Peak Ground Acceleration: 0.21g
Moment magnitude: 6.4

2.2. **RD 17 – Northern part** (Stations 1000 to 1480)

Mid-point coordinates: latitude 37.890, longitude -121.329
Harmonic mean SPT – N_{60}: 18.9
Evaluated Vs_{30}: 252 m/s (detail in Appendix B)
Peak Ground Acceleration: 0.225g
Moment magnitude: 6.4

2.3. **RD 404**

Mid-point coordinates: latitude 37.937, longitude -121.334
Harmonic mean SPT – N_{60}: 22.0
Evaluated Vs_{30}: 267 m/s (detail in Appendix B)
Peak Ground Acceleration: 0.20g
Moment magnitude: 6.4

2.4. **Calaveras River**

Western end coordinates: latitude 37.966, longitude -121.370
No deep boring was available; N_{60} and Vs_{30} were assumed as for RD 404.
Peak Ground Acceleration: 0.20g
Moment magnitude: 6.4

2.5. **Stockton Diverting Canal** and **Mormon Slough**

Western end coordinates: latitude 37.994, longitude -121.280
Eastern end coordinates: latitude 37.961, longitude -121.165
No deep boring was available; N_{60} and Vs_{30} were assumed as for RD 404.
Peak Ground Acceleration: 0.18g (0.165g for Mormon Slough)
Moment magnitude: 6.4

2.6. **Brookside** and **Lincoln Village**

Mid-point coordinates: latitude 38.014, longitude -121.370
No deep boring was available; N_{60} and Vs_{30} were assumed as for RD 404.
Peak Ground Acceleration: 0.20g
Moment magnitude: 6.4

3. First Screening.

It would have been no need for seismic evaluation if PGA < 0.1g; however, with the estimated PGA = 0.165g to 0.225g we should proceed with liquefaction assessment on all sections.

4. Water Level Conditions.

Two water elevations are of interest:

- Level of ground water when SPT's were done;
- Coincident water level with seismic action.

They were not readily available. For each zone the water level during investigation was approximated from piezometer readings at the same time of the year (sometimes in a different year than when the investigation had been done).

When information was available, the coincident water level was assumed the maximum occurred in a year without flood event; if this was not found, the conservative assumption of water at the ground surface was considered (i.e. unsaturated material in levee and saturated material – therefore potentially liquefiable – in the entire foundation soil).

The influence on the liquefaction assessment results of the ground water level during field testing is relatively minor. However, the assumed coincident water elevation (CWE) is of huge impact:

- Primarily because of relative location of some potentially liquefiable layers with respect to CWE: if these layers are above CWE they should be considered non-saturated and, therefore, non-liquefiable.
- Secondly, but not much less important, CWE has a major impact on the ratio between the total vertical stress and the effective vertical stress at the depth analyzed for liquefaction. The cyclic stress ratio (CSR) varies in direct proportionality with this ratio, which roughly can vary between 1.0 and 2.0. Consequently CSR may vary between simple and double depending on CWE and FS_{liq} may vary between a maximum value when CWE is exactly at the depth of evaluation and half of that when CWE is at the ground surface.

Taking into account the major impact of CWE selection, there is a low confidence in the calculated FS_{liq} when CWE is not well defined. The (believed) conservative assumption of CWE at the ground surface may be over-conservative. This aspect is detailed based on some actual evaluations in Appendix E.

5. Liquefaction Triggering Analysis.

The liquefaction triggering analysis was based on the procedure described in the summary report of the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, published as part of the Journal of Geotechnical and Geoenvironmental Engineering, dated October 2001 (Youd et al., October 2001). This is also the procedure recommended by the draft ETL 1110-2-580.

An Excel spreadsheet developed by the Geotechnical Branch, USACE Sacramento District, was used in this analysis. The corresponding procedure calculates the vertical stresses induced by the levee surcharge and takes them into account in normalization of N-data and, consequently, in calculation of the cyclic resistance ratio, CRR. However, these additional stresses were not included in the calculation of CSR, the cyclic stress ratio; therefore the calculated factor of safety against liquefaction corresponds to the free field, without the influence of the surcharge, for compliance with how PGA was defined. It is conservative to assume that if liquefaction would occur in free field it will also occur in the immediate vicinity of the levee and underneath it.

It was postulated that the materials labeled with soil type CL (based on either laboratory tests or visual examination by the field geologist) are not liquefiable. Although theoretically some cohesive soils, including some CL materials, may be susceptible to liquefaction, this possibility was not taken into account based on the relatively low seismicity of the zone. However, where Atterberg Limits were available, CL or ML materials were considered liquefiable when PI < 10.

6. Seismic Vulnerability Evaluation.

The results of seismic evaluations are presented in appendices as follows:

Appendix A shows primarily the location of the evaluated borings:

Plate 1 - RD 17 – Southern part (Stations 1480 to 1840):	8 borings
Plate 2 - RD 17 – Northern part (Stations 1000 to 1480):	11 borings
Plate 3 - RD 404:	10 borings
Plate 4 - Calaveras River:	9 borings
- Stockton Diverting Canal:	5 borings
- Mormon Slough:	2 borings
Plate 5 - Brookside:	9 borings
Plate 6 - Lincoln Village:	14 borings
Total analyzed borings:	68

Appendix B includes copies of Excel files used for the evaluation of harmonic mean N_{60}, correlated with the average shear wave velocity, at all locations were borings with SPT deeper than 100 feet were available:

RD 17 – Northern Part: average Vs_{30} based on 5 borings
RD 17 – Southern Part: average Vs_{30} based on 4 borings
RD 404: average Vs_{30} based on 1 boring

The results of the liquefaction triggering evaluation are presented in Appendix C. Each plot of the factor of safety against liquefaction with depth is followed by the corresponding Excel spreadsheet. Only the first spreadsheet (for boring WR0017_063B) includes the bottom notes; however, they apply to all spreadsheets. A summary of the results follows. In the tables corresponding to each unit, the locations where liquefaction was found probable under the assumption of design earthquake occurrence had the boring number shown in bold on shaded background and the corresponding boring log was included in Appendix D.

6.1. **RD 17 – Southern part** (Stations 1480 to 1840)

Station	Boring	Figure	CWE	Comments
1506+19	WR0017_063B	C-1	8.0	Mostly clayey soils in the upper 40 feet of foundation. No SPT data for thin cohesionless layers. Marginally liquefiable soil 40+ feet below the levee base.
1553+82	**WR0017_069B** (see App. D)	C-2	8.7	One test showed potentially liquefiable soil; both above and below that, the soil was found marginally liquefiable.
1595+33	**WR0017_074B** (see App. D)	C-3	7.7	Liquefaction predicted at two depths and marginally liquefiable soil above, below, and in-between. A 12-foot layer is clearly liquefiable.
1642+75	WR0017_080B	C-4	7.5	No liquefaction predicted. Marginally liquefiable soil immediately below CWE.
1684+57	WR0017_085B	C-5	7.4	No liquefaction predicted.
1724+68	WR0017_090B	C-6	7.1	No liquefaction predicted.
1784+83	WR0017_096B	C-7	6.8	No liquefaction predicted. Marginally liquefiable soil immediately below CWE.
1825+94	WR0017_102B	C-8	6.8	No liquefaction predicted. Marginally liquefiable soil between elevations -15 and -25 and at about elevation -31. See Appendix E for the effect of CWE selection.

6.2. **RD 17 – Northern part** (Stations 1000 to 1480)

Station	Boring	Figure	CWE	Comments
1007+42	WR0017_002B	C-9	2.8	Mostly clayey soils in the upper 50 feet of foundation. Thin marginally liquefiable SM layer at approximately elevation -31.0. See Appendix E for the effect of CWE selection.
1048+79	WR0017_007B	C-10	2.8	No liquefaction predicted. See Appendix E for the effect of CWE selection.

Station	Boring	Figure	CWE	Comments
1099+90	WR0017_013B	C-11	3.6	No liquefaction predicted. Mostly clayey soils in the upper 50 feet of foundation.
1151+06	**WR0017_019B** (see App. D)	C-12	4.4	A liquefiable layer was detected between elevations +1 and -2.
1191+43	**WR0017_024B** (see App. D)	C-13	4.6	Two liquefiable layers were detected: one at about elevation -3.0 and another one between elevations -13.2 and -20.3.
1231+82	WR0017_029B	C-14	4.8	No liquefaction predicted. Marginally liquefiable SM soils were detected through tests at elevations +1 and -4.
1292+29	WR0017_036B	C-15	4.8	No liquefaction predicted. Marginally liquefiable SM soils were detected through tests at elevations -26 and -31.
1330+01	WR0017_041B	C-16	5.0	No liquefaction predicted. See Appendix E for the effect of CWE selection.
1377+73	WR0017_047B	C-17	5.3	No liquefaction predicted. Marginally liquefiable soils were detected through tests at elevations +1, -20 and -24. See Appendix E for the effect of CWE selection.
1416+93	WR0017_052B	C-18	5.5	No liquefaction predicted.
1455+64	WR0017_057B	C-19	7.0	No liquefaction predicted. Marginally liquefiable soils were detected through tests at elevations +7 and -8.

6.3. **RD 404**

Station	Boring	Figure	CWE	Comments
1003+04	WR0404_030B	C-20	0.0	No liquefaction predicted. Clayey soils with PI of 10 or greater were detected in the upper 44 feet of foundation.
1201+00	WR0404_040B	C-21	4.1	No liquefaction predicted. Marginally liquefiable soil (part of test possibly in CL material) was detected at elevation -25.
1175+01	**WR0404_041B** (see App. D)	C-22	4.1	Liquefiable SW-SM layer between elevations +1.3 and -4.7 was detected through one test at elevation -1.1.
1139+55	WR0404_044B	C-23	0.0	No liquefaction predicted.
1112+49	WR0404_047B	C-24	0.0	No liquefaction predicted. One marginally liquefiable spot was found at elevation -47, too deep for affecting the levee.
1108+07	WR0404_048B	C-25	0.0	No liquefaction predicted. Mostly clayey soils or ML with PI = 10 were detected in the upper 60 feet of foundation.

Station	Boring	Figure	CWE	Comments
1087+77	WR0404_053B	C-26	0.0	No liquefaction predicted.
1070+28	WR0404_056B	C-27	0.0	No liquefaction predicted. A shallow marginally liquefiable SM/ML layer was detected at the approximate elevation -2.
1042+70	WR0404_059B	C-28	0.0	No liquefaction predicted.
1028+00	WR0404_060B	C-29	0.0	No liquefaction predicted.

From the above table it is evident the levees in the unit RD 404 have a low seismic vulnerability. Only one of the ten analyzed borings predicted liquefaction occurrence (10%). The ten analyzed borings had sufficient SPT information (especially with reference to type of sampler and delivered energy efficiency).

Recently URS performed a similar study on the levees of RD 404, analyzing 22 borings. Of these 22 borings, 17 (77%) predicted liquefaction. In general, the results obtained by the Corps and URS on the same borings were similar. Most of them did not predict liquefaction; however, in two cases in which the Corps did not consider liquefaction because the material was CL, URS found that the PI was less than 10 so liquefaction was determined to be possible. The big difference was that URS analyzed several borings that the Corps did not have access to; including, where multiple tests (up to 6 in some borings) with predicted liquefaction: Borings 1-B2, 1-B4, 1-B5, 1-B6, 1-B8, 1-B9, 1-B12, WR0404_003B, _015B, _018B, _023B, _032B, _053B, and _061B.

The length of levees (on each side of the San Joaquin River) of RD 404 is about 22,000 feet; therefore, with ten analyzed borings the average distance between them was 2200 feet (actually the distance between borings was up to 4200 feet). Such “spot checking” may not detect problem zones if they are of local extent. It should be noted that all borings the Corps did not have access to but that URS analyzed showed liquefaction potential. They may have included incomplete characterization and conservative assumptions (e.g. with respect to energy efficiency).

6.4. **Calaveras River**

Station	Boring	Figure	CWE	Comments
6505+30	WR1614_017B	C-30	3.4*	No liquefaction predicted. A blowcount of zero at elevation -35 indicated FS_{liq} = 0.53, but it was in soil with PI = 61.
3072+94	WR2074_016B	C-31	-1.0	No liquefaction predicted.
3087+75	WCNBCR_010B	C-32	-1.0	No liquefaction predicted.
6565+02	**WR1614_018B** (see App. D)	C-33	1.4*	An SP-SM layer between elevations -18.4 and -23 was determined as liquefiable (FS_{liq} = 0.4).
3130+53	WCNBCR_011B	C-34	-1.0	No liquefaction predicted.
3156+02	WCNBCR_012B	C-35	-1.0	No liquefaction predicted. Marginally liquefiable material (FS_{liq} = 1.08) was found at elevation -14.

Station	Boring	Figure	CWE	Comments
6669+40	**WR1614_019B** (see App. D)	C-36	4.0	Liquefiable material (FS_{liq} = 0.6) was found at elevation -12 (layer -10.8 to -16.0).
3238+00	WCNBCR_013B	C-37	-1.0	No liquefaction predicted.
6762+29	WCSBCR_004B	C-38	3.0	No liquefaction predicted.

Note: * CWE could not be evaluated and was conservatively considered at the ground surface elevation.

6.5. **Stockton Diverting Canal** and **Mormon Slough**

Station	Boring	Figure	CWE	Comments
811+98	WCSBDC_001B	C-39	24.8*	No liquefaction predicted.
883+93	WCSBDC_005B	C-40	24.2*	No liquefaction predicted.
940+82	WCSBDC_008B	C-41	27.4*	No liquefaction predicted.
978+49	WCSBDC_013B	C-42	33.0*	No liquefaction predicted.
1029+16	WCSBDC_014B	C-43	35.0*	No liquefaction predicted.
2527+95	WCSBMS_003B	C-44	44.0*	No liquefaction predicted.
2583+28	WCSBMS_002B	C-45	51.4*	No liquefaction predicted.

Note: * CWE could not be evaluated and was conservatively considered at the ground surface elevation or slightly (less than 1 foot) below.

From the above table it is evident that liquefaction was not predicted even with a very conservative CWE assumed. Therefore, it was not necessary to evaluate a more credible CWE along Stockton Diverting Canal and Mormon Slough.

6.6. **Brookside**

Station	Boring	Figure	CWE	Comments
117+51	**WR2074_003M** (see App. D)	C-46	3.2	There are two liquefiable layers: between elevations -15.5 and -18 and between elevations -21 and -23 (the deeper layer was disregarded).
118+02	**WR2074_009B** (see App. D)	C-47	1.1	There is one liquefiable layer between elevations -22.4 and -31.9.
133+44	WR2074_010B	C-48	-0.6*	No liquefaction predicted.
133+82	**WR2074_007B** (see App. D)	C-49	5.5*	There are two 2-foot liquefiable layers: between elevations -9.8 and -11.8 and between elevations -20.8 and -22.8 (FS_{liq} = 0.99 in both cases).
160+48	WR2074_011B	C-50	0.6*	No liquefaction predicted.
185+70	WR2074_008B	C-51	1.1	No liquefaction predicted. Marginal liquefiability (FS_{liq} = 1.23) was detected at elevation -28.5.
217+77	WR2074_012B	C-52	0.9*	No liquefaction predicted.

Station	Boring	Figure	CWE	Comments
247+31	WR2074_013B	C-53	-1.1*	No liquefaction predicted.
248+41	WR2074_005M	C-54	3.2	No liquefaction predicted. Marginal liquefiability (FS_{liq} = 1.27) was detected at elevation -17.6.

Note: * CWE was considered at the ground surface.

6.7. **Lincoln Village**

Station	Boring	Figure	CWE	Comments
5+23	WR1608_002B	C-55	5.4*	No liquefaction predicted. However, only one SPT was performed for 16 feet of cohesionless soil.
43+00	**WR1608_002M** (see App. D)	C-56	3.3*	Liquefiable SM layer was detected between elevations -10.7 and -26.7.
43+58	WR1608_001M	C-57	3.3*	No liquefaction predicted. Marginally liquefiable SM layer (FS_{liq} = 1.01 and 1.11) was found between elevations -7.4 and -26.7, probably the same as the SM above.
50+79	WR1608_004B	C-58	5.4*	No liquefaction predicted. Marginally liquefiable SP-SM layer (FS_{liq} = 1.19 and 1.02, based on Standard California sampler**) was found between elevations -7.1 and -26.6, probably the same as the SM above.
89+65	WR1608_004M	C-59	5.7*	No liquefaction predicted. However, the boring penetrated 22 feet only in foundation soil (down to elevation -16.5).
89+67	WR1608_003M	C-60	4.8*	No liquefaction predicted. Except for a 2-foot non-liquefiable cohesionless layer, only clayey soils were encountered down to 40 feet in depth (elevation -36.7).
109+90	**WR1608_008B** (see App. D)	C-61	1.0*	A thin liquefiable layer was detected (FS_{liq} = 0.89, based on Standard California sampler**).
150+00	WR1608_013B	C-62	3.2*	No liquefaction predicted. A marginally liquefiable SP-SM layer was detected (FS_{liq} = 1.25, based on Standard California sampler**).
159+20	WR1608_001B	C-63	3.1*	No liquefaction predicted. A marginally liquefiable SP-SM layer was detected (FS_{liq} = 1.27, based on Standard California sampler**).
159+41	WR1608_009B	C-64	4.1*	No liquefaction predicted. A marginally liquefiable SM layer was detected (FS_{liq} = 1.27, based on Standard California sampler**).

Station	Boring	Figure	CWE	Comments
159+48	**WR1608_010B** (see App. D)	C-65	3.7*	Liquefiable SM or SP-SM layer was detected between elevations -7.8 and -25.3.
164+99	**WR1608_011B** (see App. D)	C-66	3.6*	Liquefiable ML layer was detected between elevations -27.4 and -30.4. Marginally liquefiable layers were detected both above and below the liquefiable layer, but separated by non-liquefiable layers.
142+28	WR1608_005M	C-67	4.9*	No liquefaction predicted. Marginally liquefiable SM layers were found both above and below a cohesive layer.
201+51	**WCNBFM_001B** (see App. D)	C-68	6.6*	Liquefiable SM layer was detected between elevations -17 and -27. Marginally liquefiable soil was found at elevation -3.

Notes: * CWE could not be evaluated and was conservatively considered at the ground surface elevation.

** Many SPT's at Lincoln Village unit were performed with a "Standard California" sampler (also known as Dames & Moore sampler). A factor of 0.55 was applied to blowcounts obtained with the California sampler for converting them to regular SPT; however, there is a large scatter in correlation data; also ASTM D6066 "Determining the Normalized Penetration Resistance of Sands for Evaluation of Liquefaction Potential" states: "6.3.3 Larger diameter split barrel samplers, 3 and 3½-in. (75 and 88 mm) O.D., can be used with and without retainers to recover coarse grained soils. They are not acceptable for determining penetration resistance *N* values." Therefore, conventional SPT data were always preferred and Standard California data (multiplied by 0.55) were used only when regular SPT's were not at all available in a particular boring or in other borings in vicinity.

7. Post-Earthquake Stability Evaluation.

7.1. **General**

In accordance with draft ETL 1110-2-580 "Guidelines for Seismic Evaluation of Levees" at all locations where liquefaction potential was detected a post-earthquake stability analysis should be performed assuming residual shear strength mobilized in all potentially liquefiable layers. This analysis was performed using UTexas4 and the results are presented in Appendix F.

In accordance with the above referenced ETL, the selection of the residual strength should be done based on two state-of-the-practice procedures and selecting the lowest obtained factor of safety as final result.

The two state-of-the-practice procedures for the evaluation of the residual (post-liquefaction) undrained shear strength, S_r, of soils were: Seed and Harder, 1990 and Olson and Stark, 2002. (See references in ETL 1110-2-580.) An average relationship (actually corresponding to the lower third of the specified range) for the first procedure was recommended by Idriss and Boulanger, 2007:

a. <u>Seed and Harder, 1990 approach</u>:

$$S_r = \exp\{(N_1)_{60cs\text{-}Sr} / 5.1 - [(N_1)_{60cs\text{-}Sr} / 16.5]^2 + [(N_1)_{60\text{-}cs\text{-}Sr} / 21.4]^3 + 0.8\} / 0.0479 \quad \text{(psf)}$$

where:
$(N_1)_{60cs\text{-}Sr} = (N_1)_{60cs} + \Delta(N_1)_{60cs\text{-}Sr}$
and
$\Delta(N_1)_{60cs\text{-}Sr}$ is a function of fines content, as shown in Table 7-1.

Table 7-1. Correction for Fines

Fines Content, F (% < 0.074 mm)	$\Delta(N_1)_{60cs\text{-}Sr}$
≤ 5	0
10	1
25	2
50	4
75	5

Interpolation between values in table was based on the curve and equation in Figure 7.1.

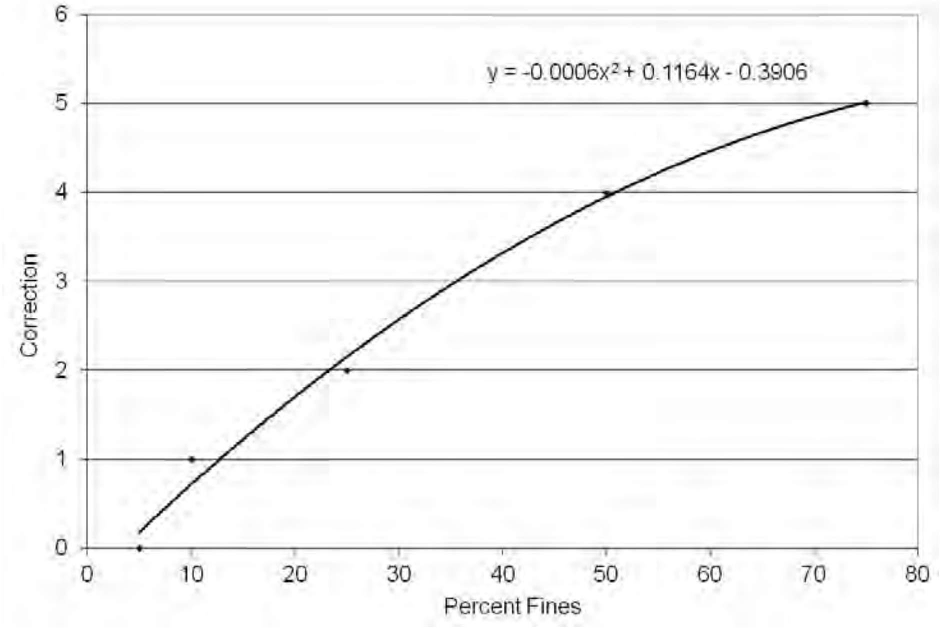


Figure 7.1. Correction for fines.

The undrained shear strength obtained through the Seed and Harder, 1990 procedure is presented in graphical form in Figure 7.2.

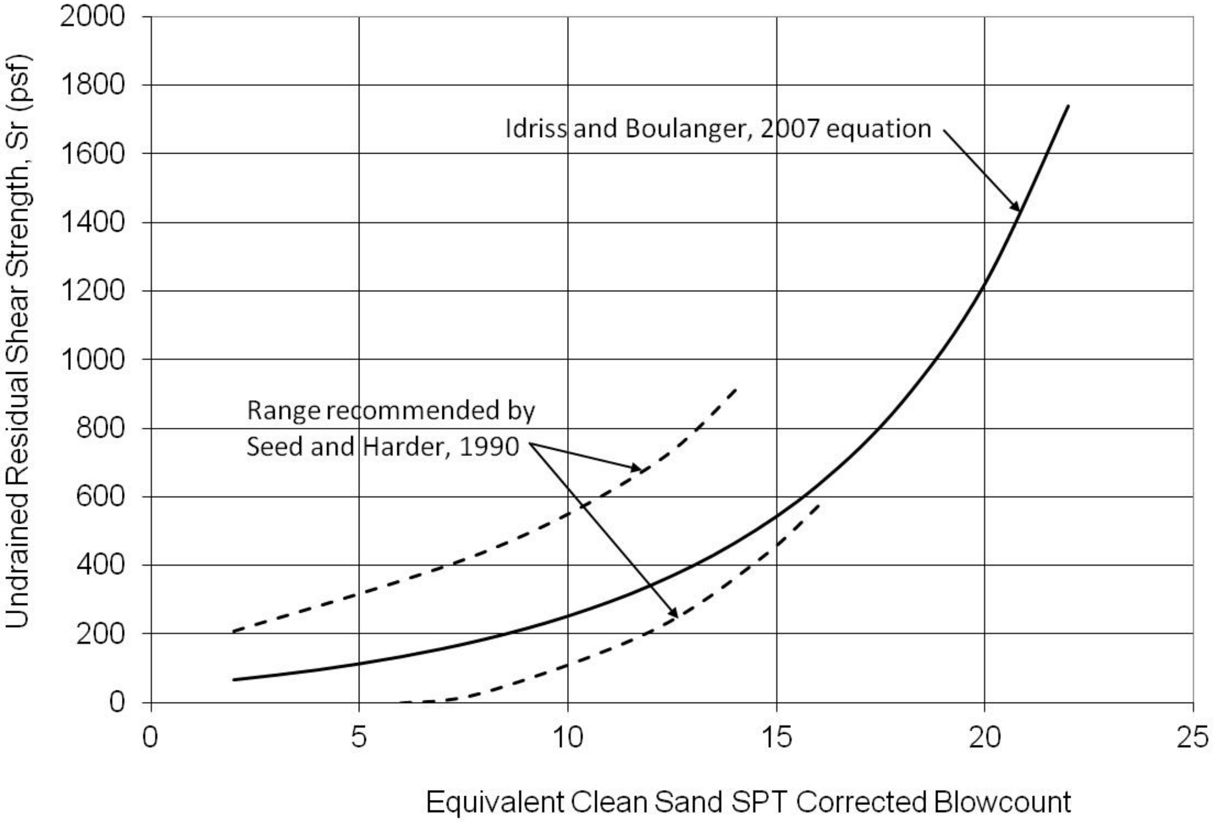


Figure 7.2. Results of Idriss and Boulanger, 2007 equation for approximation of Seed and Harder, 1990 procedure.

b. <u>Olson and Stark, 2002 approach</u>:

$S_r/\sigma'_{v0} = 0.03 + 0.0075\ [(N_1)_{60}]$

(Note that no correction for fines is applied.)

The calculated S_r, which under this definition varies with depth, was input in the limit equilibrium evaluations as an equivalent Φ-angle defined as follows:

$\Phi_{eq} = \tan^{-1}(S_r/\sigma'_{v0})$ and $S_r = \tan \Phi_{eq} * \sigma'_{v0}$

The results are summarized below. The minimum factors or safety are shown in bold if they are less than one; they are also shown on shaded background if they are critical for a given variant. Therefore, a shaded zone on a line identifies location where the levee can fail during a 200-year earthquake.

7.1. **RD 17 – Southern part** (Stations 1480 to 1840)

a. <u>Seed and Harder, 1990 approach</u>:

Station	Boring	Liquefiable Layer(s)		Factor of safety (FS)			
		Elevations	Sr (psf)	Water Side		Land Side	
				Circle	Wedge	Circle	Wedge
1553+82	69B	+1.3 to -5.7	365	**0.84**	**0.95**	1.49	1.61
1595+33	74B	+10.0 to -2.0	133	1.07*	1.19	1.26	1.26

Note: * Critical slip circle does not affect the levee.

b. <u>Olson and Stark, 2002 approach</u>:

Station	Boring	Liquefiable Layer(s)		Factor of safety (FS)			
		Elevations	Φ_{eq} (degrees)	Water Side		Land Side	
				Circle	Wedge	Circle	Wedge
1553+82	69B	+1.3 to -5.7	6.9	**0.37**	**0.80**	1.29	1.37
1595+33	74B	+10.0 to -2.0	3.9	0.95*	1.07	1.32	1.27

Note: * Critical slip circle does not affect the levee.

7.2. **RD 17 – Northern part** (Stations 1000 to 1480)

a. <u>Seed and Harder, 1990 approach</u>:

Station	Boring	Liquefiable Layer(s)		Factor of safety (FS)			
		Elevations	Sr (psf)	Water Side		Land Side	
				Circle	Wedge	Circle	Wedge
1151+06	19B	+1.0 to -2.0	201	1.00	1.15	1.93	1.59
1191+43	24B	-2.7 to -3.7 -13.2 to -20.3	164 111	**0.88**	1.38	1.62	1.31

b. <u>Olson and Stark, 2002 approach</u>:

Station	Boring	Liquefiable Layer(s)		Factor of safety (FS)			
		Elevations	Φ_{eq} (degrees)	Water Side		Land Side	
				Circle	Wedge	Circle	Wedge
1151+06	19B	+1.0 to -2.0	5.2	**0.87**	1.15	1.86	1.54
1191+43	24B	-2.7 to -3.7 -13.2 to -20.3	4.3 2.7	1.19	1.37	1.61	1.60

7.3. **RD 404**

a. Seed and Harder, 1990 approach:

Station	Boring	Liquefiable Layer(s)		Factor of safety (FS)			
		Elevations	Sr (psf)	Water Side		Land Side	
				Circle	Wedge	Circle	Wedge
1175+01	41B	+1.3 to -4.7	113	**0.88**	**0.73**	1.40	1.15

b. Olson and Stark, 2002 approach:

Station	Boring	Liquefiable Layer(s)		Factor of safety (FS)			
		Elevations	Φ_{eq} (degrees)	Water Side		Land Side	
				Circle	Wedge	Circle	Wedge
1175+01	41B	+1.3 to -4.7	3.6	**0.82**	**0.65**	1.38	1.12

7.4. **Calaveras River**

a. Seed and Harder, 1990 approach:

Station	Boring	Liquefiable Layer(s)		Factor of safety (FS)			
		Elevations	Sr (psf)	Water Side		Land Side	
				Circle	Wedge	Circle	Wedge
6565+02	18B	-18.4 to -23.0	77	1.76	1.40	N/A	N/A
6669+40	19B	-10.8 to -16.0	98	2.10	1.97	N/A	N/A

b. Olson and Stark, 2002 approach:

Station	Boring	Liquefiable Layer(s)		Factor of safety (FS)			
		Elevations	Φ_{eq} (degrees)	Water Side		Land Side	
				Circle	Wedge	Circle	Wedge
6565+02	18B	-18.4 to -23.0	2.6	1.80	1.45	N/A	N/A
6669+40	19B	-10.8 to -16.0	1.7	2.04	1.86	N/A	N/A

7.5. **Stockton Diverting Canal** and **Mormon Slough**

No potential liquefaction was detected.

7.6. **Brookside**

a. Seed and Harder, 1990 approach:

Station	Boring	Liquefiable Layer(s)		Factor of safety (FS)			
		Elevations	Sr (psf)	Water Side		Land Side	
				Circle	Wedge	Circle	Wedge
117+51	3M	-15.5 to -18.0	189	N/A	N/A	3.78	3.14
118+02	9M	-22.4 to -31.9	151	N/A	N/A	2.17	1.58
133+82	7B	-9.8 to -11.8	242	N/A	N/A	1.62	1.68

b. Olson and Stark, 2002 approach:

Station	Boring	Liquefiable Layer(s)		Factor of safety (FS)			
		Elevations	Φ_{eq} (degrees)	Water Side		Land Side	
				Circle	Wedge	Circle	Wedge
117+51	3M	-15.5 to -18.0	4.3	N/A	N/A	3.69	2.95
118+02	9M	-22.4 to -31.9	4.3	N/A	N/A	2.21	1.71
133+82	7B	-9.8 to -11.8	5.1	N/A	N/A	1.48	1.49

7.7. **Lincoln Village**

a. Seed and Harder, 1990 approach:

Station	Boring	Liquefiable Layer(s)		Factor of safety (FS)			
		Elevations	Sr (psf)	Water Side		Land Side	
				Circle	Wedge	Circle	Wedge
43+57	2M	-10.7 to -26.7	201	1.67	1.61	1.55	1.52
109+90	8B	-13.0 to -16.0	282	1.60	1.49	2.01	2.31
159+48	10B	-7.8 to -25.3	207	1.68	1.64	1.40	1.42
164+99	11B	-27.4 to -30.4	224	4.47	4.03	3.79	3.22
201+51	1B	-17.0 to -27.0	201	3.83	4.01	3.59	4.05

b. Olson and Stark, 2002 approach:

Station	Boring	Liquefiable Layer(s)		Factor of safety (FS)			
		Elevations	Φ_{eq} (degrees)	Water Side		Land Side	
				Circle	Wedge	Circle	Wedge
43+57	2M	-10.7 to -26.7	4.7	1.58	1.53	1.41	1.42
109+90	8B	-13.0 to -16.0	6.0	1.44	1.27	1.84	1.63
159+48	10B	-7.8 to -25.3	5.1	1.53	1.51	1.24	1.21
164+99	11B	-27.4 to -30.4	3.4	4.36	3.86	3.69	3.04
201+51	1B	-17.0 to -27.0	4.7	3.65	4.01	3.41	3.75

The following sections have been identified as susceptible of flow failures under the loading with the 200-year earthquake; therefore, immediately after the earthquake occurrence the levee flood retention capability may be compromised:

RD 17 – Southern part	1553+82
RD 17 – Northern part	1151+06 1191+43
RD 404	1175+01

The following section has the minimum factor of safety between 1.0 and 1.2, so the levee at this location may experience significant deformation under the loading with the 200-year earthquake:

RD 17 – Southern part	1595+33

However, the factor of safety is marginally 1.2 (1.19 and 1.07 with residual strength per Seed and Harder, 1990 and per Olson and Stark, 2002 for a very shallow potential failure surface); therefore, additional deformation analysis was not considered necessary for this location.

8. Conclusions.

Fifteen of the 68 borings evaluated indicated potentially liquefiable material under the 200-year earthquake loading. It is noted that not all layers had SPT's and in some cases the less reliable tests with the Standard California sampler had to be considered. However, the upper 50 feet of the soil were found generally non-liquefiable, including non-liquefiable cohesive soils that are predominant.

The fifteen locations with possible liquefaction occurrence were evaluated for post-earthquake stability. In three cases the potential for flow failure, i.e. complete loss of levee capability for flood protection were found. Four locations with potential flow failure condition were found in units RD 17 and RD 404. The corresponding segments of levees should be further investigated for potential vulnerability.

The rest of levee units will likely not be affected by the 200-year design level earthquake. This is due to both the relatively rare presence of liquefiable layers and in some cases their depth. In general, it was found that the layer was only vulnerable if the liquefiable layer was above or slightly below the elevation 0.0, i.e. at shallow depth in foundation. For these cases the levee was found vulnerable to the seismic action.

Report prepared by
Vlad Perlea
Laszlo Nagy
Soil Design Section

APPENDIX A

Plates

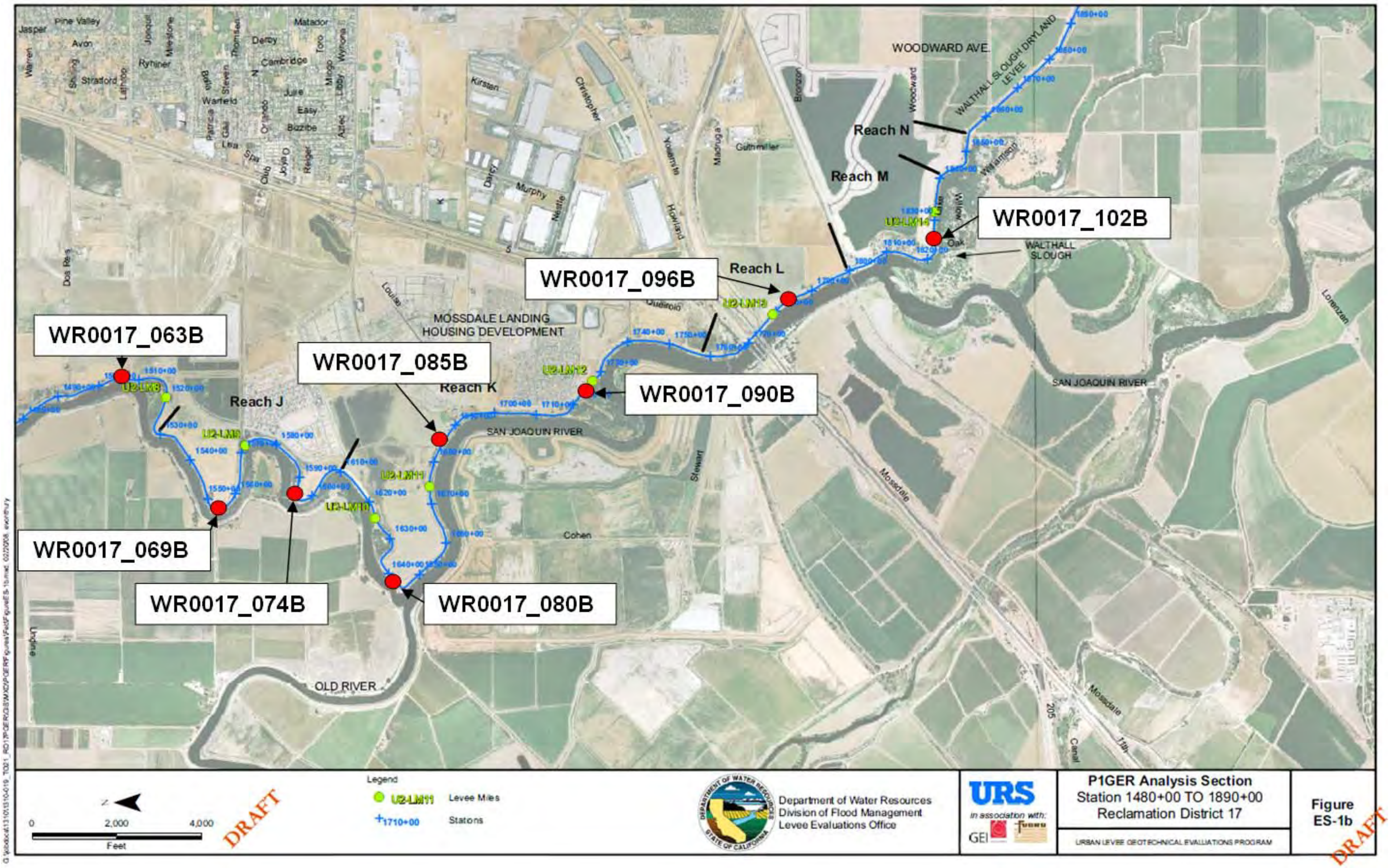


Plate 1. **RD 17 – Southern part** (Stations 1480 to 1840).

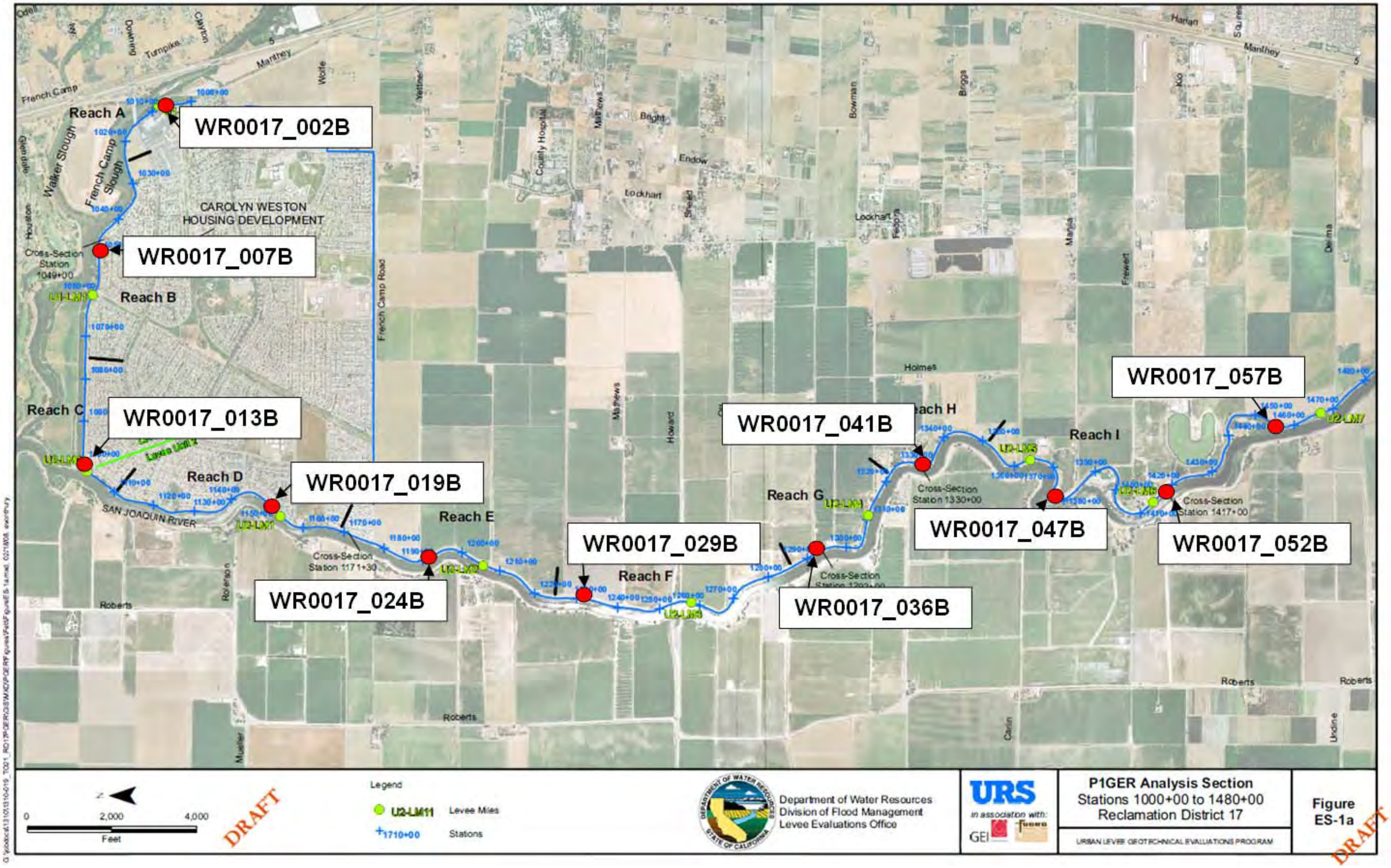


Plate 2. **RD 17 – Northern part** (Stations 1000 to 1480).

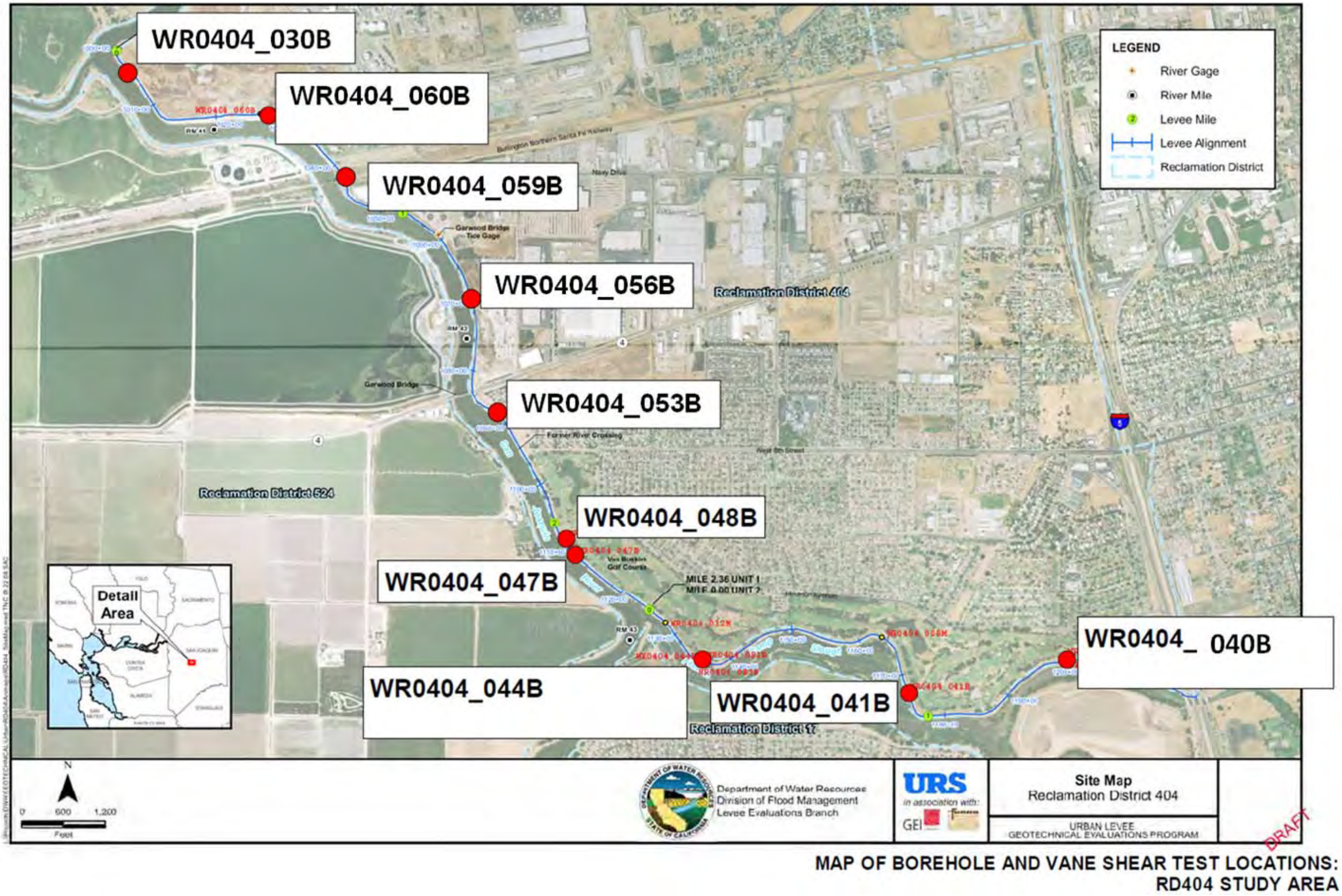


Plate 3. **RD 404.**

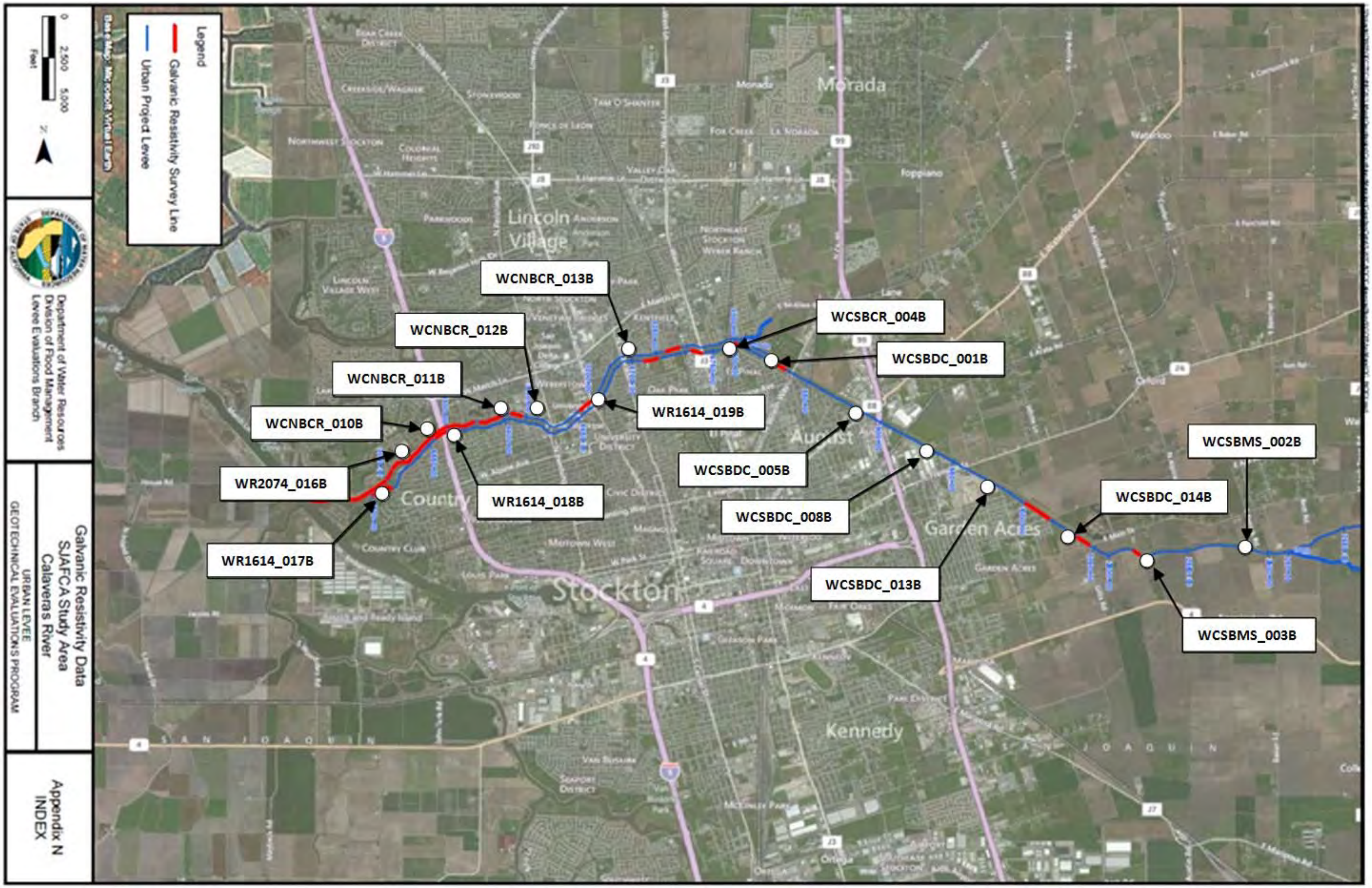


Plate 4. **Calaveras River, Stockton Diverting Canal** and **Mormon Slough**

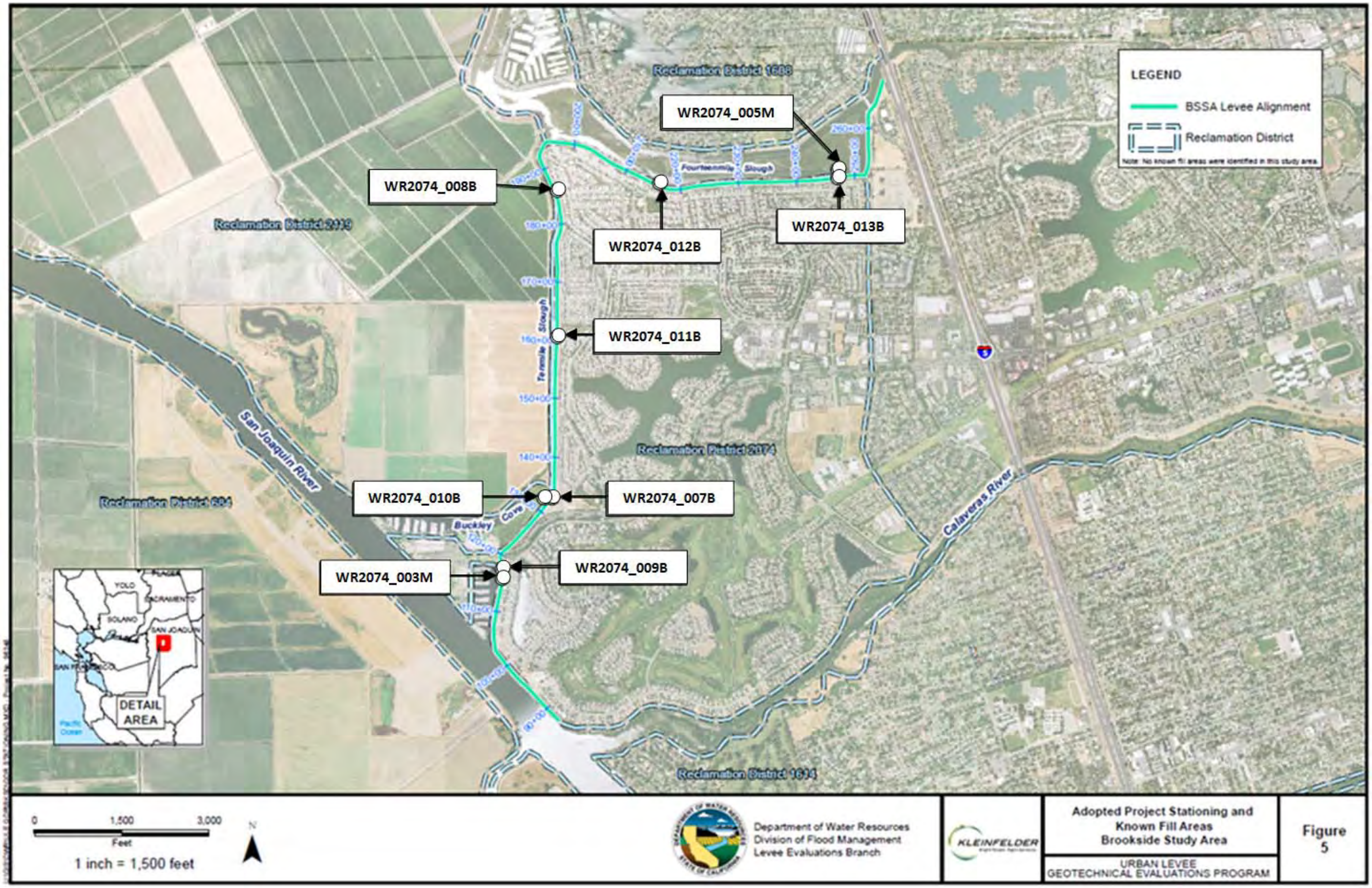


Plate 5. **Brookside**

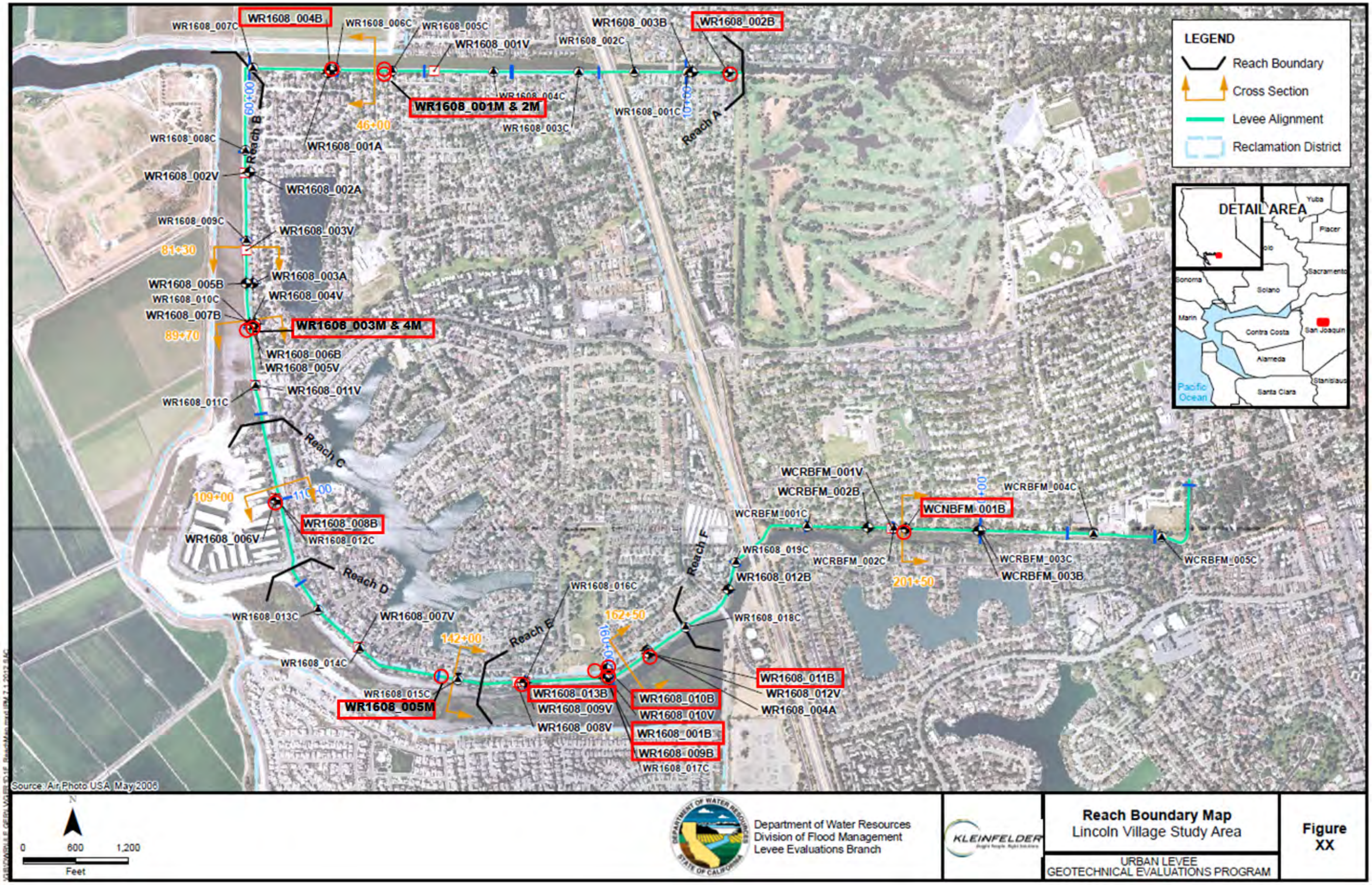
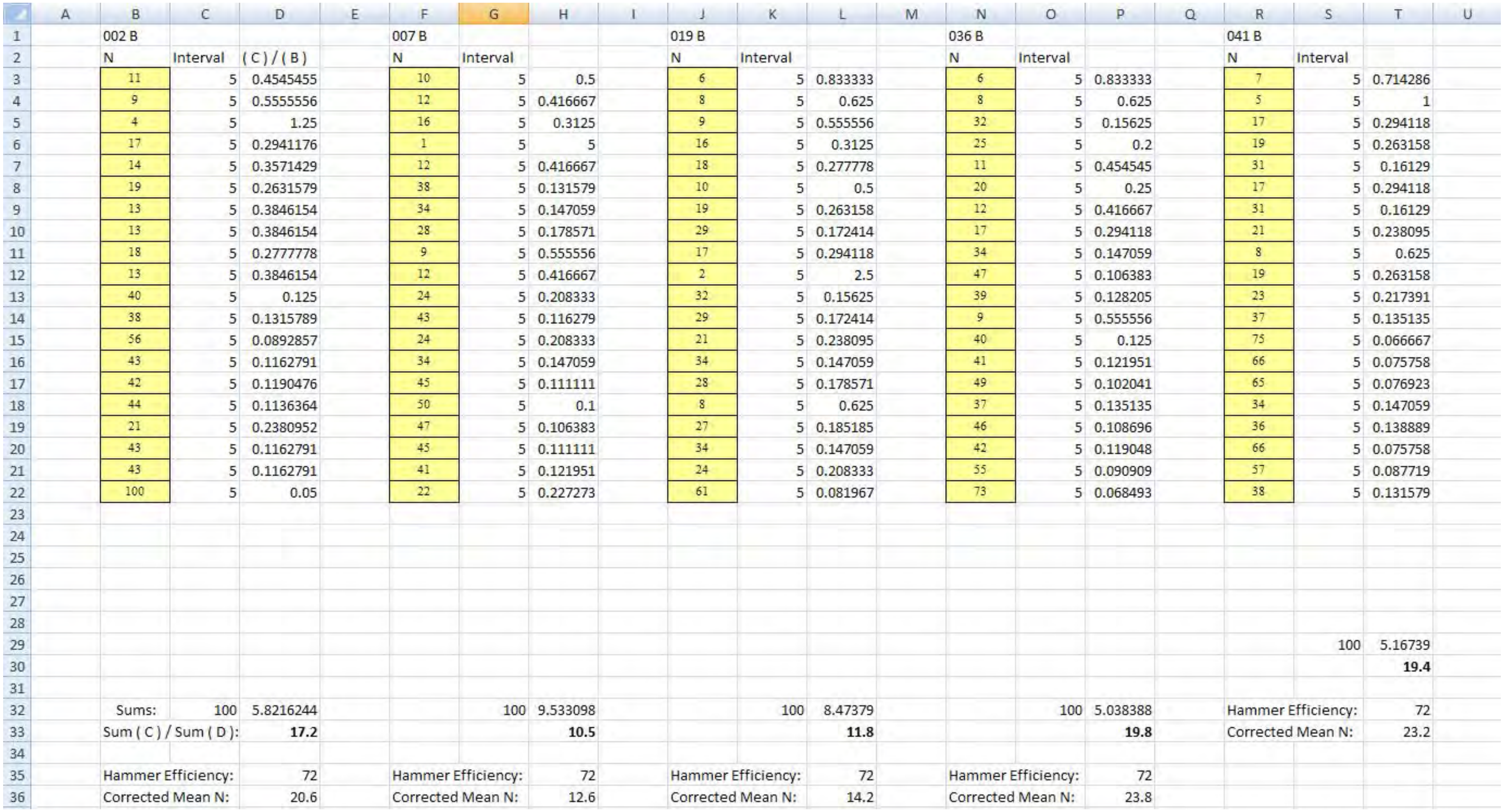


Plate 6. **Lincoln Village**

APPENDIX B

Evaluation of Weighted Harmonic Mean N (SPT)

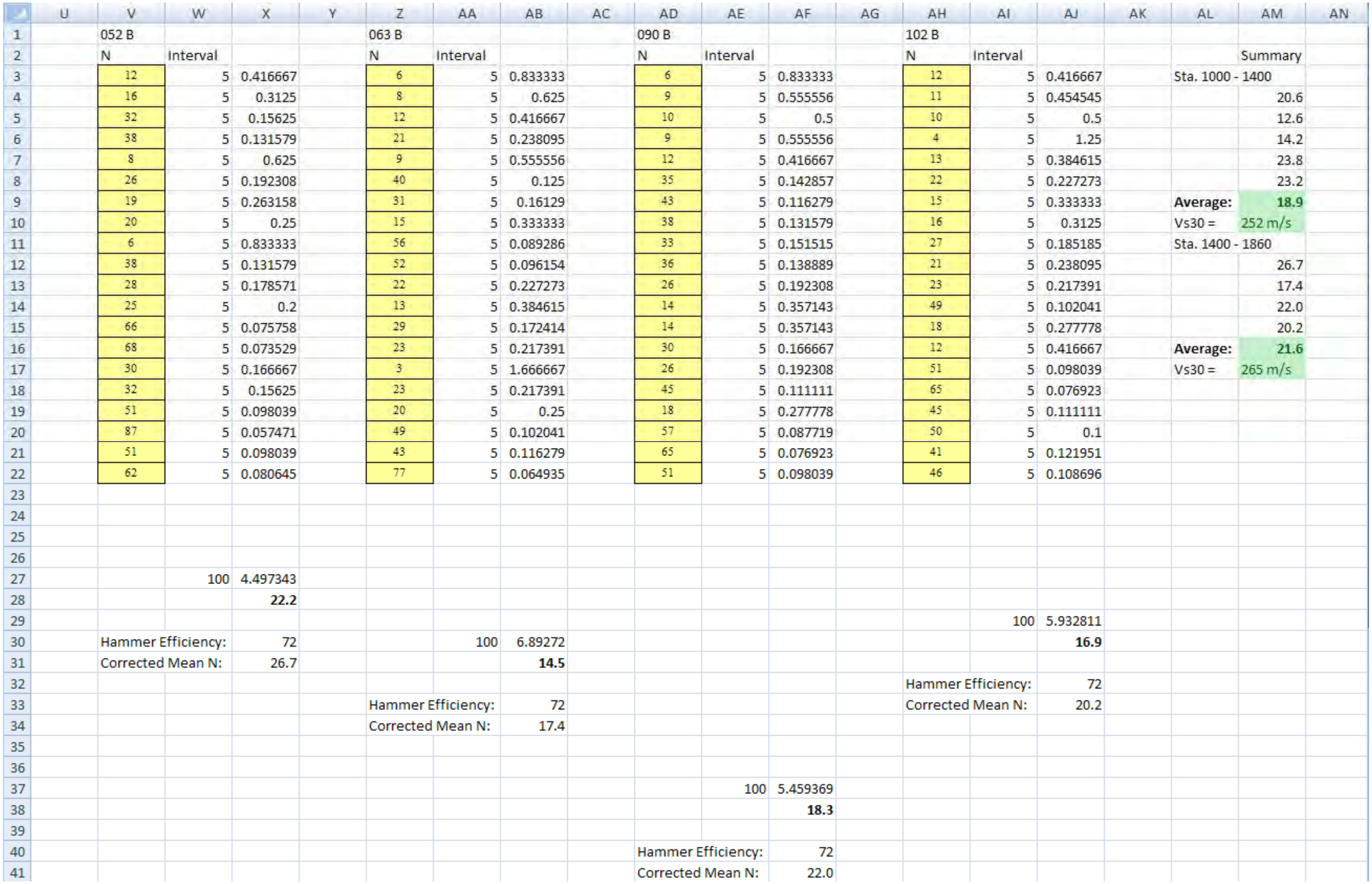
RD 17 – Northern part: Harmonic mean of N corrected for hammer efficiency (N60) for borings 100 feet deep.



	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U
1		002 B				007 B				019 B				036 B				041 B			
2		N	Interval	(C) / (B)		N	Interval			N	Interval			N	Interval			N	Interval		
3		11	5	0.4545455		10	5	0.5		6	5	0.833333		6	5	0.833333		7	5	0.714286	
4		9	5	0.5555556		12	5	0.416667		8	5	0.625		8	5	0.625		5	5	1	
5		4	5	1.25		16	5	0.3125		9	5	0.555556		32	5	0.15625		17	5	0.294118	
6		17	5	0.2941176		1	5	5		16	5	0.3125		25	5	0.2		19	5	0.263158	
7		14	5	0.3571429		12	5	0.416667		18	5	0.277778		11	5	0.454545		31	5	0.16129	
8		19	5	0.2631579		38	5	0.131579		10	5	0.5		20	5	0.25		17	5	0.294118	
9		13	5	0.3846154		34	5	0.147059		19	5	0.263158		12	5	0.416667		31	5	0.16129	
10		13	5	0.3846154		28	5	0.178571		29	5	0.172414		17	5	0.294118		21	5	0.238095	
11		18	5	0.2777778		9	5	0.555556		17	5	0.294118		34	5	0.147059		8	5	0.625	
12		13	5	0.3846154		12	5	0.416667		2	5	2.5		47	5	0.106383		19	5	0.263158	
13		40	5	0.125		24	5	0.208333		32	5	0.15625		39	5	0.128205		23	5	0.217391	
14		38	5	0.1315789		43	5	0.116279		29	5	0.172414		9	5	0.555556		37	5	0.135135	
15		56	5	0.0892857		24	5	0.208333		21	5	0.238095		40	5	0.125		75	5	0.066667	
16		43	5	0.1162791		34	5	0.147059		34	5	0.147059		41	5	0.121951		66	5	0.075758	
17		42	5	0.1190476		45	5	0.111111		28	5	0.178571		49	5	0.102041		65	5	0.076923	
18		44	5	0.1136364		50	5	0.1		8	5	0.625		37	5	0.135135		34	5	0.147059	
19		21	5	0.2380952		47	5	0.106383		27	5	0.185185		46	5	0.108696		36	5	0.138889	
20		43	5	0.1162791		45	5	0.111111		34	5	0.147059		42	5	0.119048		66	5	0.075758	
21		43	5	0.1162791		41	5	0.121951		24	5	0.208333		55	5	0.090909		57	5	0.087719	
22		100	5	0.05		22	5	0.227273		61	5	0.081967		73	5	0.068493		38	5	0.131579	
23																					
24																					
25																					
26																					
27																					
28																					
29																			100	5.16739	
30																				**19.4**	
31																					
32		Sums:	100	5.8216244			100	9.533098			100	8.47379			100	5.038388		Hammer Efficiency:		72	
33		Sum (C) / Sum (D):		**17.2**				**10.5**				**11.8**				**19.8**		Corrected Mean N:		23.2	
34																					
35		Hammer Efficiency:		72		Hammer Efficiency:		72		Hammer Efficiency:		72		Hammer Efficiency:		72					
36		Corrected Mean N:		20.6		Corrected Mean N:		12.6		Corrected Mean N:		14.2		Corrected Mean N:		23.8					

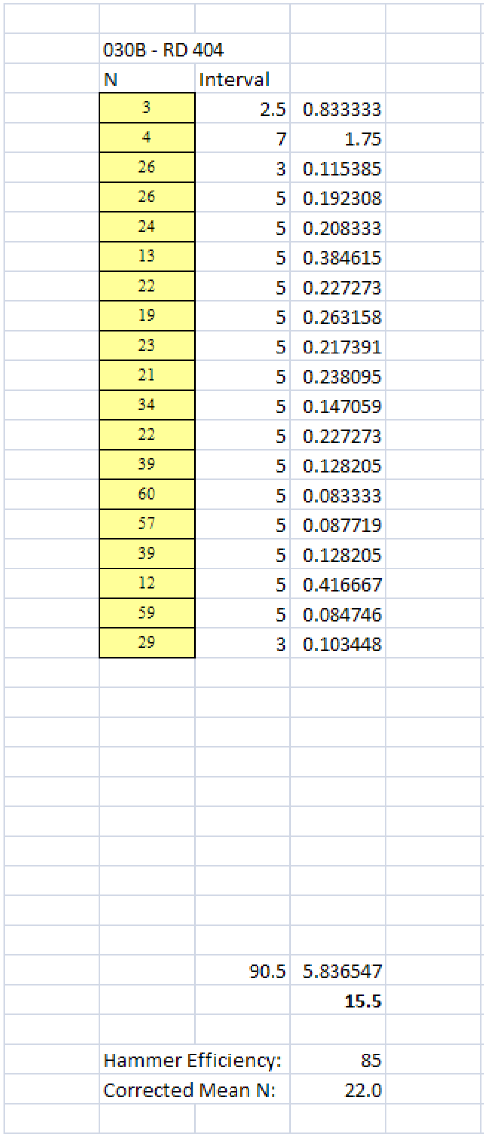
RD 17 – Southern part: Harmonic mean of N corrected for hammer efficiency (N60) for borings 100 feet deep.

Summary for both northern
and southern parts of RD 17



	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI	AJ	AK	AL	AM	AN
1		052 B				063 B				090 B				102 B						
2		N	Interval			N	Interval			N	Interval			N	Interval				Summary	
3		12	5	0.416667		6	5	0.833333		6	5	0.833333		12	5	0.416667		Sta. 1000 - 1400		
4		16	5	0.3125		8	5	0.625		9	5	0.555556		11	5	0.454545			20.6	
5		32	5	0.15625		12	5	0.416667		10	5	0.5		10	5	0.5			12.6	
6		38	5	0.131579		21	5	0.238095		9	5	0.555556		4	5	1.25			14.2	
7		8	5	0.625		9	5	0.555556		12	5	0.416667		13	5	0.384615			23.8	
8		26	5	0.192308		40	5	0.125		35	5	0.142857		22	5	0.227273			23.2	
9		19	5	0.263158		31	5	0.16129		43	5	0.116279		15	5	0.333333		**Average:**	**18.9**	
10		20	5	0.25		15	5	0.333333		38	5	0.131579		16	5	0.3125		Vs30 =	252 m/s	
11		6	5	0.833333		56	5	0.089286		33	5	0.151515		27	5	0.185185		Sta. 1400 - 1860		
12		38	5	0.131579		52	5	0.096154		36	5	0.138889		21	5	0.238095			26.7	
13		28	5	0.178571		22	5	0.227273		26	5	0.192308		23	5	0.217391			17.4	
14		25	5	0.2		13	5	0.384615		14	5	0.357143		49	5	0.102041			22.0	
15		66	5	0.075758		29	5	0.172414		14	5	0.357143		18	5	0.277778			20.2	
16		68	5	0.073529		23	5	0.217391		30	5	0.166667		12	5	0.416667		**Average:**	**21.6**	
17		30	5	0.166667		3	5	1.666667		26	5	0.192308		51	5	0.098039		Vs30 =	265 m/s	
18		32	5	0.15625		23	5	0.217391		45	5	0.111111		65	5	0.076923				
19		51	5	0.098039		20	5	0.25		18	5	0.277778		45	5	0.111111				
20		87	5	0.057471		49	5	0.102041		57	5	0.087719		50	5	0.1				
21		51	5	0.098039		43	5	0.116279		65	5	0.076923		41	5	0.121951				
22		62	5	0.080645		77	5	0.064935		51	5	0.098039		46	5	0.108696				
23																				
24																				
25																				
26																				
27			100	4.497343																
28				**22.2**																
29															100	5.932811				
30		Hammer Efficiency:		72			100	6.89272								**16.9**				
31		Corrected Mean N:		26.7				**14.5**												
32														Hammer Efficiency:		72				
33						Hammer Efficiency:		72						Corrected Mean N:		20.2				
34						Corrected Mean N:		17.4												
35																				
36																				
37											100	5.459369								
38												**18.3**								
39																				
40										Hammer Efficiency:		72								
41										Corrected Mean N:		22.0								

RD 404: Harmonic mean of N corrected for hammer efficiency (N60) for the only one available boring deeper than 100 feet.



030B - RD 404		
N	Interval	
3	2.5	0.833333
4	7	1.75
26	3	0.115385
26	5	0.192308
24	5	0.208333
13	5	0.384615
22	5	0.227273
19	5	0.263158
23	5	0.217391
21	5	0.238095
34	5	0.147059
22	5	0.227273
39	5	0.128205
60	5	0.083333
57	5	0.087719
39	5	0.128205
12	5	0.416667
59	5	0.084746
29	3	0.103448
	90.5	5.836547
		15.5
Hammer Efficiency:		85
Corrected Mean N:		22.0

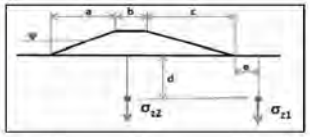
APPENDIX C

Liquefaction Triggering Evaluation

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea
Study Area:	RD 17	**Checked by:**	
River Section:	Sta. 1506+19	**Date:**	5/6/2013
Boring Number:	WR0017_063B	**Date:**	

Input Parameters

Embankment Crest Elevation (ft)	28.6 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	17.1 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.225
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	0.6 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	9.0 ft				120.0 pcf



Surcharge Information

Waterside/Upstream Slope, a (ft)	29.9 ft
Crest Width, b (ft)	21.0 ft
Landside/Downstream Slope, c (ft)	36.8 ft
Dist. of Boring from Levee Toe[1] (ft)	-47.3 ft
Embankment Height, H (ft)	11.5 ft

Boring	**WR0017_063B**
Boring on the crest	
SPT Ground Elevation Used in Analysis	
28.60 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_σ	f parameter	K_α	FS against Liquefaction
1.0	27.6	10	GC	12	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	15.3	1.55	1.03	17.3	n.a.	1.00	#N/A	1.00	0.72	#N/A	#N/A
6.0	22.6	25	SC	36	Unsaturated	120	125	720.0	720.0	0.0	Embankment	Embankment	1.70	1	0.8	1.00	40.6	5.00	1.20	54.0	n.a.	0.99	#N/A	1.00	0.60	#N/A	#N/A
11.0	17.6	16	CL	94	Unsaturated	120	125	1320.0	1320.0	0.0	Embankment	Embankment	1.27	1	0.85	1.00	20.7	5.00	1.20	29.8	n.a.	0.97	#N/A	1.00	0.67	#N/A	#N/A
13.5	15.1	6	SC	42	Unsaturated	120	125	1619.4	1619.4	1379.4	1620.0	1620.0	1.14	1	0.95	1.00	7.2	5.00	1.20	13.6	n.a.	0.97	0.14	1.00	0.80	1.00	#N/A
16.0	12.6	6	CL	94	Unsaturated	120	125	1913.8	1913.8	1373.8	1920.0	1920.0	1.05	1	0.95	1.00	7.2	5.00	1.20	13.6	n.a.	0.96	0.14	1.00	0.80	1.00	#N/A
21.0	7.6	8	CL	94	Clay	120	125	2476.6	2476.6	1336.6	1142.0	1117.0	0.92	1	0.95	1.00	n.a.	5.00	1.20	n.a.	2.00	0.95	0.14	1.00	0.60	1.00	#N/A
26.0	2.6	12	CL	94	Clay	120	125	3011.5	3011.5	1271.5	1767.0	1430.0	0.84	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.94	0.17	1.00	0.60	1.00	#N/A
31.0	-2.4	21	CL	94	Clay	120	125	3548.4	3361.2	1193.4	2392.0	1743.0	0.79	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.92	0.18	1.00	0.60	1.00	#N/A
36.0	-7.4	9	CL	94	Clay	120	125	4092.6	3593.4	1112.6	3017.0	2056.0	0.77	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.88	0.19	1.00	0.60	1.00	#N/A
42.5	-13.9	40	CL	94	Clay	120	125	4804.4	3899.6	1011.9	3629.5	2462.9	0.74	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.83	0.19	1.00	0.60	0.94	#N/A
46.0	-17.4	31	SC	22		120	125	5191.3	4068.1	961.3	4267.0	2682.0	0.72	1	1	1.00	26.6	3.93	1.09	33.3	2.00	0.80	0.19	1.00	0.62	0.91	3.00
51.0	-22.4	15	SC	20		120	125	5749.2	4314.0	894.2	4892.0	2995.0	0.70	1	1	1.00	12.6	3.61	1.08	17.2	0.18	0.76	0.18	1.00	0.74	0.91	1.39
56.0	-27.4	56	SC	20		120	125	6313.4	4566.2	833.4	5517.0	3308.0	0.66	1	1	1.00	45.7	3.61	1.08	53.0	2.00	0.72	0.18	1.00	0.60	0.84	3.00

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream

[2] Soil description may be used to estimate fines content where lab testing is not available

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used

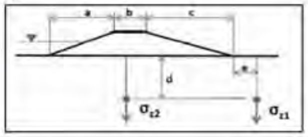
[4] It is conservative to answer "No" if unsure about sampling method, answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

Updated April 2013

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea
Study Area:	RD 17	**Checked by:**	
River Section:	Sta. 1555+82	**Date:**	5/6/2013
Boring Number:	WR0017_069B	**Date:**	

Input Parameters

Embankment Crest Elevation (ft)	29.4 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	11.9 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.235
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	3.4 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	8.7 ft				120.0 pcf



Surcharge Information

Waterside/Upstream Slope, a (ft)	40.2 ft
Crest Width, b (ft)	21.0 ft
Landside/Downstream Slope, c (ft)	36.8 ft
Dist. of Boring from Levee Toe[1] (ft)	-47.3 ft
Embankment Height, H (ft)	17.5 ft

Boring	**WR0017_069B**
Boring on the crest	
SPT Ground Elevation Used in Analysis	
29.40 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_σ	f parameter	K_α	FS against Liquefaction
1.0	28.4	31	SM	15	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	47.4	2.50	1.05	52.2	n.a.	1.00	#N/A	1.00	0.60	#N/A	#N/A
7.5	21.9	13	SP-SM	5	Unsaturated	120	125	900.0	900.0	0.0	Embankment	Embankment	1.53	1	0.85	1.00	20.3	0.00	1.00	20.3	n.a.	0.98	#N/A	1.00	0.67	#N/A	#N/A
12.5	16.9	5	SP	3	Unsaturated	120	125	1500.0	1500.0	0.0	Embankment	Embankment	1.19	1	0.85	1.00	6.1	0.00	1.00	6.1	n.a.	0.97	#N/A	1.00	0.80	#N/A	#N/A
18.0	11.4	7	ML	50	Unsaturated	120	125	2160.0	2160.0	2100.0	2160.0	2160.0	0.99	1	0.95	1.00	7.9	5.00	1.20	14.5	n.a.	0.96	0.14	1.00	0.79	1.00	#N/A
23.0	6.4	12	SM	19		120	125	2745.7	2745.7	2095.7	671.5	520.0	0.88	1	0.95	1.00	12.0	3.43	1.07	16.3	0.17	0.95	0.16	1.00	0.75	1.00	1.48
26.0	3.4	16	SP-SM	6		120	125	3075.8	3075.8	2055.8	1046.5	715.8	0.83	1	1	1.00	15.9	0.03	1.00	16.0	0.17	0.94	0.20	1.00	0.71	1.00	1.21
35.0	-5.6	13	SP-SM	6		120	125	4036.7	3475.1	1891.7	2171.5	1279.2	0.78	1	1	1.00	12.2	0.03	1.00	12.3	0.15	0.89	0.22	1.00	0.75	1.00	[illegible]
36.0	-6.6	9	ML	50		120	125	4139.9	3515.9	1869.9	2296.5	1341.8	0.78	1	1	1.00	8.4	5.00	1.20	15.1	0.16	0.88	0.22	1.00	0.79	1.00	1.09
44.5	-15.1	14	ML	50		120	125	5012.6	3858.2	1680.1	3359.0	1873.9	0.74	1	1	1.00	12.4	5.00	1.20	19.9	0.21	0.81	0.21	1.00	0.75	1.00	1.51
47.5	-18.1	17	SM	15		120	125	5322.3	3960.7	1614.8	3734.0	2061.7	0.73	1	1	1.00	14.9	2.50	1.05	18.1	0.19	0.79	0.21	1.00	0.72	1.00	1.39
55.0	-25.6	30	SM	26		120	125	6106.3	4296.7	1461.3	4671.5	2531.2	0.70	1	1	1.00	25.3	4.39	1.12	32.7	2.00	0.73	0.20	1.00	0.84	0.94	3.00
57.5	-28.1	53	SP	4		120	125	6371.3	4405.7	1413.8	4984.0	2687.7	0.69	1	1	1.00	48.2	0.00	1.00	48.2	2.00	0.71	0.19	1.00	0.60	0.91	3.00
65.0	-35.6	58	SM	27		120	125	7178.1	4744.5	1283.1	5921.5	3157.2	0.67	1	1	1.00	46.5	4.46	1.18	57.0	2.00	0.64	0.18	1.00	0.60	0.85	3.00
73.0	-43.6	16	CL		Clay	120	125	8057.1	5124.3	1162.1	6921.5	3658.0	0.64	1	1	1.00	n.a.	0.00	1.00	n.a.	2.00	0.58	0.16	1.00	0.60	0.80	#N/A

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream

[2] Soil description may be used to estimate fines content where lab testing is not available

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used

[4] It is conservative to answer "No" if unsure about sampling method, answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

Updated April 2013

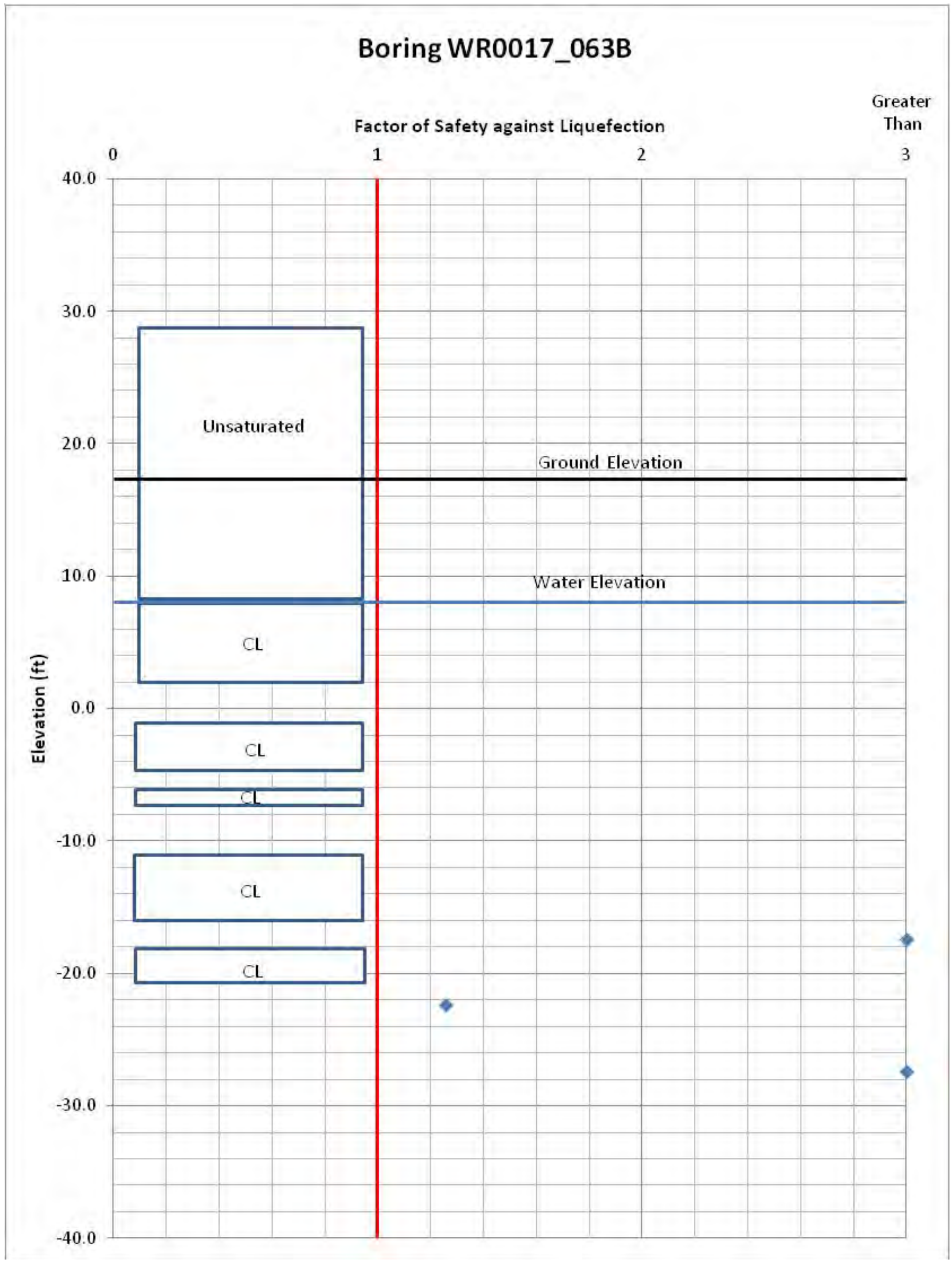


Fig. C-1. RD 17 South, Station 1506+19

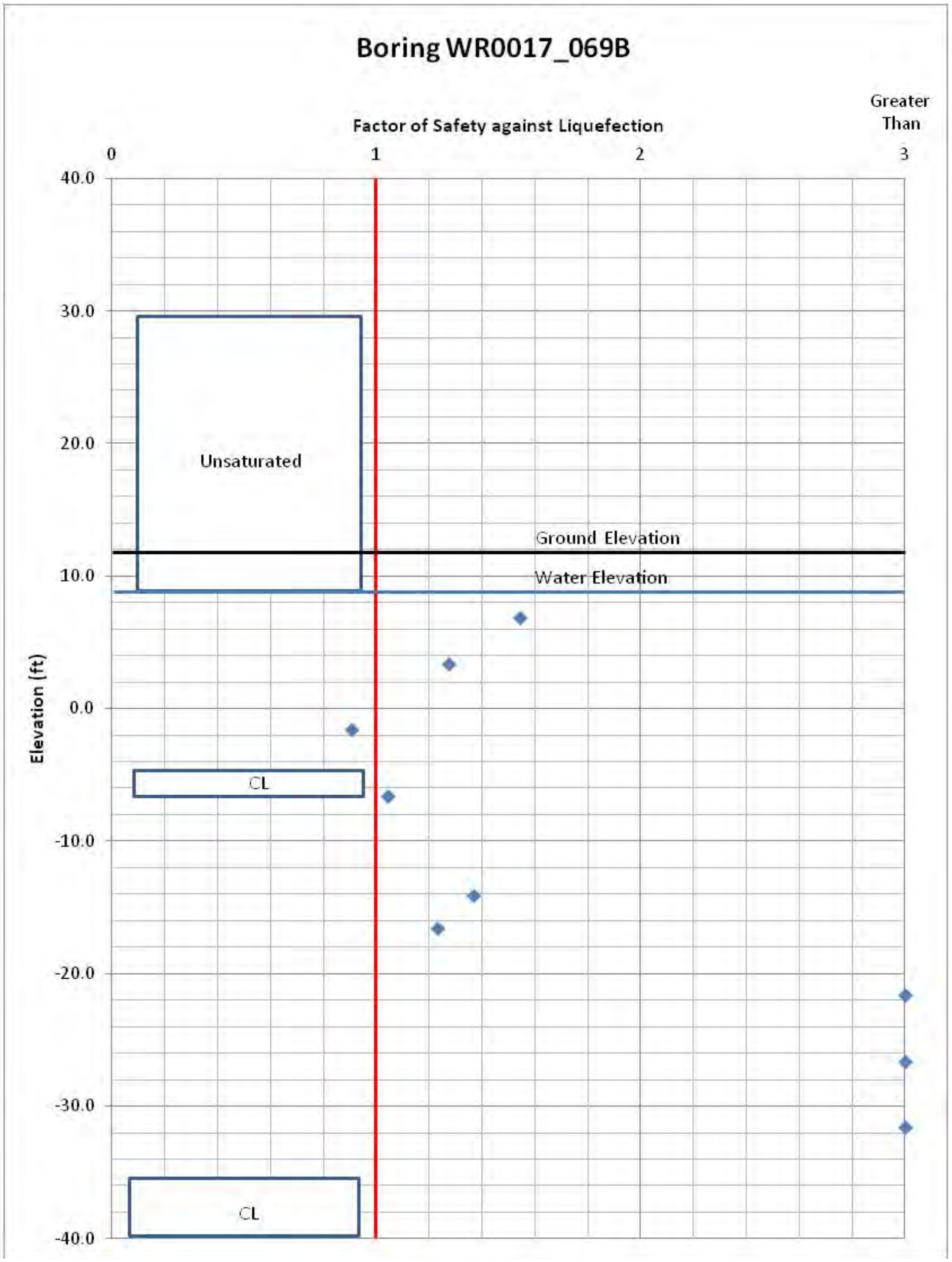


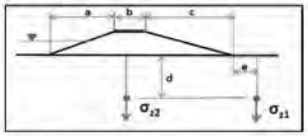
Fig. C-2. RD 17 South, Station 1553+82

Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1595+33
Boring Number: WR0017_074B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	29.9 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	19.9 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.225
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	4.4 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	7.7 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	35.0 ft
Crest Width, b (ft)	18.0 ft
Landside/Downstream Slope, c (ft)	43.0 ft
Dist. of Boring from Levee Toe[1] (ft)	-52.0 ft
Embankment Height, H (ft)	10.0 ft

Boring	**WR0017_074B**
Boring on the crest	
SPT Ground Elevation Used in Analysis	
29.90 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
4.0	25.9	30	CL	94	Unsaturated	120	125	480.0	480.0	0.0	Embankment	Embankment	1.70	1	0.8	1.00	49.0	5.00	1.20	63.8	n.a.	0.99	#N/A	1.00	0.80	#N/A	#N/A
6.0	23.9	13	SP	4	Unsaturated	120	125	720.0	720.0	0.0	Embankment	Embankment	1.70	1	0.8	1.00	19.6	0.00	1.00	19.6	n.a.	0.99	#N/A	1.00	0.68	#N/A	#N/A
12.0	17.9	6	ML	65	Unsaturated	120	125	1439.4	1439.4	1199.4	1440.0	1440.0	1.21	1	0.85	1.00	7.4	5.00	1.20	13.9	n.a.	0.97	0.14	1.00	0.80	1.00	#N/A
20.0	9.9	6	CL	94	Unsaturated	120	125	2354.2	2354.2	1154.2	2400.0	2400.0	0.95	1	0.95	1.00	6.5	5.00	1.20	12.8	n.a.	0.95	0.14	1.00	0.80	0.98	#N/A
23.0	6.9	7	SP-SM	12		120	125	2680.9	2680.9	1120.9	1564.0	1514.1	0.89	1	0.95	1.00	7.1	1.55	1.03	8.9	0.10	0.95	0.14	1.00	0.80	1.00	1.06
26.0	3.9	5	SP-SM	11		120	125	3005.7	2974.5	1088.2	1939.0	1701.9	0.84	1	1	1.00	5.1	1.21	1.03	6.4	0.08	0.94	0.16	1.00	0.80	1.00	[illegible]
32.0	-2.1	10	SP-SM	11		120	125	3675.7	3270.1	1003.2	2689.0	2077.5	0.80	1	1	1.00	9.7	1.21	1.03	11.1	0.12	0.91	0.17	1.00	0.77	1.00	1.07
36.0	-6.1	13	SC	14		120	125	4122.5	3467.3	950.0	3189.0	2327.9	0.78	1	1	1.00	12.2	2.20	1.04	14.9	0.16	0.88	0.18	1.00	0.75	0.96	1.22
42.5	-12.6	12	SC	13		120	125	4853.2	3792.4	968.2	4001.5	2734.8	0.75	1	1	1.00	10.8	1.99	1.04	13.0	0.14	0.83	0.18	1.00	0.76	0.94	1.12
47.5	-17.6	11	SW-SC	11		120	125	5420.4	4047.6	910.4	4626.5	3047.8	0.72	1	1	1.00	9.6	1.21	1.03	11.0	0.12	0.79	0.17	1.00	0.77	0.92	[illegible]
51.0	-21.1	41	SW	4		120	125	5820.3	4229.1	772.8	5064.0	3266.9	0.71	1	1	1.00	34.8	0.00	1.00	34.8	2.00	0.76	0.17	1.00	0.60	0.84	3.00
57.5	-27.6	30	SC	13		120	125	6569.1	4572.3	709.1	5876.5	3673.8	0.68	1	1	1.00	24.5	1.89	1.04	27.3	0.35	0.71	0.17	1.00	0.64	0.82	2.59
61.0	-31.1	39	SC	13		120	125	6975.5	4760.5	678.0	6314.0	3892.9	0.67	1	1	1.00	31.2	1.89	1.04	34.2	2.00	0.68	0.16	1.00	0.60	0.78	3.00

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream

[2] Soil description may be used to estimate fines content where lab testing is not available

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

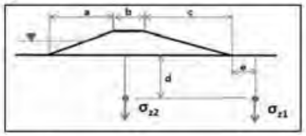
Updated April 2013

Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1642+75
Boring Number: WR0017_080B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	30.6 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	18.6 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.225
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	6.1 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	7.5 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	36.0 ft
Crest Width, b (ft)	17.0 ft
Landside/Downstream Slope, c (ft)	40.6 ft
Dist. of Boring from Levee Toe[1] (ft)	-49.3 ft
Embankment Height, H (ft)	12.0 ft

Boring	**WR0017_080B**
Boring on the crest	
SPT Ground Elevation Used in Analysis	
30.60 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
1.0	29.6	13	GC	12	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	19.9	1.55	1.03	22.1	n.a.	1.00	#N/A	1.00	0.68	#N/A	#N/A
7.5	23.1	8	SP	1	Unsaturated	120	125	900.0	900.0	0.0	Embankment	Embankment	1.53	1	0.85	1.00	12.5	0.00	1.00	12.5	n.a.	0.98	#N/A	1.00	0.74	#N/A	#N/A
11.0	19.6	5	ML	50	Unsaturated	120	125	1320.0	1320.0	0.0	Embankment	Embankment	1.27	1	0.85	1.00	6.5	5.00	1.20	12.7	n.a.	0.97	#N/A	1.00	0.80	#N/A	#N/A
16.0	14.6	6	CL	94	Unsaturated	120	125	1914.0	1914.0	1434.0	1920.0	1920.0	1.05	1	0.95	1.00	7.2	5.00	1.20	13.6	n.a.	0.96	0.14	1.00	0.80	1.00	#N/A
21.0	9.6	8	SP-SM	8	Unsaturated	120	125	2472.7	2472.7	1392.7	2520.0	2520.0	0.93	1	0.95	1.00	8.4	0.30	1.01	8.8	n.a.	0.95	0.14	1.00	0.79	0.96	#N/A
26.0	4.6	9	SM	17		120	125	3010.4	2916.8	1322.9	1694.5	1513.5	0.85	1	1	1.00	9.2	3.01	1.06	12.8	0.14	0.94	0.15	1.00	0.78	1.00	1.35
31.0	-0.4	18	SM	26		120	125	3554.4	3148.8	1241.9	2319.5	1826.5	0.82	1	1	1.00	17.7	4.39	1.12	24.3	0.28	0.90	0.17	1.00	0.69	1.00	2.44
36.0	-5.4	13	CL	94	Clay	120	125	4097.0	3379.4	1159.5	2944.5	2139.5	0.79	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.88	0.18	1.00	0.60	1.00	#N/A
41.0	-10.4	39	SP-SM	11		120	125	4642.7	3613.1	1080.2	3569.5	2452.5	0.77	1	1	1.00	35.8	1.21	1.03	38.0	2.00	0.84	0.18	1.00	0.60	0.94	3.00
47.5	-16.9	53	SP-SM	11		120	125	5359.9	3924.7	984.9	4382.0	2859.4	0.73	1	1	1.00	46.7	1.21	1.03	49.1	2.00	0.79	0.18	1.00	0.60	0.89	3.00
51.0	-20.4	44	SW	4		120	125	5750.4	4090.8	937.9	4819.5	3078.5	0.72	1	1	1.00	37.9	0.00	1.00	37.9	2.00	0.76	0.17	1.00	0.60	0.86	3.00
56.0	-25.4	65	SP	4		120	125	6213.3	4347.7	875.8	5444.5	3391.5	0.70	1	1	1.00	54.4	0.00	1.00	54.4	2.00	0.72	0.17	1.00	0.60	0.83	3.00
60.0	-29.4	59	SP	4		120	125	6767.8	4552.6	830.3	5944.5	3641.9	0.68	1	1	1.00	48.3	0.00	1.00	48.3	2.00	0.69	0.16	1.00	0.60	0.80	3.00

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream

[2] Soil description may be used to estimate fines content where lab testing is not available

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

Updated April 2013

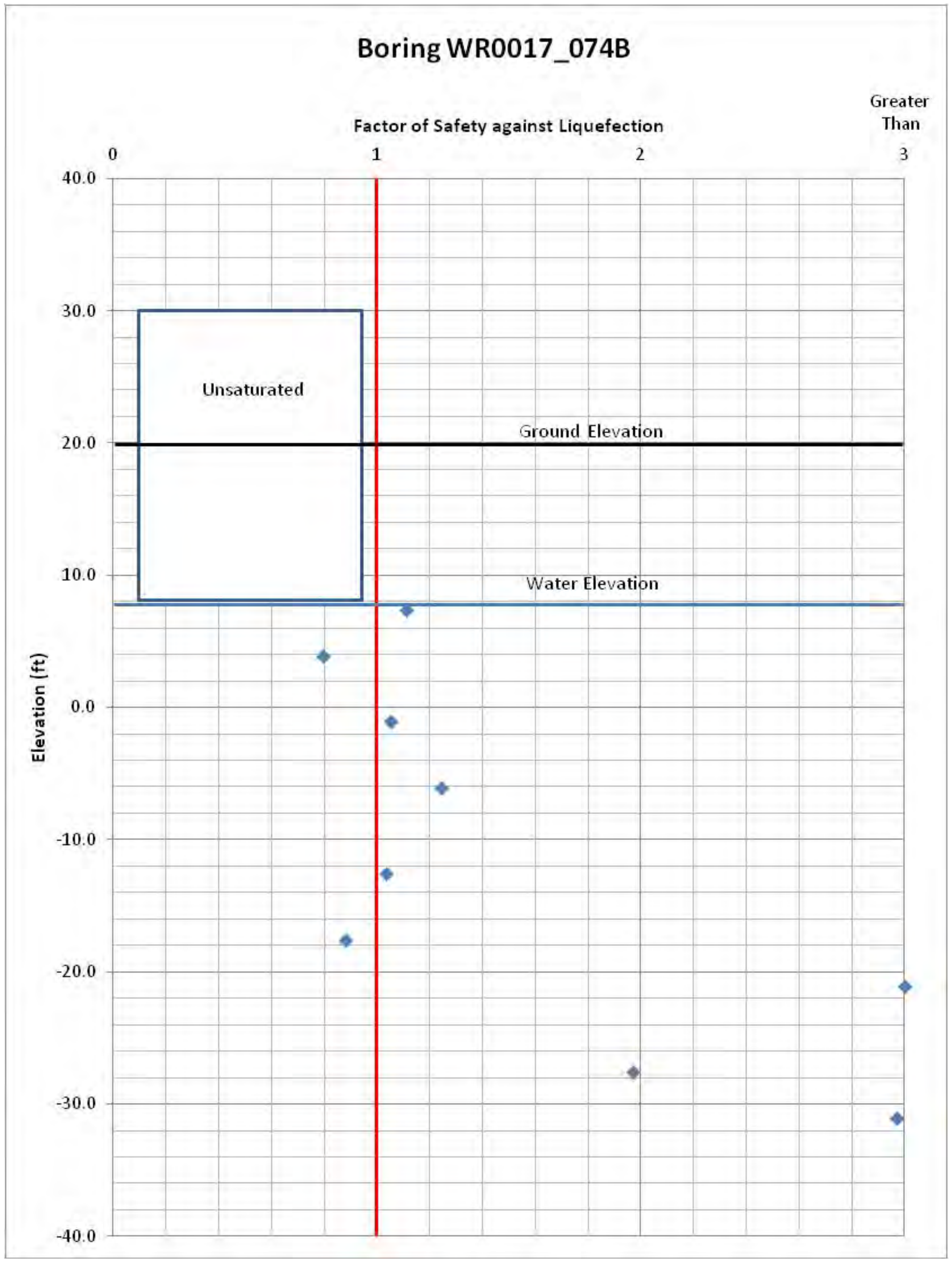


Fig. C-3. RD 17 South, Station 1595+33

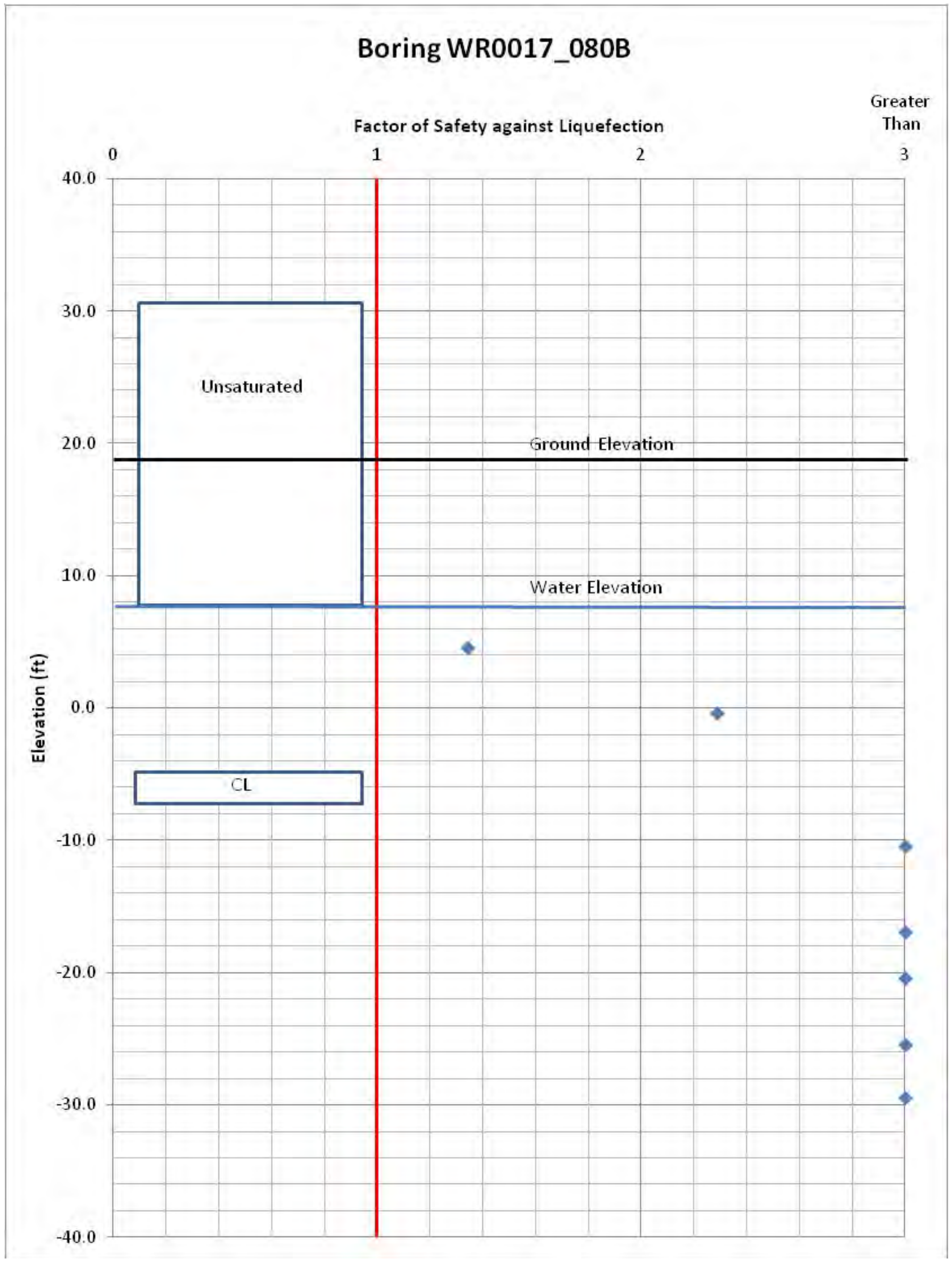


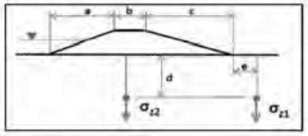
Fig. C-4. RD 17 South, Station 1642+75

Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1684+57
Boring Number: WR0017_085B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	31.4 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	18.9 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.225
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	4.9 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	7.4 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	42.5 ft
Crest Width, b (ft)	19.0 ft
Landside/Downstream Slope, c (ft)	30.0 ft
Dist. of Boring from Levee Toe[1] (ft)	-80.0 ft
Embankment Height, H (ft)	12.5 ft

Boring	WR0017_085B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
31.40 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
1.0	30.4	24	GC	12	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	36.7	1.55	1.03	39.4	n.a.	1.00	#N/A	1.00	0.60	#N/A	#N/A
7.5	23.9	26	SM	31	Unsaturated	120	125	900.0	900.0	0.0	Embankment	Embankment	1.53	1	0.85	1.00	40.7	4.77	1.16	52.0	n.a.	0.98	#N/A	1.00	0.60	#N/A	#N/A
11.0	20.4	24	SP	20	Unsaturated	120	125	1320.0	1320.0	0.0	Embankment	Embankment	1.27	1	0.85	1.00	31.0	3.61	1.08	37.1	n.a.	0.97	#N/A	1.00	0.60	#N/A	#N/A
17.5	13.9	9	CL	94	Unsaturated	120	125	2088.8	2088.8	1488.8	2100.0	2100.0	1.01	1	0.95	1.00	10.3	5.00	1.20	17.4	n.a.	0.96	0.14	1.00	0.77	1.00	#N/A
22.5	8.9	13	SP-SC	8	Unsaturated	120	125	2638.0	2638.0	1438.0	2700.0	2700.0	0.90	1	0.95	1.00	13.3	0.30	1.01	13.7	n.a.	0.95	0.14	1.00	0.74	0.94	#N/A
26.0	5.4	13	SM	21		120	125	3005.0	3005.0	1385.0	1630.0	1505.2	0.84	1	1	1.00	13.1	3.78	1.09	18.0	0.19	0.94	0.15	1.00	0.74	1.00	1.93
31.0	0.4	10	ML	63		120	125	3541.2	3260.4	1298.7	2255.0	1818.2	0.81	1	1	1.00	9.7	5.00	1.20	16.6	0.16	0.92	0.17	1.00	0.77	1.00	1.50
36.0	-4.6	28	SM	15		120	125	4077.7	3484.9	1210.2	2880.0	2131.2	0.78	1	1	1.00	26.2	2.50	1.05	29.9	0.46	0.88	0.17	1.00	0.63	1.00	3.00
42.5	-11.1	40	SW-SC	6		120	125	4780.4	3782.0	1100.4	3692.5	2538.1	0.75	1	1	1.00	35.9	0.03	1.00	36.1	2.00	0.83	0.18	1.00	0.60	0.93	3.00
46.0	-14.6	35	SM	15		120	125	5163.0	3946.2	1045.5	4130.0	2757.2	0.73	1	1	1.00	30.8	2.50	1.05	34.7	2.00	0.80	0.18	1.00	0.60	0.90	3.00
51.0	-19.6	37	SM	15		120	125	5715.3	4166.5	972.8	4755.0	3070.2	0.71	1	1	1.00	31.6	2.50	1.05	35.6	2.00	0.76	0.17	1.00	0.60	0.86	3.00
56.0	-24.6	31	SP	4		120	125	6274.3	4433.5	906.8	5380.0	3383.2	0.69	1	1	1.00	25.7	0.00	1.00	25.7	0.31	0.72	0.17	1.00	0.63	0.84	2.32
62.5	-31.1	43	SW	4		120	125	7010.6	4764.2	830.8	6192.5	3790.1	0.67	1	1	1.00	34.4	0.00	1.00	34.4	2.00	0.67	0.16	1.00	0.60	0.79	3.00

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream

[2] Soil description may be used to estimate fines content where lab testing is not available

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001

Surcharge from embankment calculation is presented in Poulos & Davis (1976) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used

[4] It is conservative to answer "No" if unsure about sampling method, answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

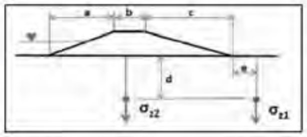
Updated April 2013

Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1724+68
Boring Number: WR0017_090B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	32.1 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	15.1 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.225
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	5.1 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	7.1 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	54.4 ft
Crest Width, b (ft)	20.0 ft
Landside/Downstream Slope, c (ft)	40.8 ft
Dist. of Boring from Levee Toe[1] (ft)	-50.8 ft
Embankment Height, H (ft)	17.0 ft

Boring	WR0017_090B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
32.10 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
1.0	31.1	14	ML	50	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	21.4	5.00	1.20	30.7	n.a.	1.00	#N/A	1.00	0.67	#N/A	#N/A
7.5	24.6	6	CL/SC	51	Unsaturated	120	125	900.0	900.0	0.0	Embankment	Embankment	1.53	1	0.85	1.00	9.4	5.00	1.20	16.3	n.a.	0.98	#N/A	1.00	0.78	#N/A	#N/A
11.0	21.1	6	CL	94	Unsaturated	120	125	1320.0	1320.0	0.0	Embankment	Embankment	1.27	1	0.85	1.00	7.7	5.00	1.20	14.3	n.a.	0.97	#N/A	1.00	0.79	#N/A	#N/A
16.0	16.1	9	ML	50	Unsaturated	120	125	1920.0	1920.0	0.0	Embankment	Embankment	1.05	1	0.95	1.00	10.8	5.00	1.20	17.9	n.a.	0.96	#N/A	1.00	0.76	#N/A	#N/A
23.5	8.6	10	CL	94	Unsaturated	120	125	2800.3	2800.3	2020.3	2620.0	2620.0	0.87	1	0.95	1.00	9.9	5.00	1.20	16.9	n.a.	0.95	0.14	1.00	0.77	0.94	#N/A
26.0	6.1	9	ML	74		120	125	3075.6	3075.6	1995.6	1085.0	1022.6	0.83	1	1	1.00	9.0	5.00	1.20	15.7	0.17	0.94	0.15	1.00	0.78	1.00	1.73
31.0	1.1	12	SM/ML	46		120	125	3622.8	3373.2	1922.6	1710.0	1335.6	0.79	1	1	1.00	11.4	5.00	1.20	18.7	0.20	0.92	0.17	1.00	0.76	1.00	1.74
36.0	-3.9	35	SP-SC	6		120	125	4157.7	3596.1	1832.7	2335.0	1648.6	0.77	1	1	1.00	32.2	0.03	1.00	32.4	2.00	0.88	0.18	1.00	0.60	1.00	3.00
41.0	-8.9	43	SM/ML	18		120	125	4686.5	3812.9	1736.5	2960.0	1961.6	0.74	1	1	1.00	38.4	3.23	1.07	44.2	2.00	0.84	0.19	1.00	0.60	1.00	3.00
46.0	-13.9	38	SP	4		120	125	5215.4	4029.8	1640.4	3585.0	2274.6	0.72	1	1	1.00	33.0	0.00	1.00	33.0	2.00	0.80	0.18	1.00	0.60	0.97	3.00
51.0	-18.9	33	SP	4		120	125	5747.5	4249.9	1547.5	4210.0	2587.6	0.71	1	1	1.00	27.9	0.00	1.00	27.9	0.37	0.76	0.18	1.00	0.62	0.93	2.83
56.0	-23.9	36	SP-SM	6		120	125	6264.6	4475.0	1459.6	4835.0	2900.6	0.69	1	1	1.00	29.7	0.03	1.00	29.9	0.46	0.72	0.18	1.00	0.60	0.88	3.00
61.0	-28.9	26	SP-SM	6		120	125	6827.4	4705.8	1377.4	5460.0	3213.6	0.67	1	1	1.00	20.9	0.03	1.00	21.0	0.23	0.68	0.17	1.00	0.67	0.87	1.78

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream

[2] Soil description may be used to estimate fines content where lab testing is not available

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001

Surcharge from embankment calculation is presented in Poulos & Davis (1976) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used

[4] It is conservative to answer "No" if unsure about sampling method, answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

Updated April 2013

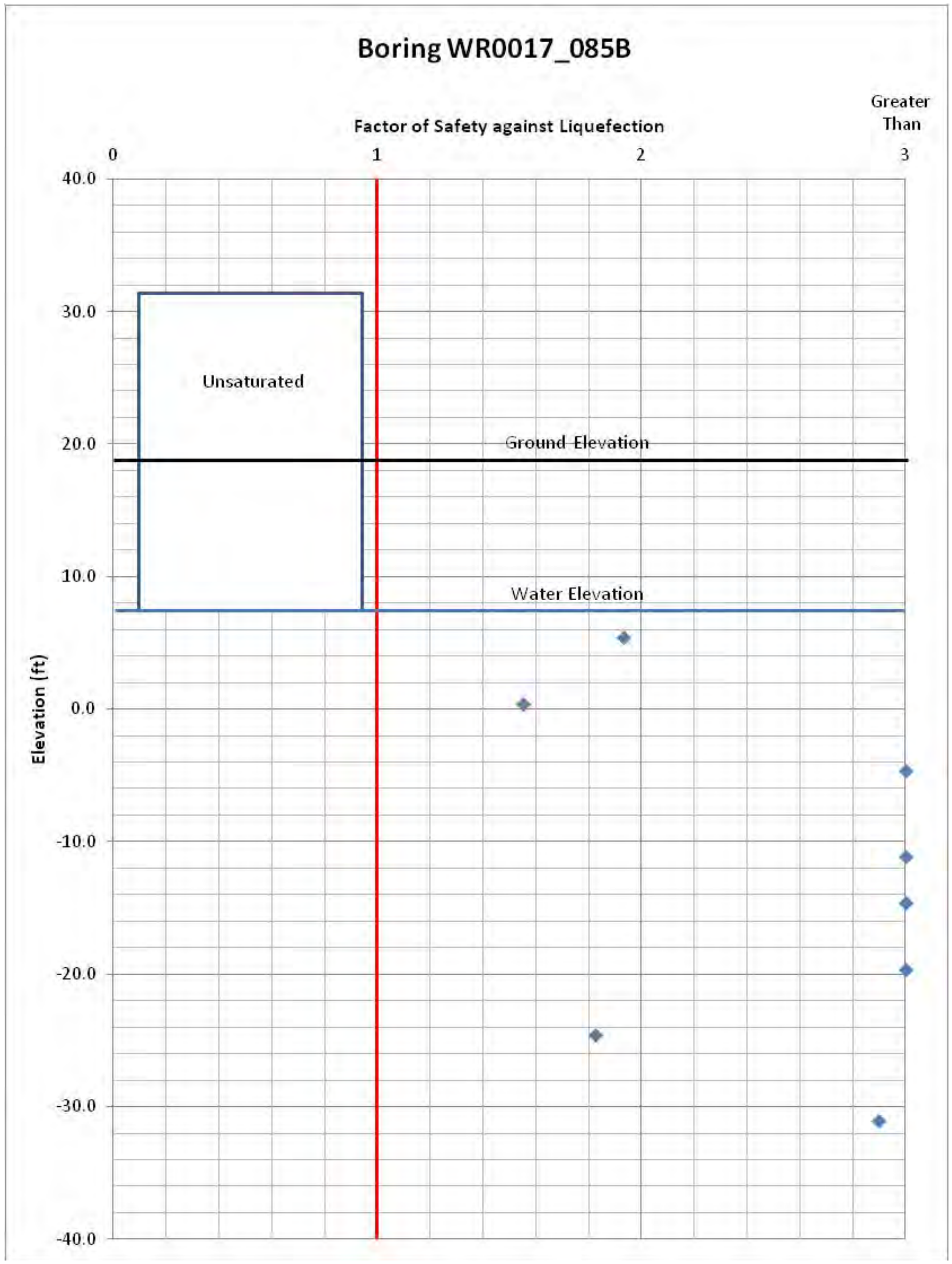


Fig. C-5. RD 17 South, Station 1684+57

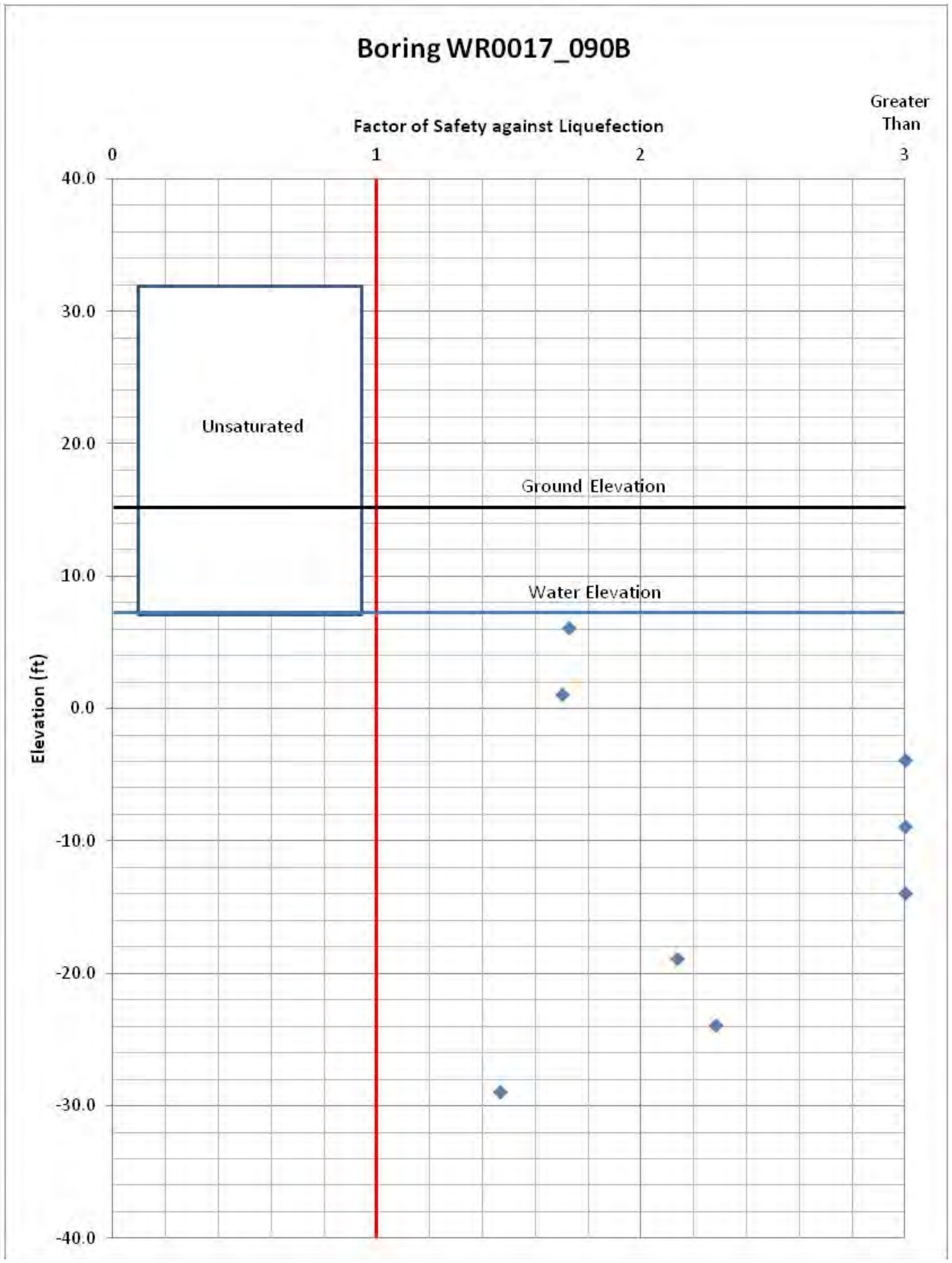


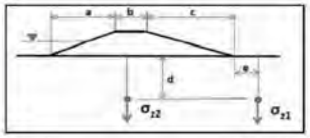
Fig. C-6. RD 17 South, Station 1724+68

Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1784+83
Boring Number: WR0017_096B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	32.8 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	19.3 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.225
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	3.8 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	6.8 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	47.3 ft
Crest Width, b (ft)	16.0 ft
Landside/Downstream Slope, c (ft)	41.9 ft
Dist. of Boring from Levee Toe[1] (ft)	50.9 ft
Embankment Height, H (ft)	13.5 ft

Boring	WR0017_096B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
32.80 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_σ	f parameter	K_α	FS against Liquefaction
1.0	31.8	8	SP-SM	6	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	12.2	0.30	1.01	12.7	n.a.	1.00	#N/A	1.00	0.75	#N/A	#N/A
6.0	26.8	19	SP-SM	6	Unsaturated	120	125	720.0	720.0	0.0	Embankment	Embankment	1.70	1	0.8	1.00	31.0	0.03	1.00	31.2	n.a.	0.99	#N/A	1.00	0.60	#N/A	#N/A
11.0	21.8	5	SP-SC	12	Unsaturated	120	125	1320.0	1320.0	0.0	Embankment	Embankment	1.27	1	0.85	1.00	6.5	1.55	1.03	8.2	n.a.	0.97	#N/A	1.00	0.80	#N/A	#N/A
16.0	16.8	6	SC	20	Unsaturated	120	125	1918.6	1918.6	1618.6	1920.0	1920.0	1.05	1	0.95	1.00	6.6	3.61	1.08	10.7	n.a.	0.96	0.14	1.00	0.80	1.00	#N/A
23.5	9.3	3	CL	34	Unsaturated	120	125	2765.6	2765.6	1565.6	2820.0	2820.0	0.87	1	0.95	1.00	3.0	5.00	1.20	8.6	n.a.	0.95	0.14	1.00	0.60	0.94	#N/A
28.5	4.3	9	SC	16		120	125	3295.8	3285.8	1495.8	1812.5	1656.5	0.80	1	1	1.00	8.7	2.77	1.05	11.9	0.13	0.93	0.15	1.00	0.78	1.00	1.31
31.0	1.8	9	SC	16		120	125	3566.3	3441.5	1456.3	2125.0	1813.0	0.78	1	1	1.00	8.5	2.77	1.05	11.7	0.13	0.92	0.16	1.00	0.79	1.00	1.22
36.0	-3.2	17	SC	16		120	125	4109.3	3672.5	1374.8	2750.0	2126.0	0.76	1	1	1.00	15.5	2.77	1.05	19.1	0.20	0.88	0.17	1.00	0.72	1.00	1.84
41.0	-8.2	17	SC	16		120	125	4652.6	3903.8	1292.6	3375.0	2439.0	0.74	1	1	1.00	15.0	2.77	1.05	18.6	0.20	0.84	0.17	1.00	0.72	0.96	1.63
46.0	-13.2	23	SC	16		120	125	5199.0	4139.2	1214.0	4000.0	2752.0	0.72	1	1	1.00	19.7	2.77	1.05	23.6	0.27	0.80	0.17	1.00	0.66	0.92	2.16
51.0	-18.2	48	SP-SM	11		120	125	5750.1	4377.3	1140.1	4625.0	3065.0	0.70	1	1	1.00	40.0	1.21	1.03	42.3	2.00	0.76	0.17	1.00	0.60	0.86	3.00
56.0	-23.2	56	SW-SM	12		120	125	6306.4	4621.6	1071.4	5250.0	3378.0	0.68	1	1	1.00	45.5	1.55	1.03	48.5	2.00	0.72	0.16	1.00	0.60	0.83	3.00
61.0	-28.2	63	SP	4		120	125	6888.1	4871.3	1008.1	5875.0	3691.0	0.66	1	1	1.00	49.8	0.00	1.00	49.8	2.00	0.68	0.16	1.00	0.60	0.80	3.00

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001

Surcharge from embankment calculation is presented in Poulos & Davis (1976) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method, answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

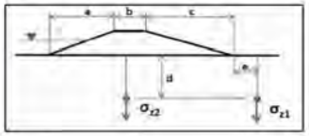
Updated April 2013

Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1625+94
Boring Number: WR0017_102B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	34.5 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	14.0 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.225
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	5.9 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	6.8 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	34.9 ft
Crest Width, b (ft)	13.0 ft
Landside/Downstream Slope, c (ft)	43.1 ft
Dist. of Boring from Levee Toe[1] (ft)	-49.6 ft
Embankment Height, H (ft)	20.5 ft

Boring	WR0017_102B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
34.50 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_σ	f parameter	K_α	FS against Liquefaction
1.0	33.5	10	SC	42	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	15.3	5.00	1.20	23.4	n.a.	1.00	#N/A	1.00	0.72	#N/A	#N/A
6.0	28.5	4	ML	50	Unsaturated	120	125	720.0	720.0	0.0	Embankment	Embankment	1.70	1	0.8	1.00	6.5	5.00	1.20	12.8	n.a.	0.99	#N/A	1.00	0.80	#N/A	#N/A
11.0	23.5	5	CL	94	Unsaturated	120	125	1320.0	1320.0	0.0	Embankment	Embankment	1.27	1	0.85	1.00	6.5	5.00	1.20	12.7	n.a.	0.97	#N/A	1.00	0.80	#N/A	#N/A
18.5	16.0	7	CL	94	Unsaturated	120	125	2220.0	2220.0	0.0	Embankment	Embankment	0.98	1	0.95	1.00	7.3	5.00	1.20	13.8	n.a.	0.96	#N/A	1.00	0.60	#N/A	#N/A
23.5	11.0	11	CL	94	Unsaturated	120	125	2812.5	2812.5	2452.5	2820.0	2820.0	0.87	1	0.95	1.00	10.9	5.00	1.20	18.1	n.a.	0.95	0.14	1.00	0.76	0.93	#N/A
33.5	1.0	10	CL	94	Clay	120	125	3822.8	3517.1	2238.3	1589.0	1227.1	0.78	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.90	0.17	1.00	0.60	1.00	#N/A
36.0	-1.5	4	CL	94	Clay	120	125	4060.7	3599.0	2163.7	1901.5	1383.6	0.77	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.88	0.18	1.00	0.60	1.00	#N/A
41.0	-6.5	13	SC	39		120	125	4534.8	3761.0	2012.8	2526.5	1696.6	0.75	1	1	1.00	11.7	5.00	1.20	19.9	0.20	0.84	0.18	1.00	0.75	1.00	1.67
46.0	-11.5	22	SP	4		120	125	5014.4	3928.6	1867.4	3151.5	2009.6	0.73	1	1	1.00	19.4	0.00	1.00	19.4	0.21	0.80	0.18	1.00	0.68	1.00	1.70
51.0	-16.5	15	SP-SM	7		120	125	5503.9	4106.1	1731.9	3776.5	2322.6	0.72	1	1	1.00	12.9	0.12	1.01	13.2	0.14	0.76	0.18	1.00	0.74	0.98	1.15
56.0	-21.5	16	SP-SM	7		120	125	6004.8	4285.1	1607.8	4401.5	2635.6	0.70	1	1	1.00	13.5	0.12	1.01	13.7	0.15	0.72	0.18	1.00	0.74	0.94	1.19
61.0	-26.5	27	SP-SM	7		120	125	6517.4	4495.7	1495.4	5026.5	2948.6	0.69	1	1	1.00	22.2	0.12	1.01	22.5	0.25	0.68	0.17	1.00	0.66	0.89	1.98
66.0	-31.5	21	SP-SM	7		120	125	7041.1	4707.3	1394.1	5651.5	3261.6	0.67	1	1	1.00	16.9	0.12	1.01	17.2	0.18	0.64	0.16	1.00	0.70	0.88	1.49

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001

Surcharge from embankment calculation is presented in Poulos & Davis (1976) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method, answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

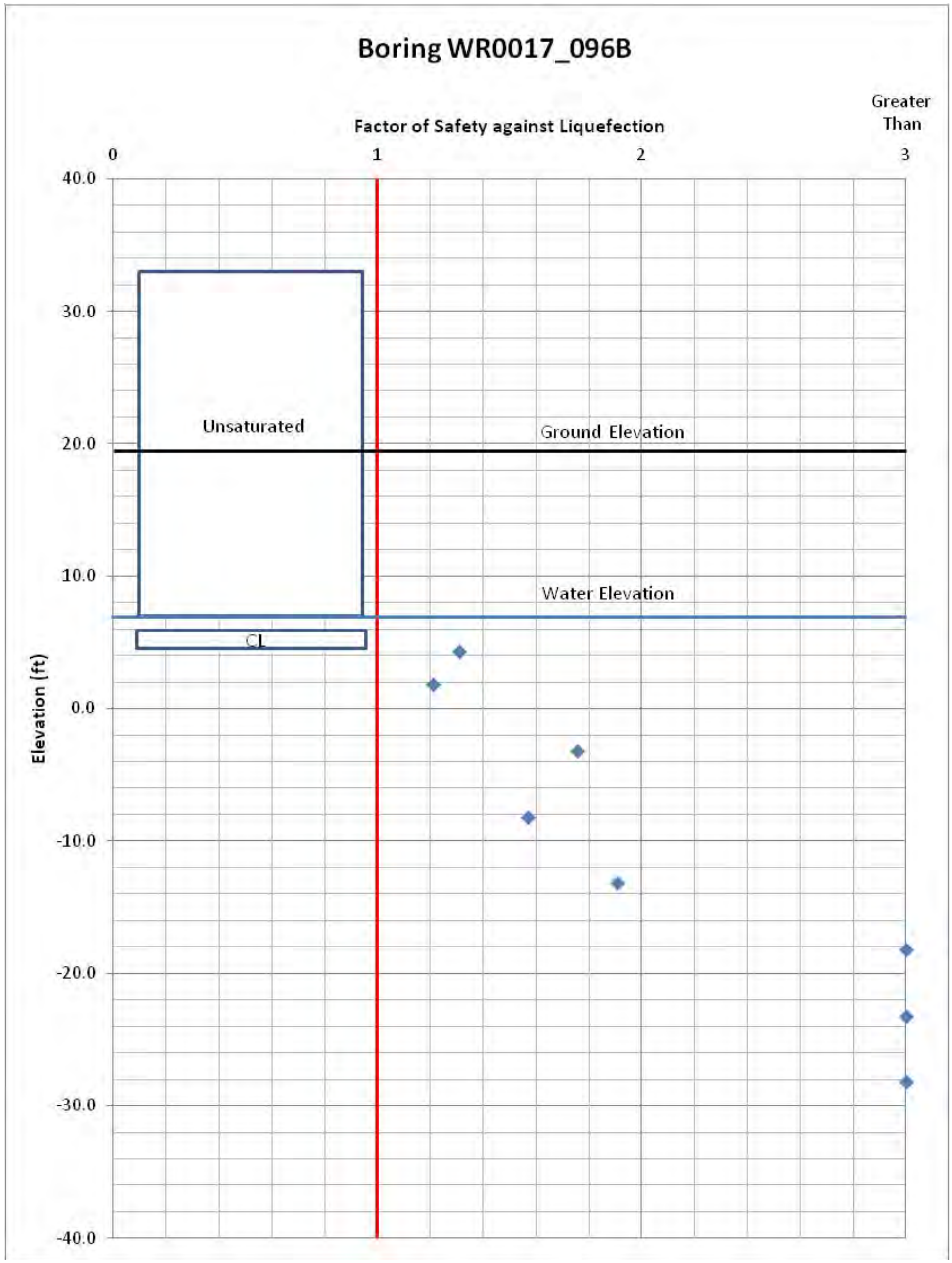


Fig. C-7. RD 17 South, Station 1784+83

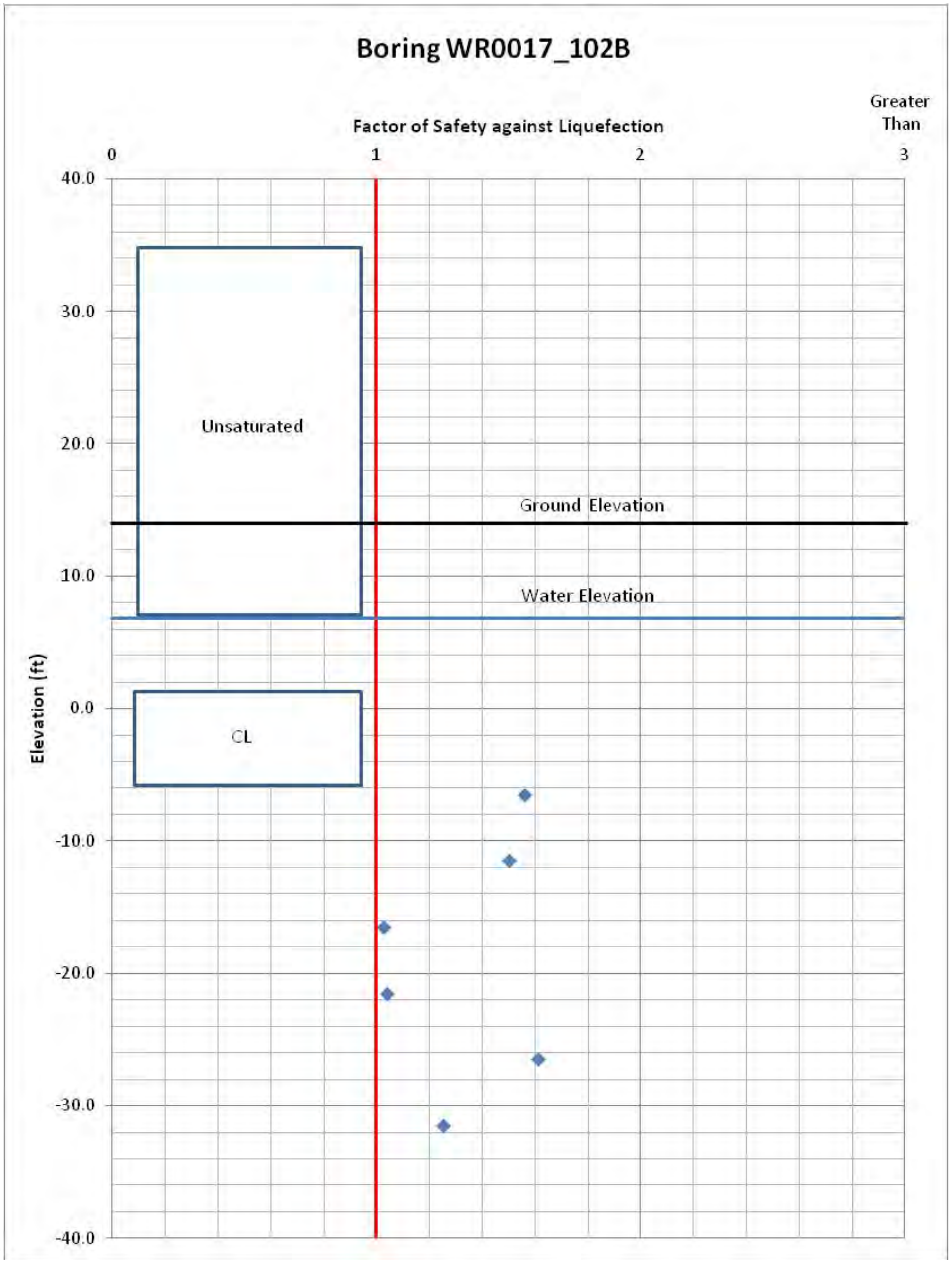


Fig. C-8. RD 17 South, Station 1825+94

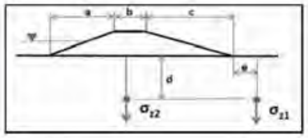
Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1007+42
Boring Number: WR0017_002B

Prepared by: Vlad Perlea
Checked by:

Date: 5/3/2013
Date:

Input Parameters

Embankment Crest Elevation (ft)	20.2 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	12.7 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.21
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	1.7 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	2.8 ft				120.0 pcf



Surcharge Information

Waterside/Upstream Slope, a (ft)	15.8 ft
Crest Width, b (ft)	40.0 ft
Landside/Downstream Slope, c (ft)	37.5 ft
Dist. of Boring from Levee Toe[1] (ft)	-57.5 ft
Embankment Height, H (ft)	7.5 ft

Boring WR0017_002B
Boring on the crest
SPT Ground Elevation Used in Analysis
20.20 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_σ	f parameter	K_α	FS against Liquefaction
1.0	19.2	10	GC	12	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	15.3	1.55	1.03	17.3	n.a.	1.00	#N/A	1.00	0.72	#N/A	#N/A
6.0	14.2	6	SM	15	Unsaturated	120	125	720.0	720.0	0.0	Embankment	Embankment	1.70	1	0.8	1.00	9.8	2.50	1.05	12.8	n.a.	0.99	#N/A	1.00	0.77	#N/A	#N/A
11.0	9.2	7	SM	15	Unsaturated	120	125	1319.3	1319.3	699.3	1320.0	1320.0	1.27	1	0.85	1.00	9.0	2.50	1.05	12.0	n.a.	0.97	0.13	1.00	0.78	1.00	#N/A
16.0	4.2	11	CL	94	Unsaturated	120	125	1911.3	1911.3	691.3	1920.0	1920.0	1.05	1	0.95	1.00	13.2	5.00	1.20	20.8	n.a.	0.96	0.13	1.00	0.74	1.00	#N/A
21.0	-0.8	9	CL	94	Clay	120	125	2503.1	2347.1	670.6	1638.0	1413.4	0.95	1	0.95	1.00	n.a.	5.00	1.20	n.a.	2.00	0.95	0.15	1.00	0.60	1.00	#N/A
26.0	-5.8	4	CL	94	Clay	120	125	3095.8	2627.8	838.3	2263.0	1726.4	0.90	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.94	0.17	1.00	0.60	1.00	#N/A
32.5	-12.3	17	CL	94	Clay	120	125	3855.9	2982.3	785.9	3075.5	2133.3	0.84	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.91	0.18	1.00	0.60	1.00	#N/A
36.0	-15.8	14	CL	94	Clay	120	125	4263.1	3171.1	755.6	3513.0	2352.4	0.82	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.88	0.18	1.00	0.60	0.96	#N/A
42.5	-22.3	19	CL	94	Clay	120	125	5019.2	3521.6	699.3	4325.5	2759.3	0.78	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.83	0.18	1.00	0.60	0.90	#N/A
46.0	-25.8	13	CH	100	Clay	120	125	5427.5	3711.5	670.0	4763.0	2978.4	0.76	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.80	0.17	1.00	0.60	0.87	#N/A
51.0	-30.8	13	SM	15		120	125	6012.7	3984.7	630.2	5388.0	3291.4	0.73	1	1	1.00	11.4	2.50	1.05	14.4	0.15	0.76	0.17	1.00	0.76	0.90	1.21
56.0	-35.8	18	CL	94	Clay	120	125	6800.7	4260.7	593.2	6013.0	3604.4	0.70	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.72	0.16	1.00	0.60	0.81	#N/A

NOTE
[1] "e" is the distance from landside toe, positive downstream and negative going upstream
[2] Soil description may be used to estimate fines content where lab testing is not available
Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1998 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001
Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space
[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used
[4] It is conservative to answer "No" if unsure about sampling method, answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

Updated April 2013

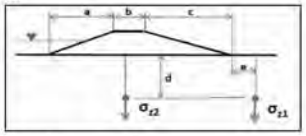
Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1048+79
Boring Number: WR0017_007B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:

Input Parameters

Embankment Crest Elevation (ft)	21.7 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	8.7 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.21
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	1.7 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	2.6 ft				120.0 pcf



Surcharge Information

Waterside/Upstream Slope, a (ft)	39.0 ft
Crest Width, b (ft)	15.0 ft
Landside/Downstream Slope, c (ft)	45.5 ft
Dist. of Boring from Levee Toe[1] (ft)	-53.0 ft
Embankment Height, H (ft)	13.0 ft

Boring WR0017_007B
Boring on the crest
SPT Ground Elevation Used in Analysis
21.70 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_σ	f parameter	K_α	FS against Liquefaction
1.0	20.7	17	GC	12	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	26.0	1.55	1.03	28.4	n.a.	1.00	#N/A	1.00	0.63	#N/A	#N/A
6.0	15.7	7	CL	94	Unsaturated	120	125	720.0	720.0	0.0	Embankment	Embankment	1.70	1	0.8	1.00	11.4	5.00	1.20	18.7	n.a.	0.99	#N/A	1.00	0.76	#N/A	#N/A
11.0	10.7	7	CL	94	Unsaturated	120	125	1320.0	1320.0	0.0	Embankment	Embankment	1.27	1	0.85	1.00	9.0	5.00	1.20	15.8	n.a.	0.97	#N/A	1.00	0.78	#N/A	#N/A
16.0	5.7	10	CL	94	Unsaturated	120	125	1916.6	1916.6	1556.6	1920.0	1920.0	1.05	1	0.95	1.00	12.0	5.00	1.20	19.4	n.a.	0.96	0.13	1.00	0.75	1.00	#N/A
22.5	-0.8	12	SC	23		120	125	2650.6	2494.6	1498.1	1158.0	933.4	0.92	1	0.95	1.00	12.6	4.06	1.10	17.9	0.19	0.95	0.16	1.00	0.74	1.00	1.79
26.0	-4.3	16	SC	23		120	125	3035.2	2660.8	1445.2	1595.5	1152.5	0.89	1	1	1.00	17.1	4.06	1.10	22.9	0.26	0.94	0.18	1.00	0.70	1.00	2.16
31.0	-9.3	1	CL	94	Clay	120	125	3575.5	2889.1	1360.5	2220.5	1465.5	0.86	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.92	0.19	1.00	0.60	1.00	#N/A
36.0	-14.3	12	CL	94	Clay	120	125	4113.9	3115.6	1273.9	2845.5	1778.6	0.82	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.88	0.19	1.00	0.60	1.00	#N/A
41.0	-19.3	38	ML	54		120	125	4655.1	3344.7	1190.1	3470.5	2091.5	0.80	1	1	1.00	36.3	5.00	1.20	48.5	2.00	0.84	0.19	1.00	0.60	1.00	3.00
46.0	-24.3	34	ML	54		120	125	5201.4	3579.0	1111.4	4095.5	2404.5	0.77	1	1	1.00	31.4	5.00	1.20	42.6	2.00	0.80	0.19	1.00	0.60	0.95	3.00
52.5	-30.8	28	CL	94	Clay	120	125	5920.4	3892.4	1017.9	4908.0	2811.4	0.74	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.75	0.18	1.00	0.60	0.89	#N/A
56.0	-34.3	9	CL	94	Clay	120	125	6311.8	4065.4	971.8	5345.5	3030.5	0.72	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.72	0.17	1.00	0.60	0.87	#N/A

NOTE
[1] "e" is the distance from landside toe, positive downstream and negative going upstream
[2] Soil description may be used to estimate fines content where lab testing is not available
Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1998 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001
Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space
[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used
[4] It is conservative to answer "No" if unsure about sampling method, answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

Updated April 2013

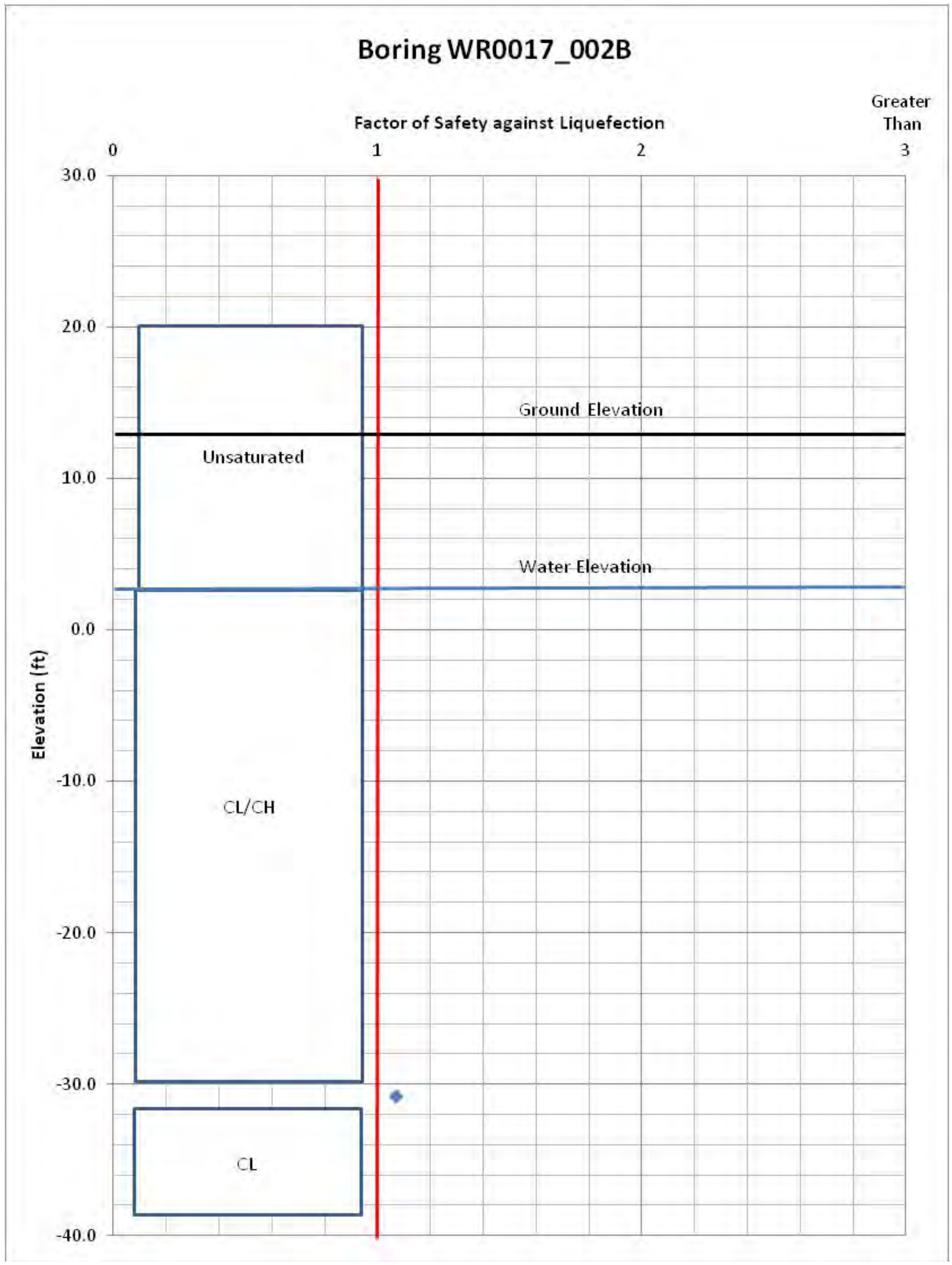


Fig. C-9. RD 17 North, Station 1007+42

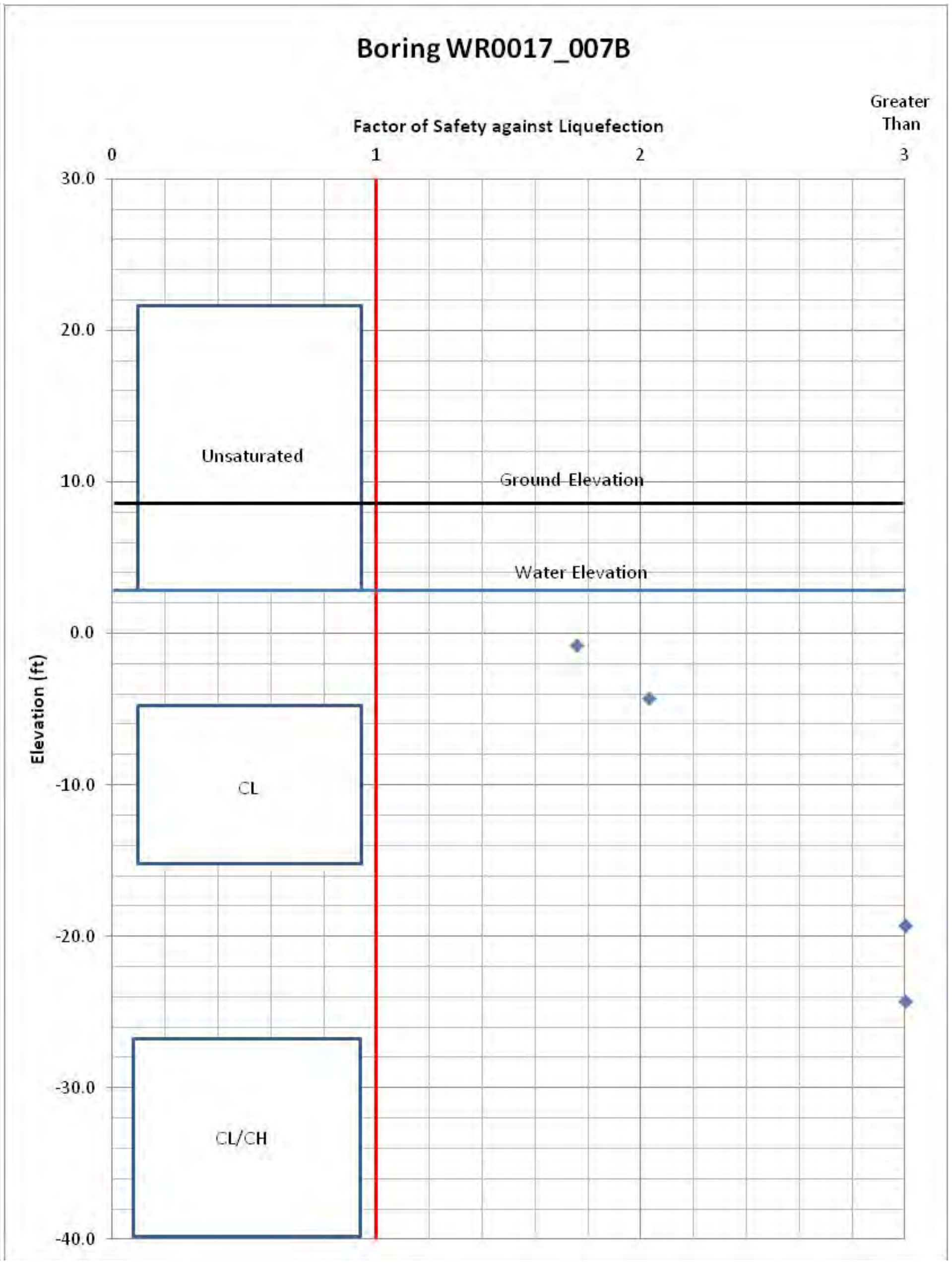


Fig. C-10. RD 17 North, Station 1048+79

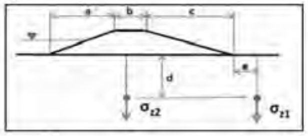
Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1099+90
Boring Number: WR0017_013B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:

Input Parameters

Embankment Crest Elevation (ft)	22.6 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	9.8 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.21
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	2.8 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	3.6 ft			120.0 pcf	



Surcharge Information

Waterside/Upstream Slope, a (ft)	31.2 ft
Crest Width, b (ft)	25.0 ft
Landside/Downstream Slope, c (ft)	52.0 ft
Dist. of Boring from Levee Toe[1] (ft)	-64.5 ft
Embankment Height, H (ft)	13.0 ft

Boring WR0017_013B
Boring on the crest
SPT Ground Elevation Used in Analysis
22.80 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_σ	f parameter	K_α	FS against Liquefaction
1.0	21.8	11	GC	12	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	16.8	1.55	1.03	18.9	n.a.	1.00	#N/A	1.00	0.70	#N/A	#N/A
6.5	14.3	15	CL	94	Unsaturated	120	125	1020.0	1020.0	0.0	Embankment	Embankment	1.44	1	0.85	1.00	22.0	5.00	1.20	31.4	n.a.	0.98	#N/A	1.00	0.66	#N/A	#N/A
13.5	9.3	18	SM	26	Unsaturated	120	125	1620.0	1620.0	1560.0	1620.0	1620.0	1.14	1	0.95	1.00	23.5	4.39	1.12	30.7	n.a.	0.97	0.13	1.00	0.65	1.00	#N/A
16.0	6.8	7	SM	30	Unsaturated	120	125	1916.7	1916.7	1556.7	1920.0	1920.0	1.05	1	0.95	1.00	8.4	4.71	1.15	14.4	n.a.	0.96	0.13	1.00	0.79	1.00	#N/A
21.0	1.8	6	CL	94	Clay	120	125	2504.3	2441.9	1539.3	969.0	856.7	0.93	1	0.95	1.00	n.a.	5.00	1.20	n.a.	2.00	0.95	0.15	1.00	0.60	1.00	#N/A
26.0	-3.2	3	CL	94	Clay	120	125	3062.6	2706.2	1492.6	1594.0	1169.7	0.88	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.94	0.17	1.00	0.60	1.00	#N/A
31.0	-8.2	24	ML/SM	48		120	125	3642.4	2956.0	1427.4	2219.0	1482.7	0.85	1	1	1.00	24.4	5.00	1.20	34.2	2.00	0.92	0.19	1.00	0.64	1.00	3.00
36.0	-13.2	3	CL	94	Clay	120	125	4193.6	3195.2	1353.6	2844.0	1795.7	0.81	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.88	0.19	1.00	0.60	1.00	#N/A
41.0	-18.2	20	CL	94	Clay	120	125	4742.8	3432.4	1277.8	3469.0	2108.7	0.79	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.84	0.19	1.00	0.60	1.00	#N/A
46.0	-23.2	18	CL	94	Clay	120	125	5293.6	3671.2	1203.6	4094.0	2421.7	0.76	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.80	0.16	1.00	0.60	0.95	#N/A
51.0	-28.2	18	CL	94	Clay	120	125	5848.1	3918.7	1133.1	4719.0	2734.7	0.74	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.76	0.16	1.00	0.60	0.90	#N/A
56.0	-33.2	20	CH	100	Clay	120	125	6407.1	4160.7	1067.1	5344.0	3047.7	0.71	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.72	0.17	1.00	0.60	0.86	#N/A

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

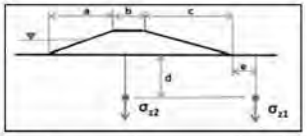
Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1151+06
Boring Number: WR0017_019B

Prepared by: Vlad Perlee
Checked by:

Date: 5/6/2013
Date:

Input Parameters

Embankment Crest Elevation (ft)	22.9 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	9.9 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.21
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-0.1 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	4.4 ft			120.0 pcf	



Surcharge Information

Waterside/Upstream Slope, a (ft)	19.5 ft
Crest Width, b (ft)	23.0 ft
Landside/Downstream Slope, c (ft)	28.6 ft
Dist. of Boring from Levee Toe[1] (ft)	-40.1 ft
Embankment Height, H (ft)	13.0 ft

Boring WR0017_019B
Boring on the crest
SPT Ground Elevation Used in Analysis
22.90 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_σ	f parameter	K_α	FS against Liquefaction
6.0	16.9	6	CL	80	Unsaturated	120	125	720.0	720.0	0.0	Embankment	Embankment	1.70	1	0.8	1.00	9.8	5.00	1.20	16.8	n.a.	0.99	#N/A	1.00	0.77	#N/A	#N/A
10.0	12.9	7	SC-SM	30	Unsaturated	120	125	1200.0	1200.0	0.0	Embankment	Embankment	1.33	1	0.85	1.00	9.5	4.71	1.15	15.7	n.a.	0.98	#N/A	1.00	0.78	#N/A	#N/A
14.0	8.9	6	SC	16	Unsaturated	120	125	1679.9	1679.9	1559.9	1680.0	1680.0	1.12	1	0.95	1.00	7.7	2.77	1.05	10.9	n.a.	0.97	0.13	1.00	0.79	1.00	#N/A
18.5	4.4	6	CL	94	Unsaturated	120	125	2206.2	2206.2	1546.2	2220.0	2220.0	0.98	1	0.95	1.00	6.7	5.00	1.20	13.0	n.a.	0.96	0.13	1.00	0.80	0.99	#N/A
22.5	0.4	8	SP-SC	6		120	125	2643.4	2643.4	1503.4	1160.0	910.4	0.69	1	0.95	1.00	8.2	0.03	1.00	8.2	0.10	0.95	0.16	1.00	0.79	1.00	[illegible]
26.0	-3.1	9	CL	94	Clay	120	125	3020.2	2833.6	1445.2	1597.5	1129.5	0.86	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.94	0.18	1.00	0.60	1.00	#N/A
31.0	-8.1	16	ML	50		120	125	3544.6	3045.4	1344.6	2222.5	1442.5	0.83	1	1	1.00	16.0	5.00	1.20	24.2	0.26	0.92	0.19	1.00	0.71	1.00	2.14
36.0	-13.1	16	ML	50		120	125	4063.8	3252.6	1238.8	2847.5	1755.5	0.81	1	1	1.00	17.4	5.00	1.20	25.9	0.31	0.88	0.20	1.00	0.70	1.00	2.39
41.0	-18.1	10	CL	94	Clay	120	125	4587.4	3464.2	1137.4	3472.5	2068.5	0.78	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.84	0.19	1.00	0.60	1.00	#N/A
46.0	-23.1	19	CL	94	Clay	120	125	5119.4	3684.2	1044.4	4097.5	2381.5	0.76	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.80	0.19	1.00	0.60	0.95	#N/A
51.0	-28.1	29	SM	26		120	125	5661.0	3913.8	961.0	4722.5	2694.5	0.74	1	1	1.00	25.6	4.39	1.12	33.1	2.00	0.76	0.16	1.00	0.63	0.92	3.00
56.0	-33.1	17	CL	94	Clay	120	125	6211.9	4152.7	886.9	5347.5	3007.5	0.71	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.72	0.17	1.00	0.60	0.87	#N/A

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

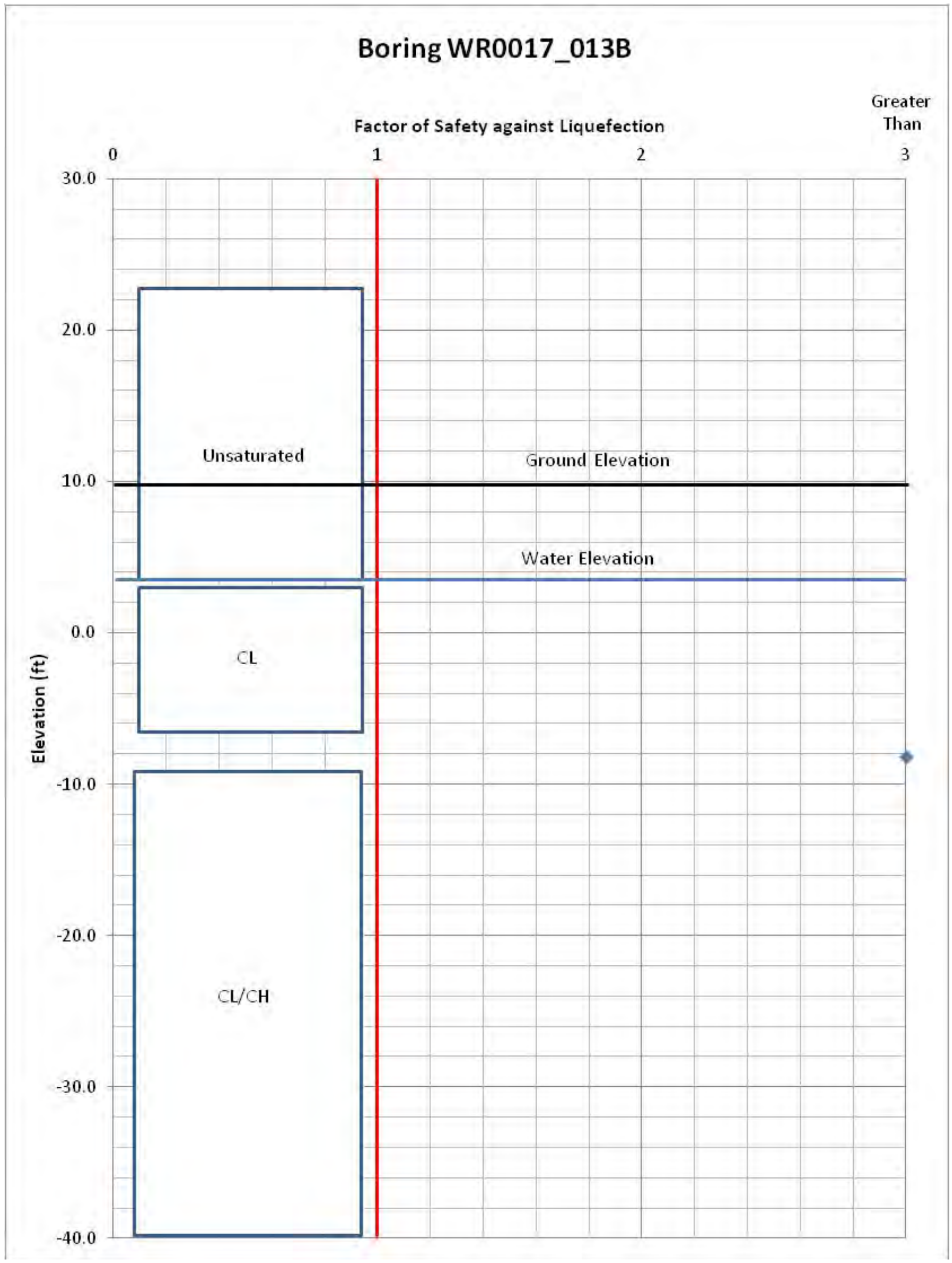


Fig. C-11. RD 17 North, Station 1099+90

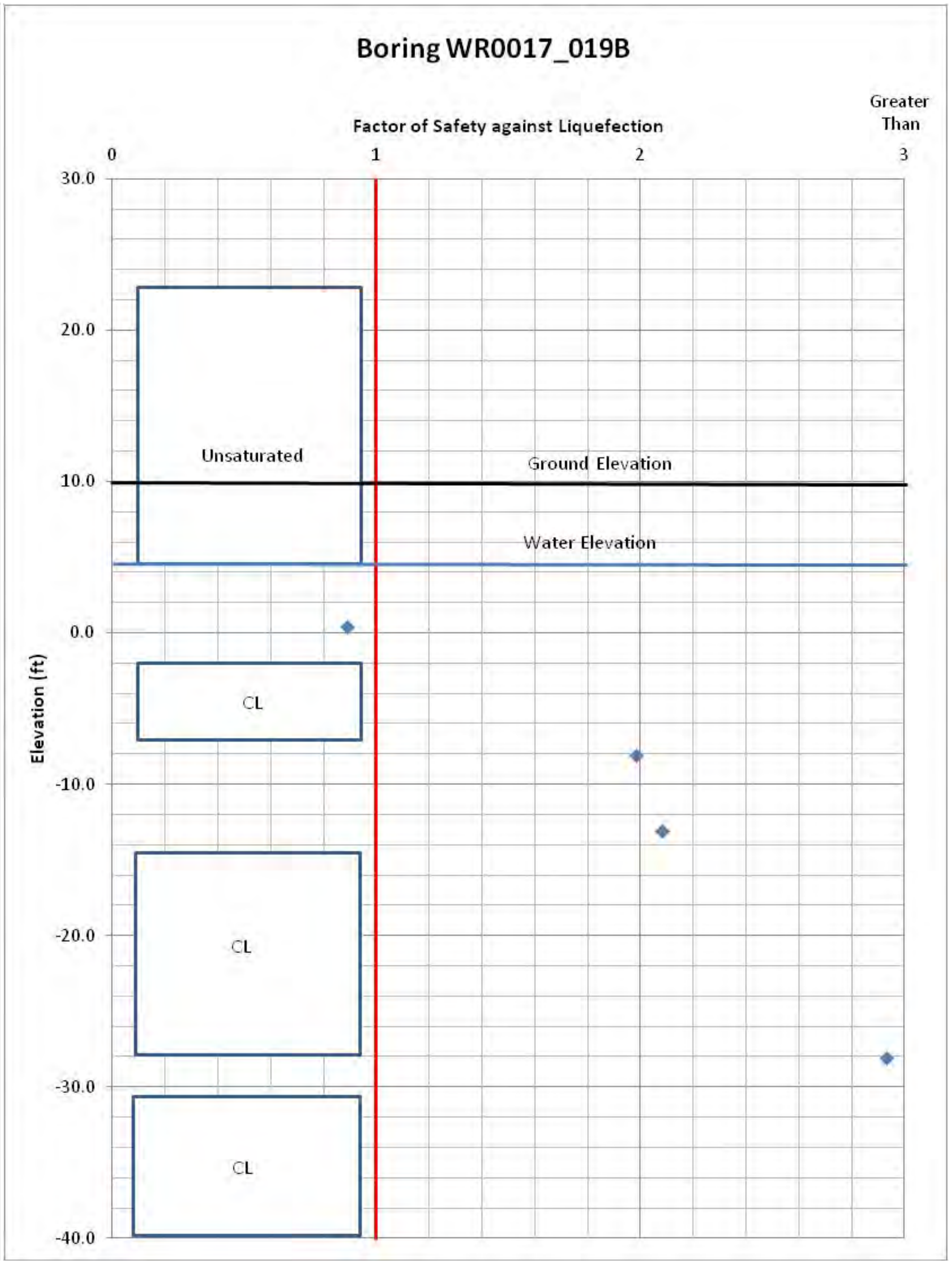


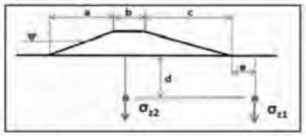
Fig. C-12. RD 17 North, Station 1151+06

Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1191+43
Boring Number: WR0017_024B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	22.8 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	7.8 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.21
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-0.2 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	4.6 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	31.5 ft
Crest Width, b (ft)	34.0 ft
Landside/Downstream Slope, c (ft)	36.0 ft
Dist. of Boring from Levee Toe[1] (ft)	53.0 ft
Embankment Height, H (ft)	15.0 ft

Boring	WR0017_024B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
22.80 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
1.0	21.8	13	GC	12	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	19.9	1.55	1.03	22.1	n.a.	1.00	#N/A	1.00	0.68	#N/A	#N/A
6.0	16.8	13	CL	63	Unsaturated	120	125	720.0	720.0	0.0	Embankment	Embankment	1.70	1	0.8	1.00	21.2	5.00	1.20	30.5	n.a.	0.99	#N/A	1.00	0.67	#N/A	#N/A
11.0	11.8	9	CL	94	Unsaturated	120	125	1320.0	1320.0	0.0	Embankment	Embankment	1.27	1	0.85	1.00	11.6	5.00	1.20	18.9	n.a.	0.97	#N/A	1.00	0.75	#N/A	#N/A
16.5	4.3	8	CH	100	Clay	120	125	2218.5	2218.5	1798.5	421.5	402.8	0.98	1	0.95	1.00	n.a.	5.00	1.20	n.a.	2.00	0.96	0.14	1.00	0.60	1.00	#N/A
23.5	-0.7	6	CL	69	Clay	120	125	2804.1	2772.9	1781.6	1046.5	715.8	0.87	1	0.95	1.00	n.a.	5.00	1.20	n.a.	2.00	0.95	0.19	1.00	0.60	1.00	#N/A
26.0	-3.2	6	SM	15		120	125	3098.6	2911.4	1763.6	1359.0	872.3	0.85	1	1	1.00	6.1	2.50	1.05	8.9	0.10	0.94	0.20	1.00	0.80	1.00	[illegible]
31.0	-8.2	14	ML	90		120	125	3669.4	3170.2	1709.4	1984.0	1185.3	0.82	1	1	1.00	13.7	5.00	1.20	21.5	0.23	0.92	0.21	1.00	0.73	1.00	1.67
36.0	-13.2	16	SC	26		120	125	4222.5	3411.3	1637.5	2609.0	1498.3	0.79	1	1	1.00	15.1	4.56	1.14	21.8	0.24	0.88	0.21	1.00	0.72	1.00	1.71
41.0	-18.2	3	SC	28		120	125	4766.2	3643.0	1556.2	3234.0	1811.3	0.76	1	1	1.00	2.7	4.56	1.14	7.7	0.09	0.84	0.20	1.00	0.80	1.00	[illegible]
46.0	-23.2	31	SC	27		120	125	5306.7	3871.5	1471.7	3859.0	2124.3	0.74	1	1	1.00	27.5	4.48	1.13	35.6	2.00	0.80	0.20	1.00	0.62	1.00	3.00
51.0	-28.2	10	CH	100	Clay	120	125	5848.3	4101.1	1388.3	4484.0	2437.3	0.72	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.76	0.19	1.00	0.60	0.95	#N/A
56.0	-33.2	21	CL	94	Clay	120	125	6393.3	4334.1	1308.3	5109.0	2750.3	0.70	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.72	0.18	1.00	0.60	0.90	#N/A

NOTE
[1] "e" is the distance from landside toe, positive downstream and negative going upstream
[2] Soil description may be used to estimate fines content where lab testing is not available
Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001
Surcharge from embankment calculation is presented in Poulos & Davis (1974) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space
[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used
[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

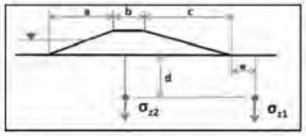
Updated April 2013

Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1231+82
Boring Number: WR0017_029B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	23.7 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	11.2 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.21
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	0.7 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	4.8 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	25.0 ft
Crest Width, b (ft)	35.0 ft
Landside/Downstream Slope, c (ft)	38.3 ft
Dist. of Boring from Levee Toe[1] (ft)	-53.6 ft
Embankment Height, H (ft)	12.5 ft

Boring	WR0017_029B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
23.70 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
1.0	22.7	21	GC	12	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	32.1	1.55	1.03	34.7	n.a.	1.00	#N/A	1.00	0.60	#N/A	#N/A
6.0	17.7	6	SC	51	Unsaturated	120	125	720.0	720.0	0.0	Embankment	Embankment	1.70	1	0.8	1.00	9.8	5.00	1.20	16.8	n.a.	0.99	#N/A	1.00	0.77	#N/A	#N/A
12.5	11.2	8	SM	38	Unsaturated	120	125	1500.0	1500.0	1500.0	Embankment	Embankment	1.19	1	0.85	1.00	9.7	5.00	1.20	16.6	n.a.	0.97	#N/A	1.00	0.77	#N/A	#N/A
17.5	6.2	8	CL	94	Unsaturated	120	125	2096.4	2096.4	1496.4	2100.0	2100.0	1.00	1	0.95	1.00	9.2	5.00	1.20	16.0	n.a.	0.96	0.13	1.00	0.78	1.00	#N/A
22.5	1.2	6	SM	37		120	125	2675.4	2675.4	1475.4	1216.0	993.4	0.89	1	0.95	1.00	6.1	5.00	1.20	12.3	0.13	0.95	0.16	1.00	0.80	1.00	1.27
27.5	-3.8	11	SM	15		120	125	3254.6	2973.8	1432.1	1843.0	1306.4	0.84	1	1	1.00	11.1	2.50	1.05	14.2	0.15	0.94	0.16	1.00	0.76	1.00	1.26
32.5	-8.8	19	SP	4		120	125	3819.4	3226.6	1371.9	2468.0	1619.4	0.81	1	1	1.00	18.5	0.00	1.00	18.5	0.20	0.91	0.19	1.00	0.69	1.00	1.56
36.0	-12.3	20	CL	94	Clay	120	125	4208.8	3397.6	1323.8	2905.5	1838.5	0.79	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.88	0.19	1.00	0.60	1.00	#N/A
41.0	-17.3	9	CL	94	Clay	120	125	4761.5	3638.3	1251.5	3530.5	2151.5	0.76	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.84	0.19	1.00	0.60	0.99	#N/A
46.0	-22.3	17	SC	42		120	125	5313.8	3878.6	1178.8	4155.5	2464.5	0.74	1	1	1.00	15.1	5.00	1.20	23.1	0.26	0.80	0.18	1.00	0.72	0.96	2.02
51.0	-27.3	20	SC	26		120	125	5866.6	4121.4	1106.6	4780.5	2777.5	0.72	1	1	1.00	17.2	4.39	1.12	23.7	0.27	0.76	0.18	1.00	0.70	0.92	2.08
56.0	-32.3	27	SC	26		120	125	6427.4	4368.2	1042.4	5405.5	3090.5	0.70	1	1	1.00	22.6	4.39	1.12	29.7	0.45	0.72	0.17	1.00	0.66	0.88	3.00

NOTE
[1] "e" is the distance from landside toe, positive downstream and negative going upstream
[2] Soil description may be used to estimate fines content where lab testing is not available
Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001
Surcharge from embankment calculation is presented in Poulos & Davis (1974) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space
[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used
[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

Updated April 2013

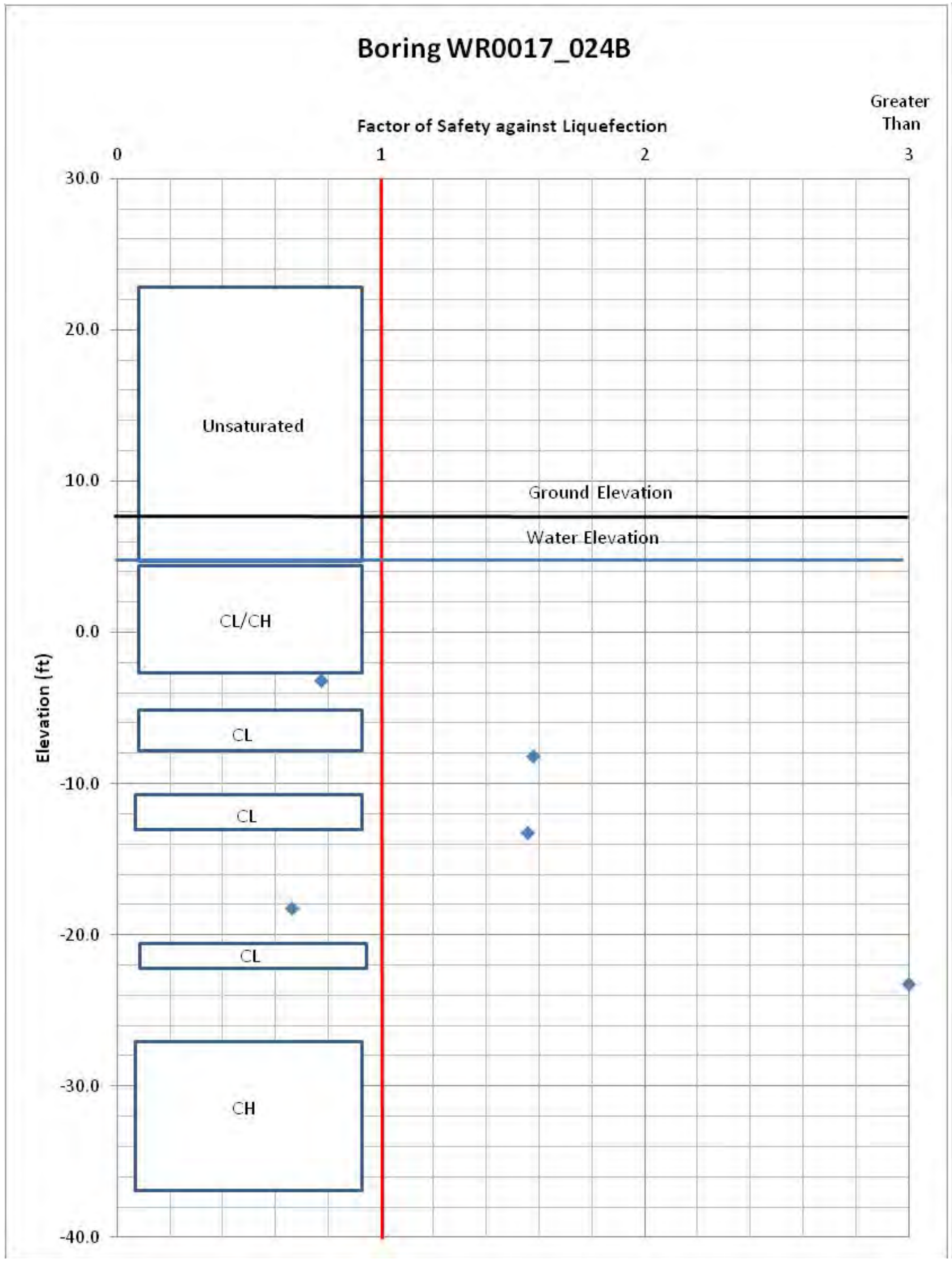


Fig. C-13. RD 17 North, Station 1191+43

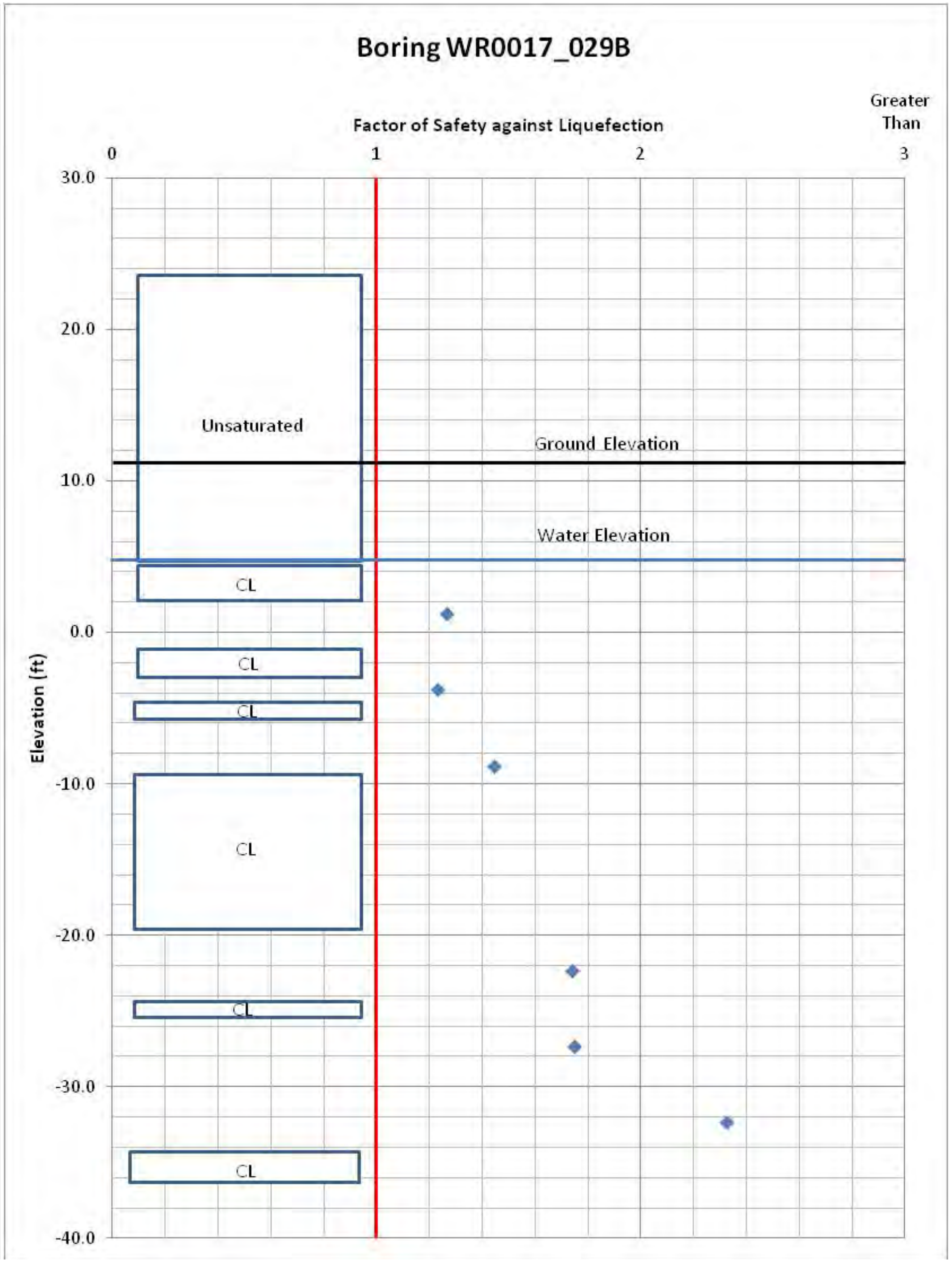
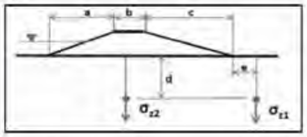


Fig. C-14. RD 17 North, Station 1231+82

Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1292+29
Boring Number: WR0017_036B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:



Input Parameters					
Embankment Crest Elevation (ft)	25.2 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	12.7 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.21
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	2.2 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	4.8 ft			120.0 pcf	

Surcharge Information	
Waterside/Upstream Slope, a (ft)	33.8 ft
Crest Width, b (ft)	19.0 ft
Landside/Downstream Slope, c (ft)	37.5 ft
Dist. of Boring from Levee Toe[1] (ft)	-47.0 ft
Embankment Height, H (ft)	12.5 ft

Boring **WR0017_036B**
Boring on the crest
SPT Ground Elevation Used in Analysis
25.20 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_σ	f parameter	K_α	FS against Liquefaction
1.0	24.2	8	GC	12	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	12.2	1.55	1.03	14.2	n.a.	1.00	#N/A	1.00	0.75	#N/A	#N/A
6.0	19.2	6	CL	94	Unsaturated	120	125	720.0	720.0	0.0	Embankment	Embankment	1.70	1	0.9	1.00	8.8	5.00	1.20	16.8	n.a.	0.99	#N/A	1.00	0.77	#N/A	#N/A
12.5	12.7	5	ML	50	Unsaturated	120	125	1500.0	1500.0	1500.0	Embankment	Embankment	1.19	1	0.85	1.00	6.1	5.00	1.20	12.3	n.a.	0.97	#N/A	1.00	0.60	#N/A	#N/A
16.0	9.2	4	ML	50	Unsaturated	120	125	1916.2	1916.2	1496.2	1920.0	1920.0	1.05	1	0.95	1.00	4.8	5.00	1.20	10.8	n.a.	0.96	0.13	1.00	0.60	1.00	#N/A
21.0	4.2	6	CL	94	Clay	120	125	2480.6	2480.6	1460.6	1023.0	985.6	0.92	1	0.95	1.00	n.a.	5.00	1.20	n.a.	2.00	0.95	0.13	1.00	0.60	1.00	#N/A
27.5	-2.3	8	CL	94	Clay	120	125	3190.3	2909.5	1367.8	1835.5	1392.5	0.85	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.94	0.17	1.00	0.60	1.00	#N/A
31.0	-5.8	32	SM	15		120	125	3568.3	3069.1	1308.3	2273.0	1611.8	0.83	1	1	1.00	31.9	2.50	1.05	35.9	2.00	0.92	0.18	1.00	0.60	1.00	3.00
36.0	-10.8	25	CL	94	Clay	120	125	4106.8	3294.8	1221.0	2898.0	1924.6	0.80	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.88	0.18	1.00	0.60	1.00	#N/A
41.0	-15.8	11	CL	94	Clay	120	125	4646.3	3523.1	1136.3	3523.0	2237.6	0.77	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.84	0.18	1.00	0.60	0.98	#N/A
46.0	-20.8	20	CL	94	Clay	120	125	5191.7	3756.5	1056.7	4148.0	2550.6	0.75	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.80	0.18	1.00	0.60	0.93	#N/A
51.0	-25.8	12	SM	15		120	125	5743.8	3996.4	983.6	4773.0	2863.5	0.73	1	1	1.00	10.5	2.50	1.05	13.5	0.15	0.76	0.17	1.00	0.77	0.93	1.11
56.0	-30.8	17	SM	15		120	125	6302.0	4242.8	917.0	5398.0	3176.6	0.71	1	1	1.00	14.4	2.50	1.05	17.6	0.19	0.72	0.17	1.00	0.73	0.89	1.51
61.0	-35.8	34	SM	15		120	125	6866.8	4495.6	856.8	6023.0	3489.6	0.69	1	1	1.00	28.0	2.50	1.05	31.8	2.00	0.68	0.16	1.00	0.62	0.82	3.00

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream

[2] Soil description may be used to estimate fines content where lab testing is not available

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001

Surcharge from embankment calculation is presented in Poulos & Davis (1976) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used

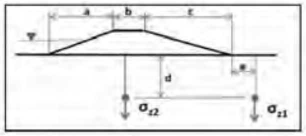
[4] It is conservative to answer "No" if unsure about sampling method, answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

Updated April 2013

Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1330+01
Boring Number: WR0017_041B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:



Input Parameters					
Embankment Crest Elevation (ft)	25.7 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	14.2 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.21
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	2.7 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	5.0 ft			120.0 pcf	

Surcharge Information	
Waterside/Upstream Slope, a (ft)	42.6 ft
Crest Width, b (ft)	18.0 ft
Landside/Downstream Slope, c (ft)	39.1 ft
Dist. of Boring from Levee Toe[1] (ft)	-48.1 ft
Embankment Height, H (ft)	11.5 ft

Boring **WR0017_041B**
Boring on the crest
SPT Ground Elevation Used in Analysis
25.70 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_σ	f parameter	K_α	FS against Liquefaction
1.0	24.7	18	GC	12	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	27.5	1.55	1.03	30.0	n.a.	1.00	#N/A	1.00	0.62	#N/A	#N/A
6.0	19.7	10	ML	86	Unsaturated	120	125	720.0	720.0	0.0	Embankment	Embankment	1.70	1	0.8	1.00	16.3	5.00	1.20	24.6	n.a.	0.99	#N/A	1.00	0.71	#N/A	#N/A
11.0	14.7	7	ML	86	Unsaturated	120	125	1320.0	1320.0	0.0	Embankment	Embankment	1.27	1	0.85	1.00	9.0	5.00	1.20	15.8	n.a.	0.97	#N/A	1.00	0.78	#N/A	#N/A
17.5	8.2	7	SP	2	Unsaturated	120	125	2085.4	2085.4	1365.4	2100.0	2100.0	1.01	1	0.95	1.00	8.0	0.00	1.00	8.0	n.a.	0.96	0.13	1.00	0.79	1.00	#N/A
21.0	4.7	5	ML	79		120	125	2475.3	2475.3	1335.3	1141.5	1122.8	0.92	1	0.95	1.00	5.3	5.00	1.20	11.3	0.12	0.95	0.13	1.00	0.60	1.00	1.42
26.0	-0.3	17	SP-SM	8		120	125	3027.9	2840.7	1272.9	1766.5	1435.8	0.86	1	1	1.00	17.6	0.30	1.01	18.1	0.19	0.94	0.16	1.00	0.70	1.00	1.84
31.0	-5.3	19	SP-SM	8		120	125	3580.3	3081.1	1200.3	2391.5	1746.8	0.83	1	1	1.00	18.9	0.30	1.01	19.4	0.21	0.92	0.17	1.00	0.69	1.00	1.62
36.0	-10.3	31	SP-SM	8		120	125	4130.8	3319.6	1125.8	3016.5	2061.8	0.80	1	1	1.00	29.7	0.30	1.01	30.4	2.00	0.88	0.18	1.00	0.60	1.00	3.00
41.0	-15.3	17	CL	94	Clay	120	125	4683.4	3560.2	1052.4	3641.5	2374.8	0.77	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.84	0.19	1.00	0.60	0.95	#N/A
46.0	-20.3	31	SC	26		120	125	5240.2	3805.0	985.2	4266.5	2687.8	0.75	1	1	1.00	27.7	4.39	1.12	35.5	2.00	0.80	0.17	1.00	0.62	0.91	3.00
52.5	-26.8	21	CL	94	Clay	120	125	5971.3	4130.5	903.8	5079.0	3094.7	0.72	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.75	0.17	1.00	0.60	0.86	#N/A
56.0	-30.3	8	CL	94	Clay	120	125	6368.7	4309.5	863.7	5516.5	3313.8	0.70	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.72	0.16	1.00	0.60	0.84	#N/A
61.0	-35.3	19	SC	50		120	125	6940.5	4569.3	810.5	6141.5	3626.8	0.66	1	1	1.00	15.5	5.00	1.20	23.6	0.27	0.68	0.16	1.00	0.71	0.86	2.19

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream

[2] Soil description may be used to estimate fines content where lab testing is not available

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001

Surcharge from embankment calculation is presented in Poulos & Davis (1976) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used

[4] It is conservative to answer "No" if unsure about sampling method, answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

Updated April 2013

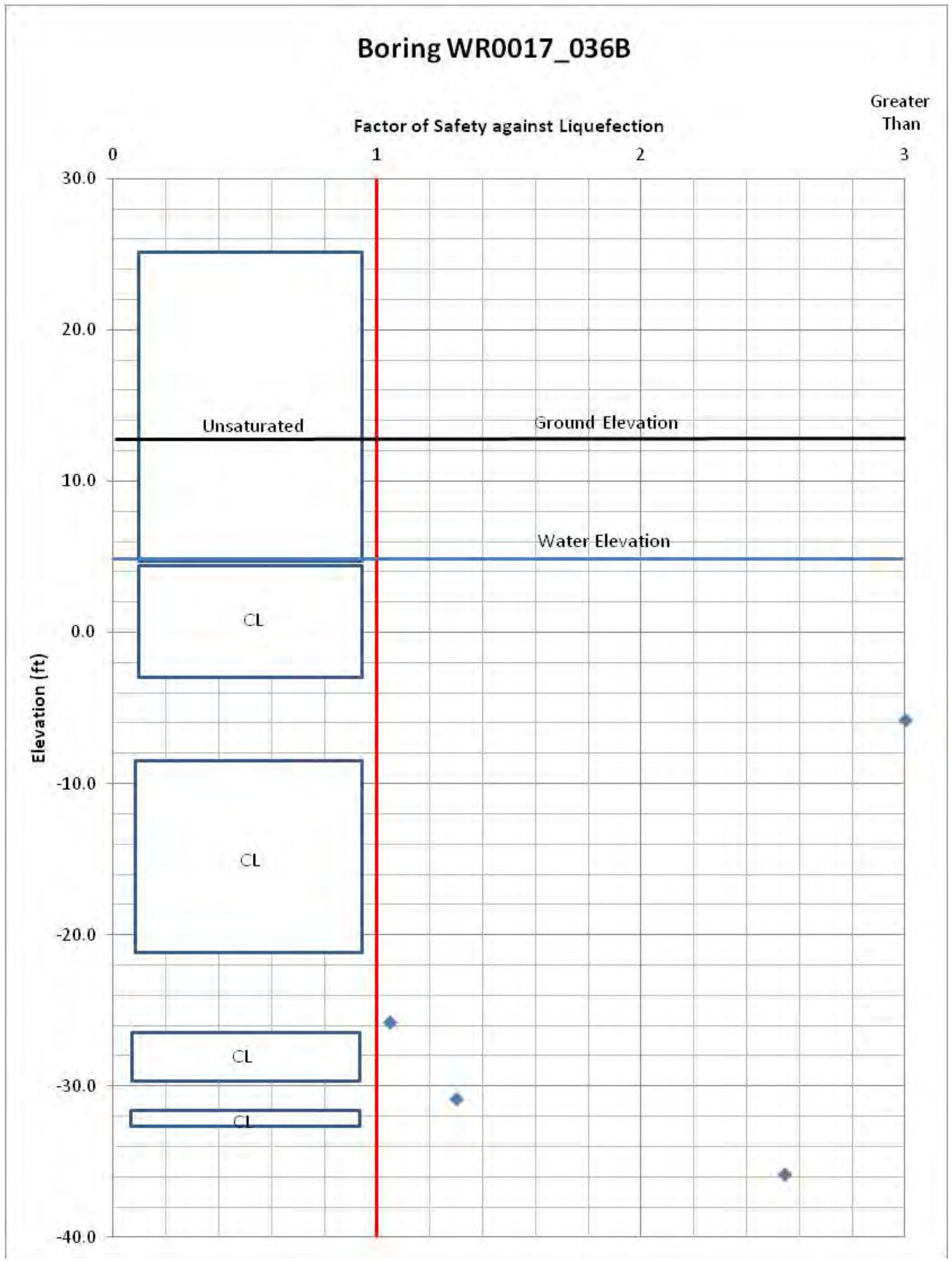


Fig. C-15. RD 17 North, Station 1292+29

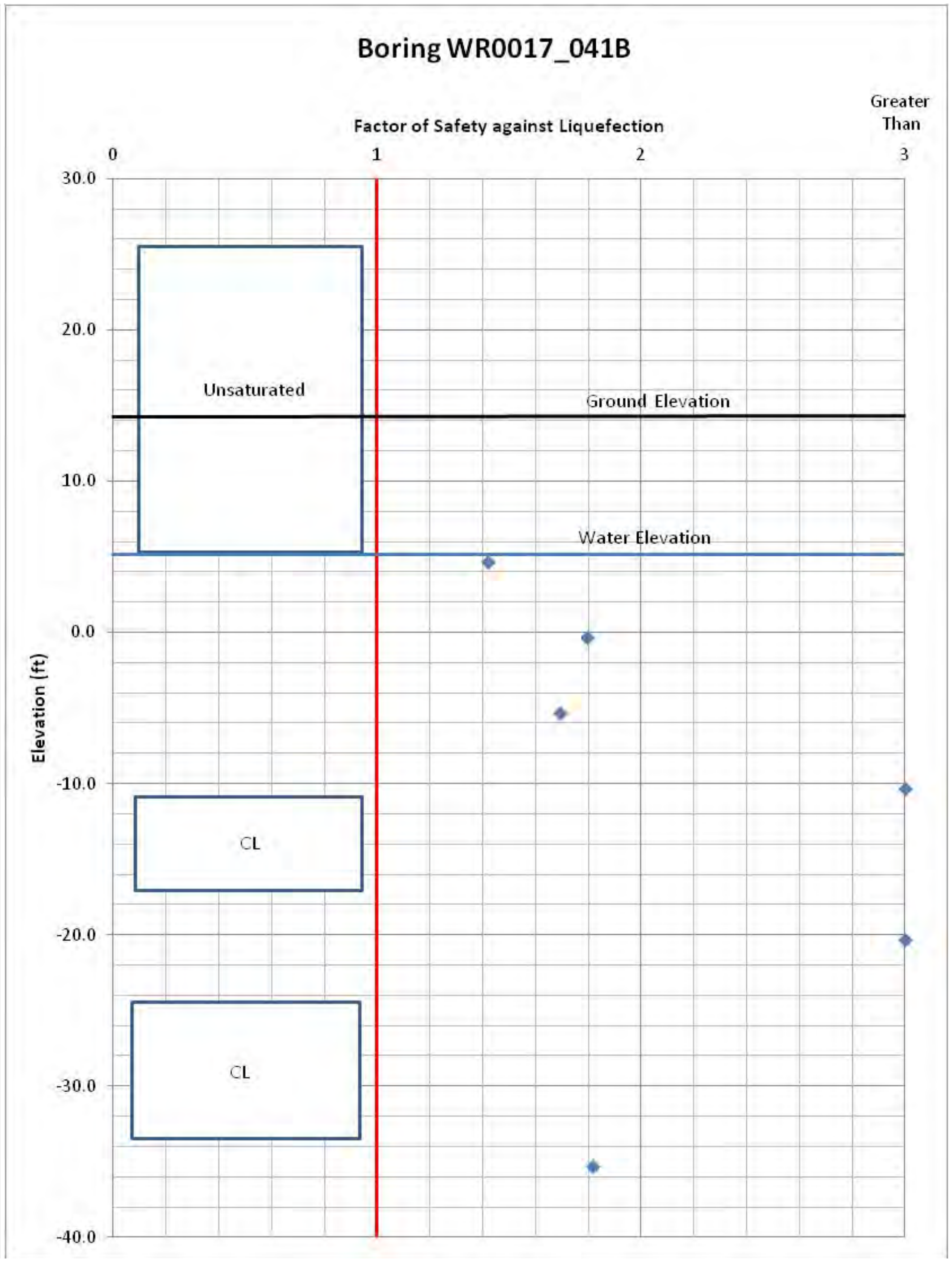


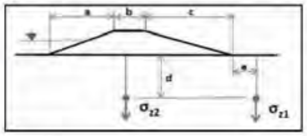
Fig. C-16. RD 17 North, Station 1330+01

Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1377+73
Boring Number: WR0017_047B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	27.2 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	14.2 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.21
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	4.2 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	5.3 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	33.6 ft
Crest Width, b (ft)	19.0 ft
Landside/Downstream Slope, c (ft)	32.5 ft
Dist. of Boring from Levee Toe[1] (ft)	-42.0 ft
Embankment Height, H (ft)	13.0 ft

Boring	WR0017_047B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
27.20 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_E	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
1.0	26.2	15	GC	12	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	23.0	1.55	1.03	25.2	n.a.	1.00	#N/A	1.00	0.65	#N/A	#N/A
7.5	19.7	21	SC	32	Unsaturated	120	125	900.0	900.0	0.0	Embankment	Embankment	1.53	1	0.85	1.00	32.8	4.83	1.17	43.3	n.a.	0.98	#N/A	1.00	0.60	#N/A	#N/A
11.0	16.2	5	SP	4	Unsaturated	120	125	1320.0	1320.0	0.0	Embankment	Embankment	1.27	1	0.85	1.00	6.5	0.00	1.00	6.5	n.a.	0.97	#N/A	1.00	0.80	#N/A	#N/A
16.0	11.2	6	ML	50	Unsaturated	120	125	1917.3	1917.3	1557.3	1920.0	1920.0	1.05	1	0.95	1.00	6.6	5.00	1.20	12.9	n.a.	0.96	0.13	1.00	0.80	1.00	#N/A
21.0	6.2	11	SC	14	Unsaturated	120	125	2482.3	2482.3	1522.3	2520.0	2520.0	0.92	1	0.95	1.00	11.6	2.20	1.04	14.3	n.a.	0.95	0.13	1.00	0.75	0.96	#N/A
26.0	1.2	5	SC-SM	33		120	125	3024.4	2837.2	1449.4	1580.5	1324.7	0.86	1	1	1.00	5.2	4.88	1.18	11.0	0.12	0.94	0.15	1.00	0.80	1.00	1.20
31.0	-3.8	11	SM	29		120	125	3558.5	3059.3	1358.5	2205.5	1637.7	0.83	1	1	1.00	11.0	4.64	1.15	17.2	0.18	0.92	0.17	1.00	0.76	1.00	1.62
36.0	-8.8	17	SM	29		120	125	4088.6	3277.4	1263.6	2830.6	1950.7	0.80	1	1	1.00	16.4	4.64	1.15	23.4	0.26	0.88	0.17	1.00	0.71	1.00	2.27
41.0	-13.8	23	SP-SC	10		120	125	4621.8	3498.4	1171.6	3455.5	2263.7	0.78	1	1	1.00	21.5	0.87	1.02	22.8	0.25	0.84	0.18	1.00	0.67	0.98	2.13
47.5	-20.3	16	SP-SC	10		120	125	5324.0	3795.2	1061.6	4268.0	2670.6	0.75	1	1	1.00	14.3	0.87	1.02	15.5	0.17	0.79	0.17	1.00	0.73	0.94	1.36
51.0	-23.8	18	SP-SC	10		120	125	5707.3	3960.1	1007.3	4705.5	2889.7	0.73	1	1	1.00	15.8	0.87	1.02	17.0	0.19	0.76	0.17	1.00	0.71	0.91	1.47
56.0	-28.8	19	ML	69		120	125	6261.4	4202.2	936.4	5330.6	3202.7	0.71	1	1	1.00	16.2	5.00	1.20	24.4	0.28	0.72	0.16	1.00	0.71	0.89	2.29
61.0	-33.8	7	CL	94	Clay	120	125	6822.5	4451.3	872.5	5955.5	3515.7	0.69	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.68	0.16	1.00	0.60	0.82	#N/A

NOTE
[1] "e" is the distance from landside toe, positive downstream and negative going upstream
[2] Soil description may be used to estimate fines content where lab testing is not available
Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001
Surcharge from embankment calculation is presented in Poulos & Davis (1976) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space
[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used
[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

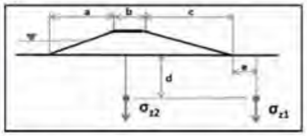
Updated April 2013

Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1416+93
Boring Number: WR0017_052B

Prepared by: Vlad Perlea
Checked by:

Date: 5/7/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	27.6 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	14.1 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.225
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	5.1 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	5.5 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	32.4 ft
Crest Width, b (ft)	16.0 ft
Landside/Downstream Slope, c (ft)	29.7 ft
Dist. of Boring from Levee Toe[1] (ft)	-37.7 ft
Embankment Height, H (ft)	13.5 ft

Boring	WR0017_052B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
27.60 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_E	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
1.0	26.6	18	ML	50	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	27.5	5.00	1.20	38.0	n.a.	1.00	#N/A	1.00	0.62	#N/A	#N/A
7.5	20.1	21	SM	43	Unsaturated	120	125	900.0	900.0	0.0	Embankment	Embankment	1.53	1	0.85	1.00	32.8	5.00	1.20	44.4	n.a.	0.98	#N/A	1.00	0.60	#N/A	#N/A
11.0	16.6	3	SM	43	Unsaturated	120	125	1320.0	1320.0	0.0	Embankment	Embankment	1.27	1	0.85	1.00	3.9	5.00	1.20	9.6	n.a.	0.97	#N/A	1.00	0.80	#N/A	#N/A
16.0	11.6	3	SP	4	Unsaturated	120	125	1917.6	1917.6	1617.6	1920.0	1920.0	1.05	1	0.95	1.00	3.6	0.00	1.00	3.6	n.a.	0.96	0.14	1.00	0.80	1.00	#N/A
21.0	6.6	12	SC	24	Unsaturated	120	125	2473.9	2473.9	1573.9	2520.0	2520.0	0.92	1	0.95	1.00	12.7	4.18	1.11	18.2	n.a.	0.95	0.14	1.00	0.74	0.98	#N/A
26.0	1.6	16	SC	24		120	125	3001.6	2783.2	1484.1	1519.5	1276.1	0.87	1	1	1.00	16.7	4.18	1.11	22.7	0.25	0.94	0.16	1.00	0.70	1.00	2.32
32.5	-4.9	32	SM	14		120	125	3573.9	3049.9	1343.9	2332.0	1683.0	0.83	1	1	1.00	32.0	2.20	1.04	35.5	2.00	0.91	0.18	1.00	0.60	1.00	3.00
38.0	-8.4	38	SP	4	Clay	120	125	4035.8	3193.4	1268.3	2769.5	1902.1	0.81	1	1	1.00	n.a.	0.00	1.00	n.a.	2.00	0.88	0.19	1.00	0.60	1.00	#N/A
41.0	-13.4	8	CL	94	Clay	120	125	4558.5	3404.1	1166.0	3394.5	2215.1	0.79	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.84	0.19	1.00	0.60	0.98	#N/A
46.0	-18.4	26	CL	94	Clay	120	125	5090.1	3623.7	1072.6	4019.5	2528.1	0.76	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.80	0.19	1.00	0.60	0.93	#N/A
51.0	-23.4	19	CL	94	Clay	120	125	5631.3	3852.9	988.8	4644.5	2841.1	0.74	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.76	0.18	1.00	0.60	0.89	#N/A
56.0	-28.4	20	CL	94	Clay	120	125	6181.6	4091.2	914.1	5269.5	3154.1	0.72	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.72	0.18	1.00	0.60	0.85	#N/A
61.0	-33.4	6	CL	94	Clay	120	125	6740.4	4338.0	847.9	5894.5	3467.1	0.70	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.68	0.17	1.00	0.60	0.82	#N/A

NOTE
[1] "e" is the distance from landside toe, positive downstream and negative going upstream
[2] Soil description may be used to estimate fines content where lab testing is not available
Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1999 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001
Surcharge from embankment calculation is presented in Poulos & Davis (1976) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space
[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used
[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted

Updated April 2013

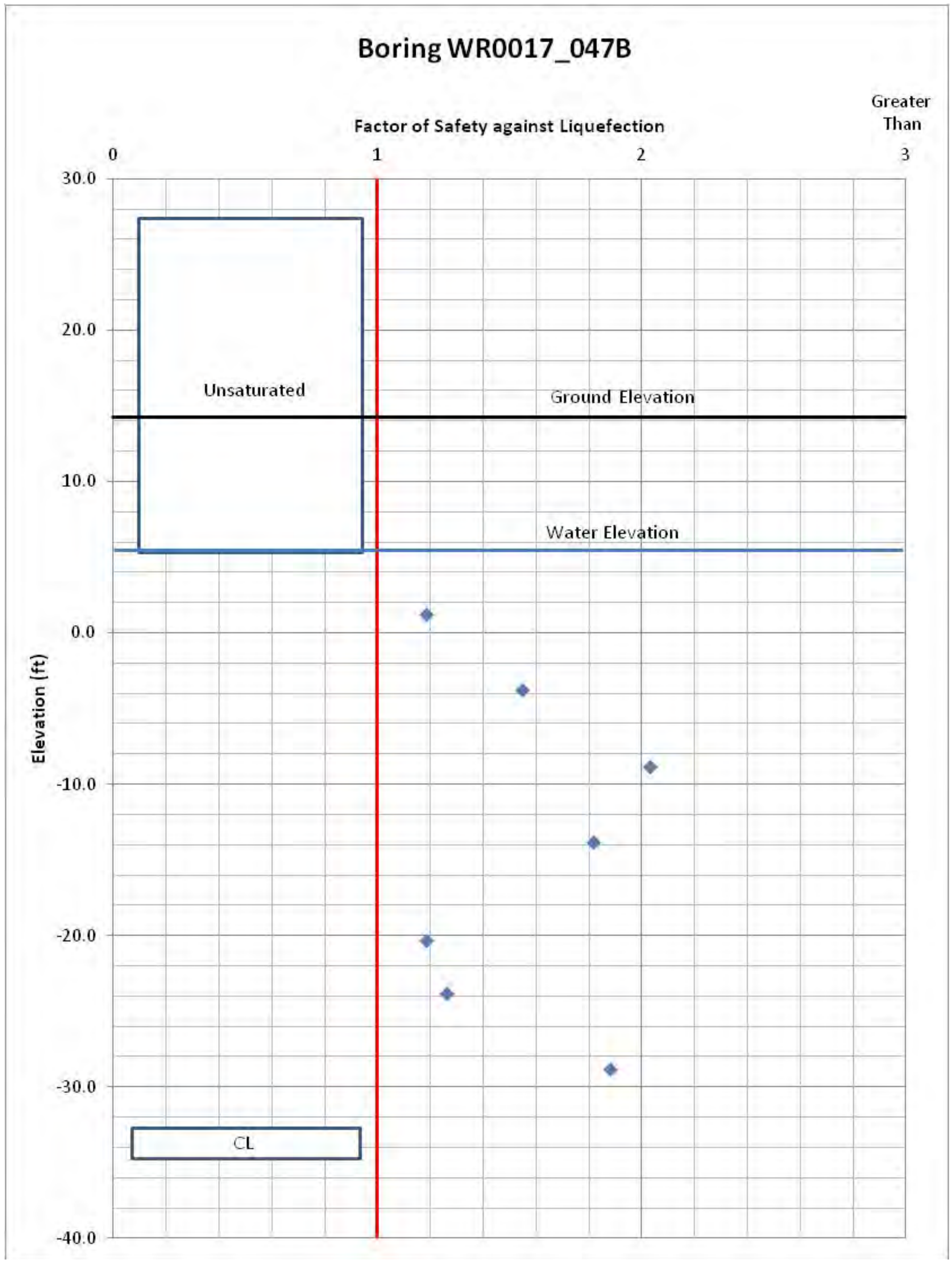


Fig. C-17. RD 17 North, Station 1377+73

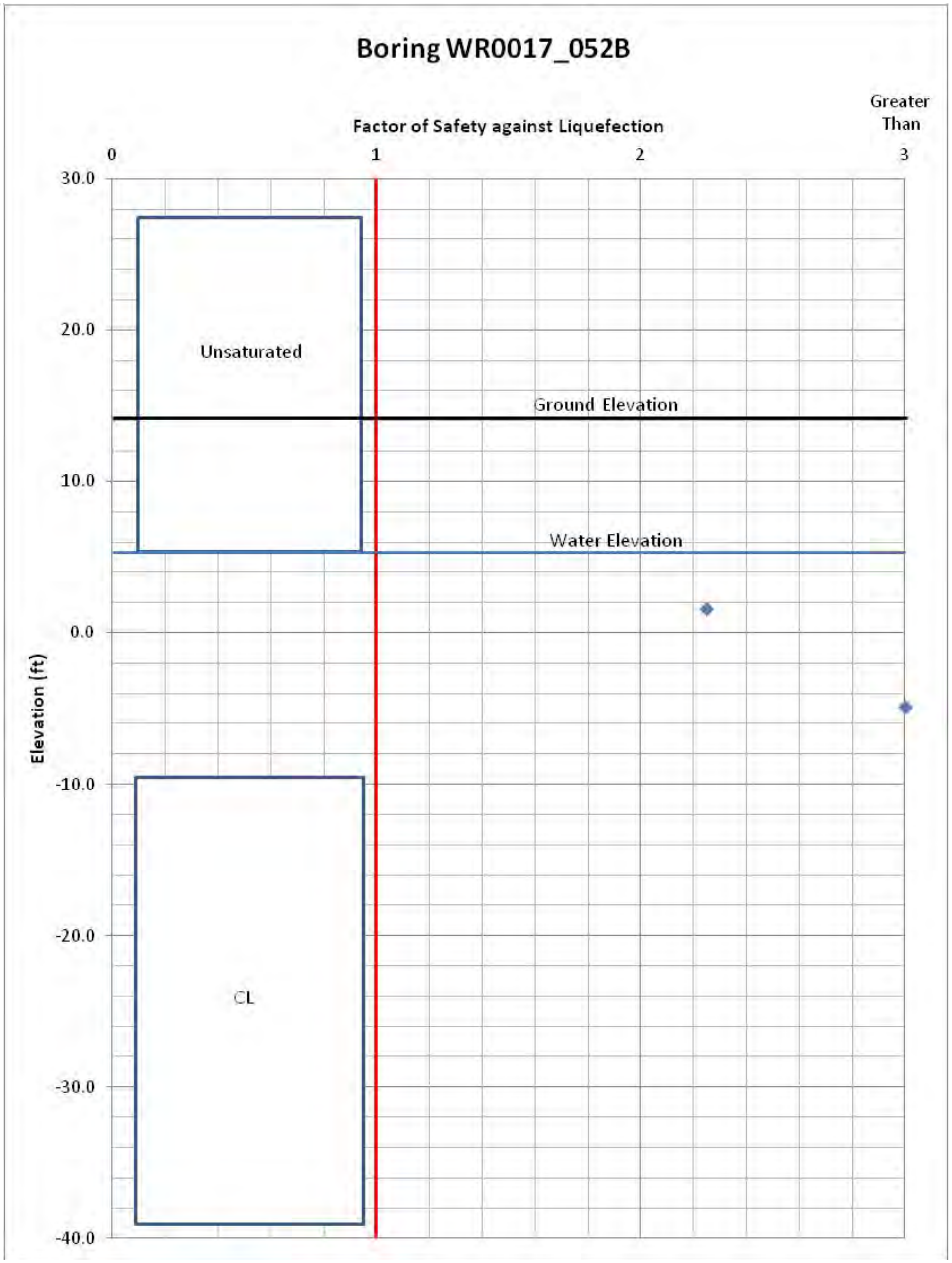


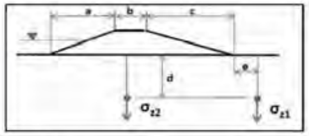
Fig. C-18. RD 17 North, Station 1416+93

Project: Lower San Joaquin
Study Area: RD 17
River Section: Sta. 1455+64
Boring Number: WR0017_057B

Prepared by: Vlad Perlea
Checked by:

Date: 5/6/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	27.7 ft	Rod Length Above GS (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	17.7 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.225
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	5.7 ft	Hammer Efficiency	72	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	7.0 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	24.0 ft
Crest Width, b (ft)	20.0 ft
Landside/Downstream Slope, c (ft)	21.0 ft
Dist. of Boring from Levee Toe[1] (ft)	-32.5 ft
Embankment Height, H (ft)	10.0 ft

Boring	WR0017_057B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
27.70 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
1.0	26.7	16	GC	12	Unsaturated	120	125	120.0	120.0	0.0	Embankment	Embankment	1.70	1	0.75	1.00	24.5	1.55	1.03	26.8	n.a.	1.00	#N/A	1.00	0.64	#N/A	#N/A
6.0	21.7	6	CL	94	Unsaturated	120	125	720.0	720.0	0.0	Embankment	Embankment	1.70	1	0.8	1.00	9.8	5.00	1.20	16.8	n.a.	0.99	#N/A	1.00	0.77	#N/A	#N/A
11.0	16.7	7	ML	50	Unsaturated	120	125	1319.9	1319.9	1199.9	1320.0	1320.0	1.27	1	0.85	1.00	9.0	5.00	1.20	15.8	n.a.	0.97	0.14	1.00	0.78	1.00	#N/A
16.0	11.7	5	SM	42	Unsaturated	120	125	1906.1	1906.1	1186.1	1920.0	1920.0	1.05	1	0.95	1.00	6.0	5.00	1.20	12.2	n.a.	0.96	0.14	1.00	0.80	1.00	#N/A
21.0	6.7	10	SP	4		120	125	2456.6	2456.6	1126.6	1321.5	1302.8	0.93	1	0.95	1.00	10.6	0.00	1.00	10.6	0.12	0.95	0.14	1.00	0.76	1.00	1.26
26.0	1.7	11	SC	28		120	125	3002.0	3752.4	1062.0	1946.5	1615.8	0.88	1	1	1.00	11.6	4.56	1.14	17.7	0.19	0.94	0.17	1.00	0.75	1.00	1.71
31.0	-3.3	15	SC	23		120	125	3543.6	2982.0	979.6	2571.5	1928.8	0.84	1	1	1.00	15.2	4.06	1.10	20.7	0.22	0.92	0.18	1.00	0.72	1.00	1.89
36.0	-8.3	15	SC	15		120	125	4086.7	3213.1	896.7	3196.5	2241.8	0.81	1	1	1.00	14.6	2.50	1.05	17.8	0.19	0.88	0.18	1.00	0.72	0.98	1.52
41.0	-13.3	40	SW-SM	10		120	125	4635.8	3450.2	820.8	3821.5	2554.8	0.78	1	1	1.00	37.6	0.87	1.02	39.3	2.00	0.84	0.18	1.00	0.60	0.93	3.00
46.0	-18.3	11	CL	94	Clay	120	125	5192.6	3695.0	752.6	4446.5	2867.9	0.76	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.80	0.18	1.00	0.60	0.89	#N/A
52.5	-24.8	18	CH	100	Clay	120	125	5928.0	4024.8	675.5	5259.0	3274.7	0.73	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.75	0.18	1.00	0.60	0.84	#N/A
56.0	-28.3	10	CH	100	Clay	120	125	6328.9	4207.3	638.9	5696.5	3493.8	0.71	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.73	0.17	1.00	0.60	0.82	#N/A
61.0	-33.3	19	CL	94	Clay	120	125	6907.0	4473.4	592.0	6321.5	3806.9	0.69	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.68	0.16	1.00	0.60	0.79	#N/A

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream

[2] Soil description may be used to estimate fines content where lab testing is not available

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for linter (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

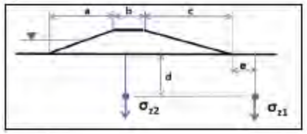
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: RD 404
Levee Station: 1003+04
Boring Number: WR0404_030B

Prepared by: Vlad Perlea
Checked by:

Date: 6/4/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	23.0 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	15.0 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Driling (ft)	-2.2 ft	Hammer Efficiency	85	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	0.0 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	8.8 ft
Crest Width, b (ft)	20.0 ft
Landside/Downstream Slope, c (ft)	80.0 ft
Dist. of Boring from Levee Toe[1] (ft)	-90.0 ft
Embankment Height, H (ft)	8.0 ft

Boring	WR0404_030B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
23.00 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
56.0	-33.0	23	ML/SM	51		120	125	6450.1	4528.2	536.1	5925.0	3865.8	0.68	1	1	1.00	22.3	5.00	1.20	31.7	2.00	0.72	0.14	1.00	0.66	0.81	3.00
61.0	-38.0	21	CL	94	PI = 8	120	125	7046.7	4812.8	507.7	6550.0	4178.8	0.66	1	1	1.00	19.7	5.00	1.20	28.7	0.40	0.68	0.14	1.00	0.68	0.80	3.00
66.0	-43.0	34	SM	14		120	125	7646.0	5100.1	462.0	7175.0	4491.8	0.64	1	1	1.00	31.0	2.20	1.04	34.5	2.00	0.64	0.13	1.00	0.60	0.74	3.00

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for linter (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

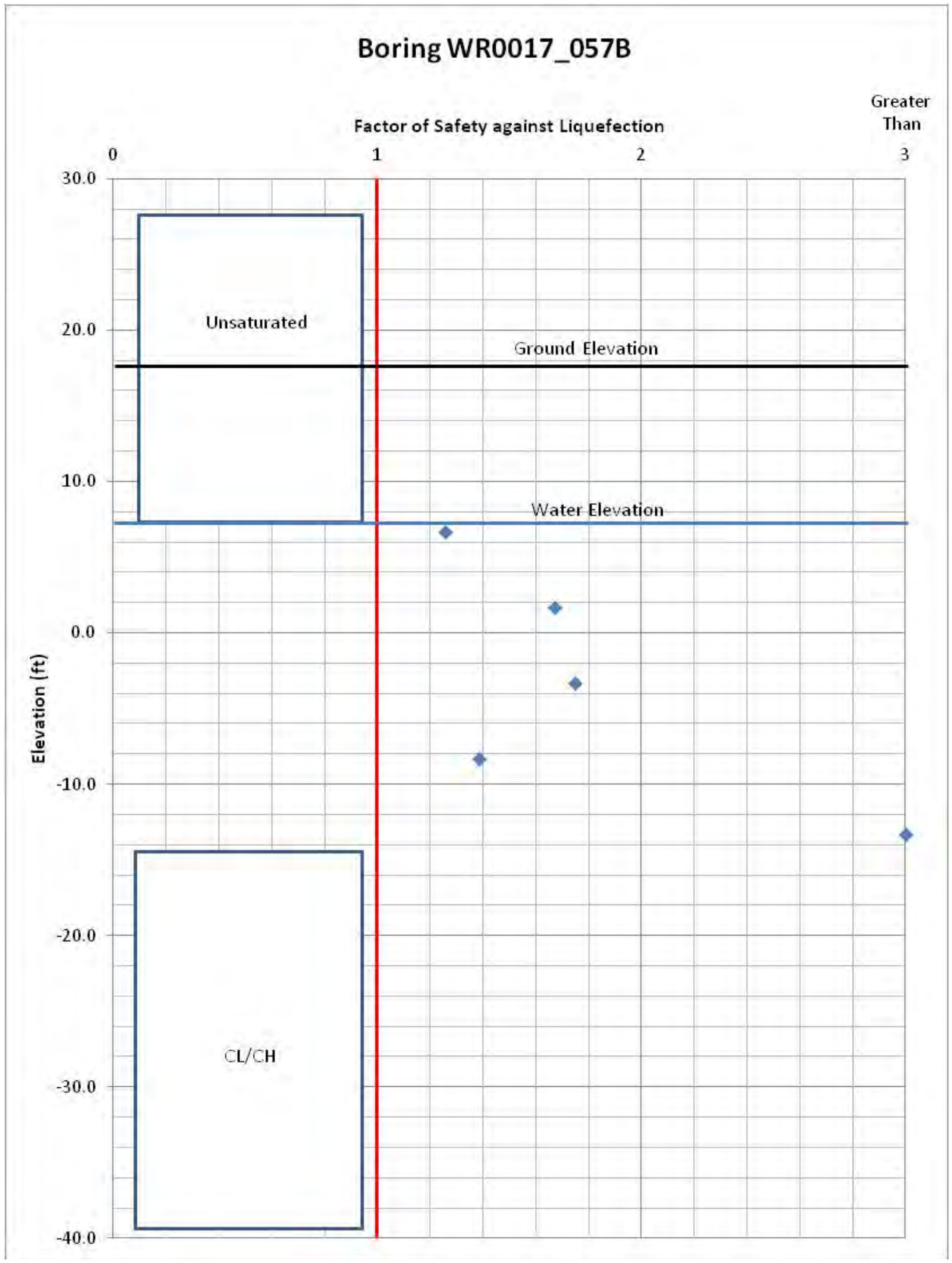


Fig. C-19. RD 17 North, Station 1455+64

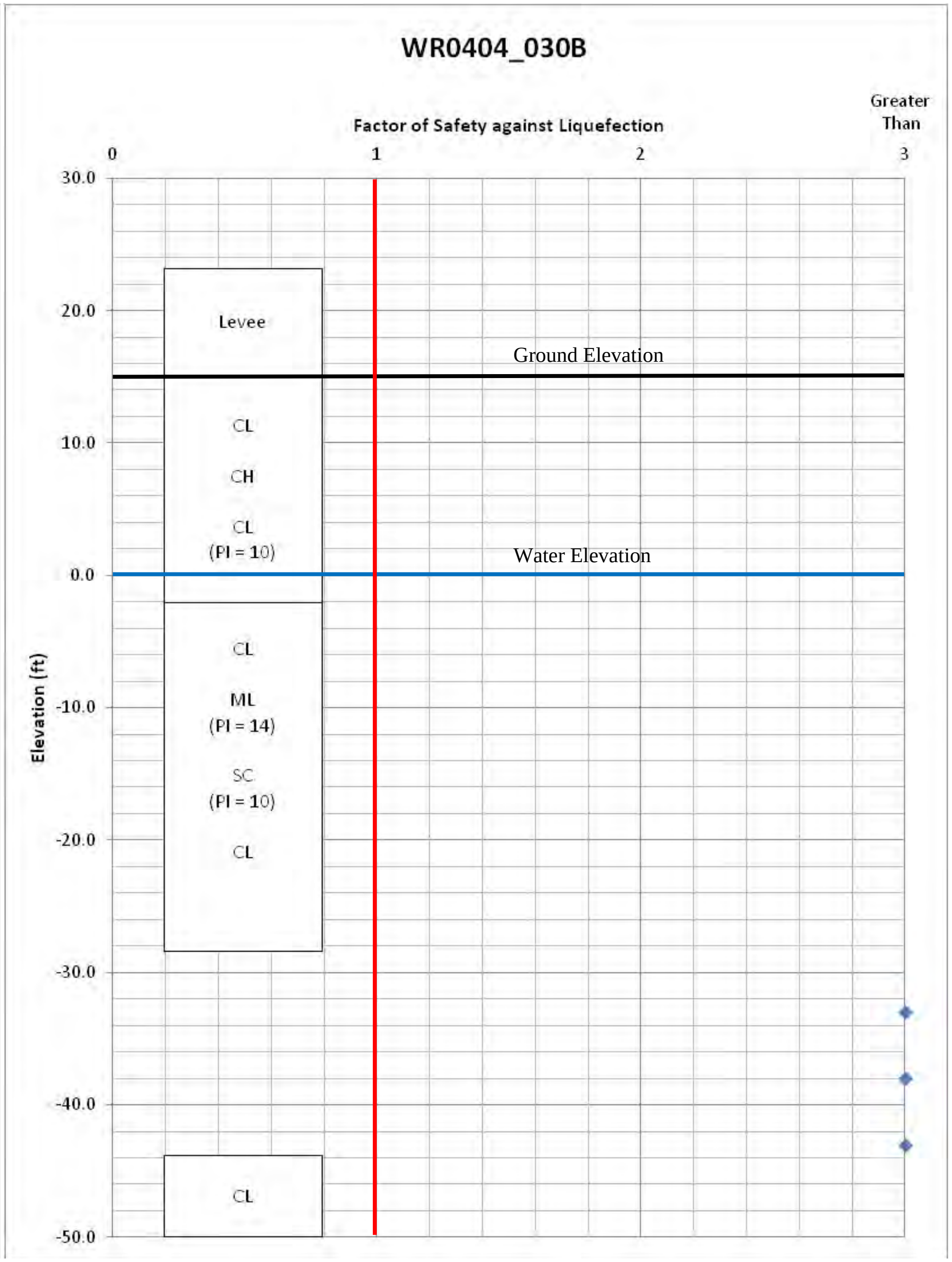


Fig. C-20. RD 404, Station 1003+04

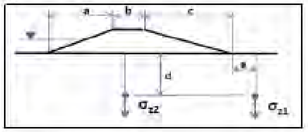
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: RD 404
Levee Station: 1201+00
Boring Number: WR0404_040B

Prepared by: Vlad Perlea
Checked by:

Date: 5/20/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	22.2 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	12.7 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.8		
Groundwater Elevation during Drilling (ft)	-3.3 ft	Hammer Efficiency	80	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	4.1 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	28.4 ft
Crest Width, b (ft)	28.0 ft
Landside/Downstream Slope, c (ft)	50.0 ft
Dist. of Boring from Levee Toe [1] (ft)	-64.0 ft
Embankment Height, H (ft)	9.5 ft

Boring WR0404_040B
Boring on the crest
SPT Ground Elevation Used in Analysis
22.20 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
15.5	6.7	17	ML	88	Unsaturated	120	125	1854.0	1854.0	1134.0	1860.0	1860.0	1.07	1	0.95	1.00	23.0	5.00	1.20	32.6	n.a.	0.96	0.13	1.00	0.65	1.00	#N/A
19.5	2.7	29	ML	50		120	125	2316.5	2316.5	1116.5	1207.0	1119.6	0.96	1	0.95	1.00	35.1	5.00	1.20	47.1	2.00	0.95	0.13	1.00	0.60	1.00	3.00
42.5	-20.3	20	SP	5		120	125	4938.4	3971.2	888.4	4082.0	2559.4	0.73	1	1	1.00	19.5	0.00	1.00	19.5	0.21	0.83	0.17	1.00	0.68	0.94	1.72
47.5	-25.3	17	SP	5		120	125	5511.6	4544.4	836.6	4707.0	2872.4	0.68	1	1	1.00	15.5	0.00	1.00	15.5	0.16	0.79	0.17	1.00	0.72	0.92	1.35
52.5	-30.3	55	SM	15		120	125	6087.9	5120.7	787.9	5332.0	3185.4	0.64	1	1	1.00	47.1	2.50	1.05	51.9	2.00	0.75	0.16	1.00	0.60	0.85	3.00
57.5	-35.3	68	SW-SM	8		120	125	6667.6	5700.5	742.8	5957.0	3498.4	0.61	1	1	1.00	55.2	0.30	1.01	56.2	2.00	0.71	0.16	1.00	0.60	0.82	3.00
72.5	-50.3	57	CL	94	Clay	120	125	8427.7	7460.5	627.7	7832.0	4437.4	0.53	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.58	0.13	1.00	0.60	0.74	#N/A

NOTE
[1] "e" is the distance from landside toe, positive downstream and negative going upstream.
[2] Soil description may be used to estimate fines content where lab testing is not available.
Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.
Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.
[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.
[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

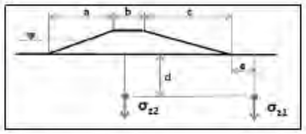
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: RD 404
Levee Station: 1175+01
Boring Number: WR0404_041B

Prepared by: Vlad Perlea
Checked by:

Date: 5/20/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	22.4 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	7.9 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.3 ft	Hammer Efficiency	80	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	4.1 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	20.3 ft
Crest Width, b (ft)	33.0 ft
Landside/Downstream Slope, c (ft)	52.2 ft
Dist. of Boring from Levee Toe [1] (ft)	-68.7 ft
Embankment Height, H (ft)	14.5 ft

Boring WR0404_041B
Boring on the crest
SPT Ground Elevation Used in Analysis
22.40 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
13.5	8.9	4	ML	63	Unsaturated	120	125	1620.0	1620.0	0.0	Embankment	Embankment	1.14	1	0.95	1.00	5.8	5.00	1.20	11.9	n.a.	0.97	#N/A	1.00	0.80	#N/A	#N/A
18.5	3.9	5	ML	54		120	125	2217.6	2217.6	1737.6	481.0	468.5	0.98	1	0.95	1.00	6.2	5.00	1.20	12.4	0.14	0.96	0.13	1.00	0.80	1.00	1.59
23.5	-1.1	4	SW-SM	8		120	125	2796.4	2796.4	1716.4	1106.0	781.5	0.87	1	0.95	1.00	4.4	0.30	1.01	4.8	0.07	0.95	0.17	1.00	0.80	1.00	0.61
58.0	-35.6	27	ML	55		120	125	6598.4	5756.0	1211.9	5418.5	2941.2	0.61	1	1	1.00	21.8	5.00	1.20	31.2	2.00	0.70	0.17	1.00	0.66	0.89	3.00
63.0	-40.6	57	SM	24		120	125	7155.1	6312.7	1143.6	6043.5	3254.2	0.58	1	1	1.00	44.0	4.18	1.11	52.9	2.00	0.66	0.16	1.00	0.60	0.84	3.00
73.0	-50.6	46	CH/CL	94	Clay	120	125	8284.2	7441.8	1022.7	7293.5	3880.2	0.53	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.58	0.14	1.00	0.60	0.78	#N/A

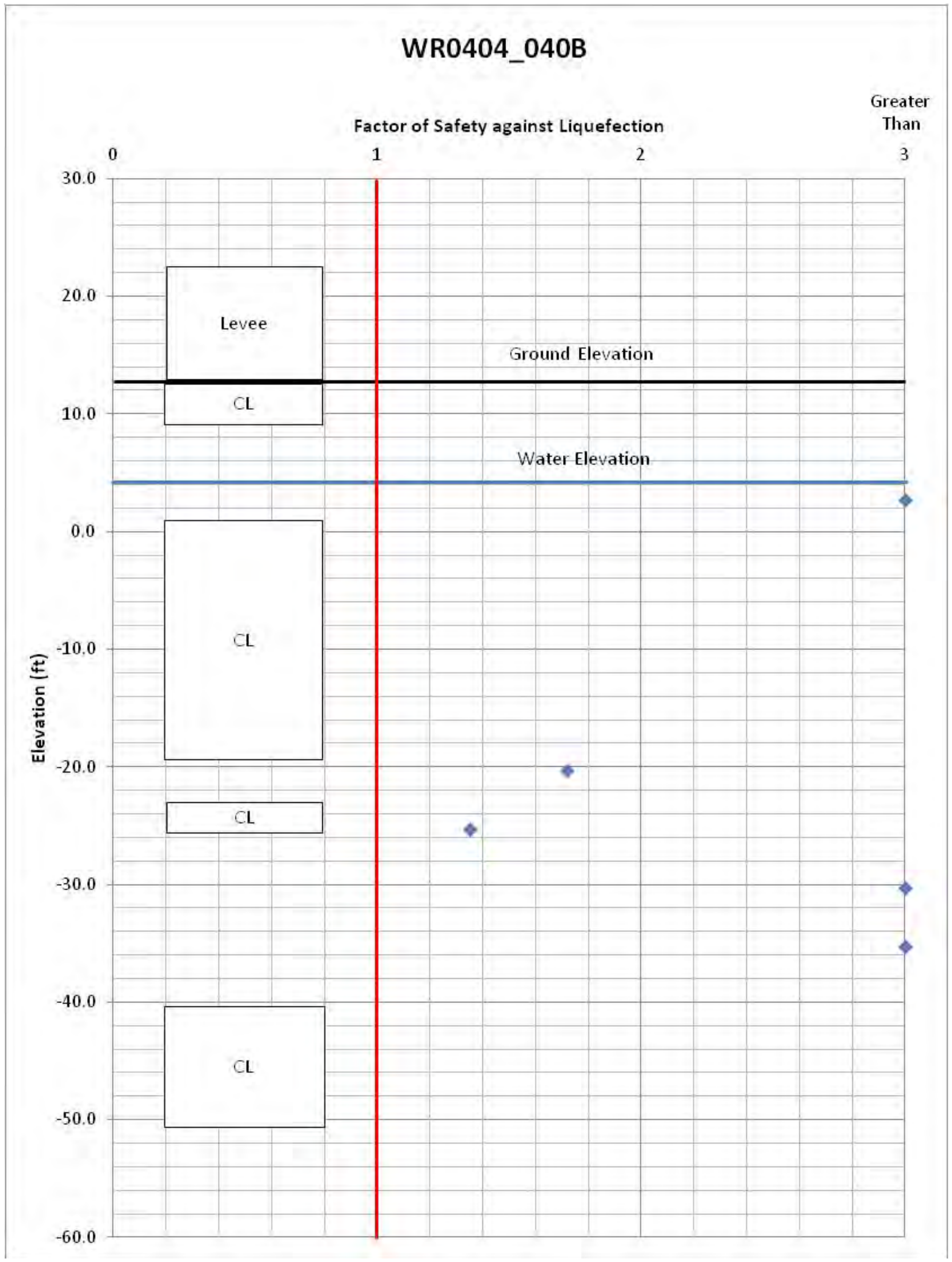


Fig. C-21. RD 404, Station 1201+00

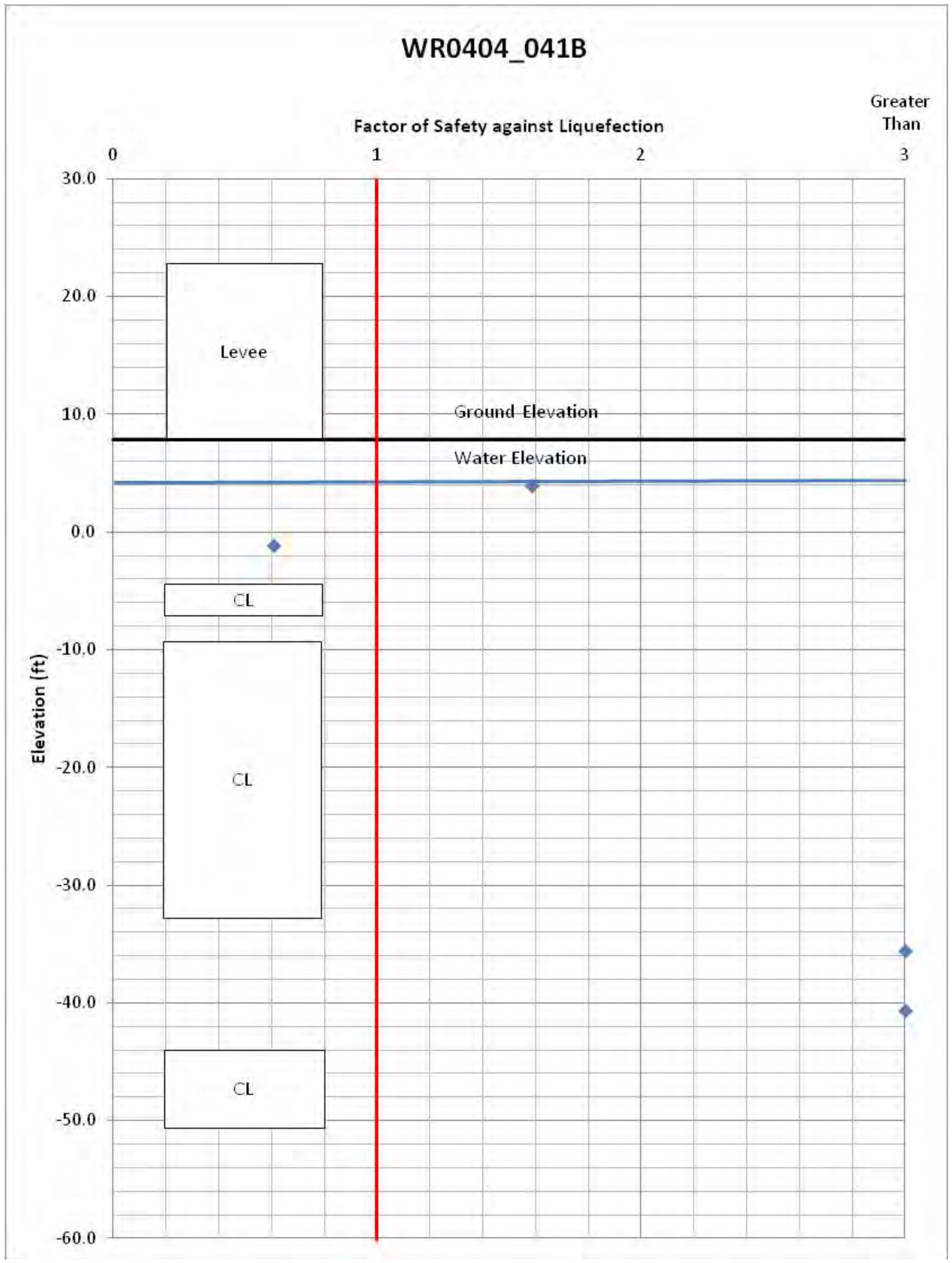


Fig. C-22. RD 404, Station 1175+01

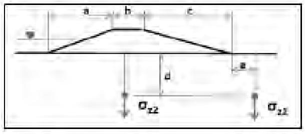
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: RD 404
Levee Station: 1139+55
Boring Number: WR0404_044B

Prepared by: Vlad Perlea
Checked by:

Date: 5/20/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	21.8 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	10.8 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.3 ft	Hammer Efficiency	80	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	0.0 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	28.4 ft
Crest Width, b (ft)	28.0 ft
Landside/Downstream Slope, c (ft)	50.0 ft
Dist. of Boring from Levee Toe[1] (ft)	-64.0 ft
Embankment Height, H (ft)	11.0 ft

Boring WR0404_044B
Boring on the crest
SPT Ground Elevation Used in Analysis
21.80 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
23.5	-1.7	20	ML	95		120	125	2773.1	2773.1	1273.1	1508.5	1402.4	0.87	1	0.95	1.00	22.1	5.00	1.20	31.6	2.00	0.95	0.13	1.00	0.66	1.00	3.00
47.5	-25.7	30	SM	14		120	125	5483.3	4023.1	985.3	4508.5	2904.8	0.73	1	1	1.00	29.0	2.20	1.04	32.4	2.00	0.79	0.16	1.00	0.60	0.88	3.00
52.5	-30.7	21	SM	13		120	125	6050.8	4584.4	928.8	5133.5	3217.8	0.68	1	1	1.00	19.0	1.89	1.04	21.6	0.24	0.75	0.15	1.00	0.68	0.88	2.01
67.5	-45.7	50	SP-SM	10		120	125	7777.4	6311.0	780.4	7008.5	4156.8	0.58	1	1	1.00	38.6	0.87	1.02	40.3	2.00	0.62	0.14	1.00	0.60	0.76	3.00
72.5	-50.7	45	SP	5		120	125	8360.7	6894.3	738.7	7633.5	4469.8	0.55	1	1	1.00	33.2	0.00	1.00	33.2	2.00	0.58	0.13	1.00	0.60	0.74	3.00

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

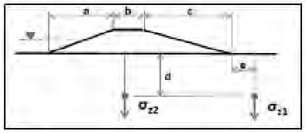
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: RD 404
Levee Station: 1112+49
Boring Number: WR0404_047B

Prepared by: Vlad Perlea
Checked by:

Date: 5/20/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	21.3 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	4.8 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.3 ft	Hammer Efficiency	80	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	0.0 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	24.6 ft
Crest Width, b (ft)	25.0 ft
Landside/Downstream Slope, c (ft)	47.6 ft
Dist. of Boring from Levee Toe[1] (ft)	-60.1 ft
Embankment Height, H (ft)	16.5 ft

Boring WR0404_047B
Boring on the crest
SPT Ground Elevation Used in Analysis
21.30 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
26.0	-4.7	12	ML	82		120	125	3084.6	2934.9	1932.6	1163.5	870.2	0.85	1	1	1.00	13.6	5.00	1.20	21.3	0.23	0.94	0.16	1.00	0.73	1.00	2.13
63.0	-41.7	13	ML	60		120	125	7015.2	5392.8	1238.2	5788.5	3186.4	0.63	1	1	1.00	10.9	5.00	1.20	18.0	0.19	0.66	0.16	1.00	0.76	0.91	1.68
68.0	-46.7	9	SM	15		120	125	7565.6	5943.2	1163.6	6413.5	3499.4	0.60	1	1	1.00	7.2	2.50	1.05	10.0	0.11	0.62	0.15	1.00	0.80	0.90	1.04
73.0	-51.7	24	CL	94	Clay	120	125	8122.7	6500.3	1095.7	7038.5	3812.4	0.57	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.58	0.14	1.00	0.60	0.79	#N/A

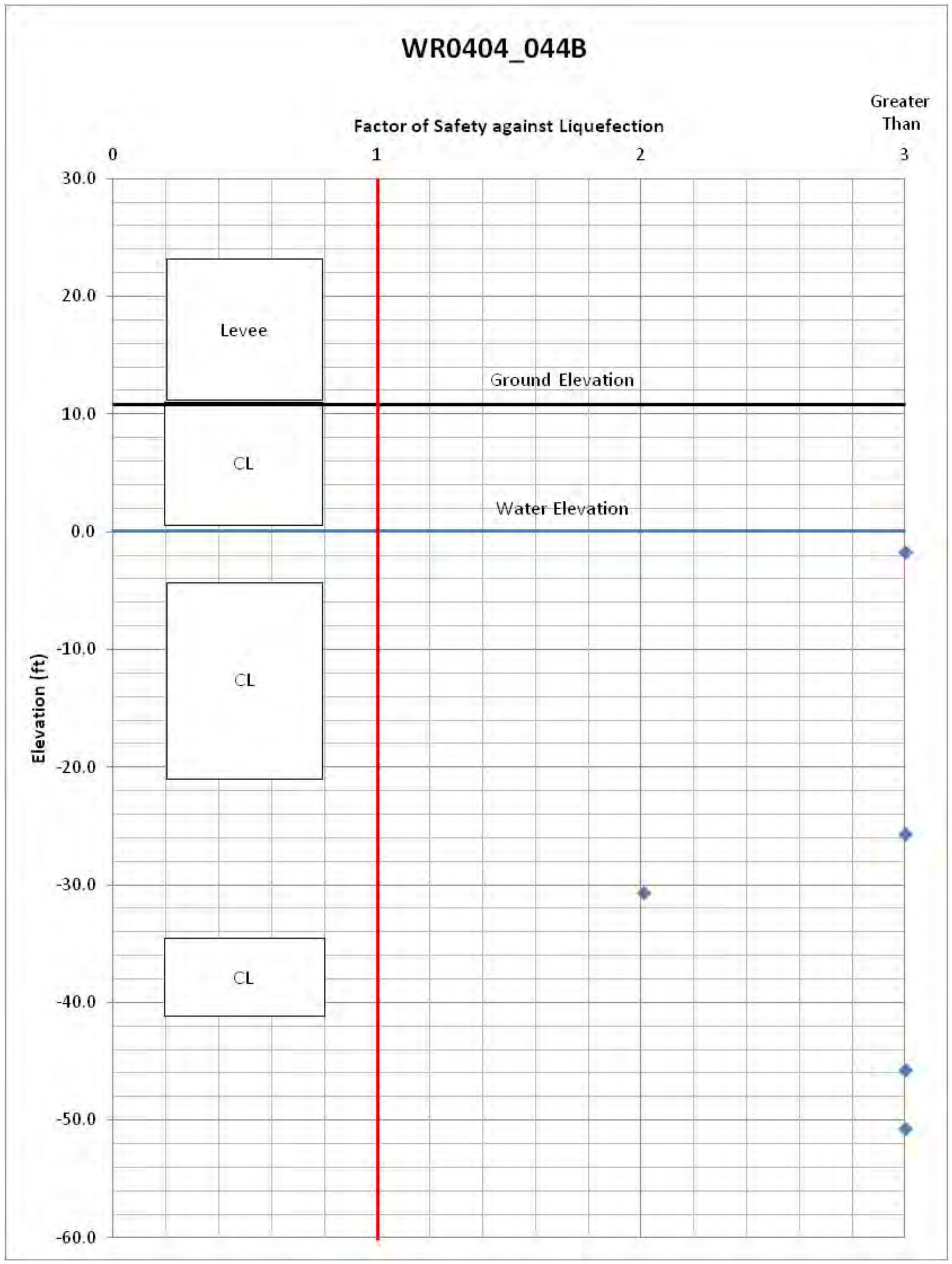


Fig. C-23. RD 404, Station 1139+55

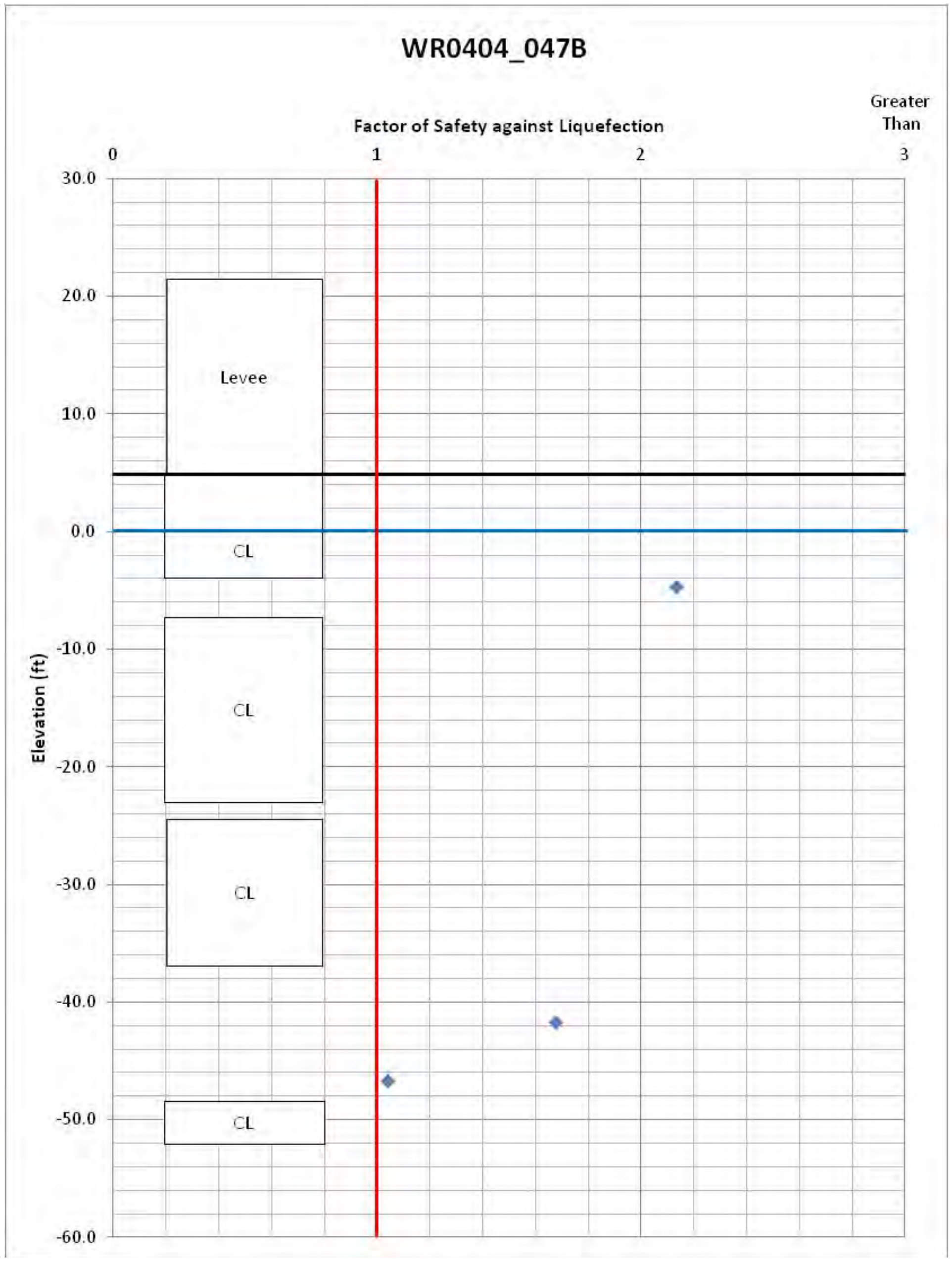


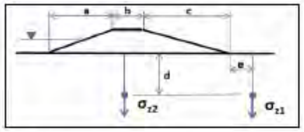
Fig. C-24. RD 404, Station 1112+49

LIQUERFACTION TRIGGERING ANALYSIS

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea	**Date:**	6/4/2013
Study Area:	RD 404	**Checked by:**		**Date:**	
Levee Station:	January-03				
Boring Number:	WR0404_048B				

Input Parameters

Embankment Crest Elevation (ft)	21.5 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	12.1 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	9.4 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.2 ft	Hammer Efficiency	80	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	0.0 ft			120.0 pcf	



Surcharge Information

Waterside/Upstream Slope, a (ft)	24.5 ft
Crest Width, b (ft)	25.0 ft
Landside/Downstream Slope, c (ft)	48.0 ft
Dist. of Boring from Levee Toe [1] (ft)	0.0 ft
Embankment Height, H (ft)	9.4 ft

Boring	**WR0404_048B**
Boring on waterside or landside field	
SPT Ground Elevation Used in Analysis	
12.10 ft	

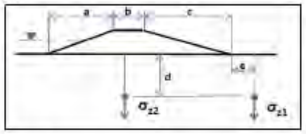
Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
38.0	-25.9	44	SM	25		120	125	4910.7	3431.8	232.2	4689.5	3073.3	0.79	1	1	1.00	46.1	4.29	1.12	55.7	2.00	0.86	0.17	1.00	0.60	0.86	3.00

LIQUERFACTION TRIGGERING ANALYSIS

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea	**Date:**	5/20/2013
Study Area:	RD 404	**Checked by:**		**Date:**	
Levee Station:	1087+77				
Boring Number:	WR0404_053B				

Input Parameters

Embankment Crest Elevation (ft)	22.4 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	2.2 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.2 ft	Hammer Efficiency	80	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	0.0 ft			120.0 pcf	



Surcharge Information

Waterside/Upstream Slope, a (ft)	59.9 ft
Crest Width, b (ft)	18.0 ft
Landside/Downstream Slope, c (ft)	36.2 ft
Dist. of Boring from Levee Toe [1] (ft)	-45.2 ft
Embankment Height, H (ft)	20.2 ft

Boring	**WR0404_053B**
Boring on the crest	
SPT Ground Elevation Used in Analysis	
22.40 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
28.5	-6.1	10	CL/ML	94		120	125	3386.4	3143.1	2370.9	1026.5	645.9	0.82	1	1	1.00	10.9	5.00	1.20	18.1	0.19	0.93	0.19	1.00	0.76	1.00	1.50
33.0	-10.6	11	CL/ML	94		120	125	3865.5	3341.3	2287.5	1589.0	927.6	0.80	1	1	1.00	11.7	5.00	1.20	19.0	0.20	0.91	0.20	1.00	0.75	1.00	1.51
58.0	-35.6	22	SM	13		120	125	6410.6	4632.2	1707.6	4714.0	2492.6	0.68	1	1	1.00	19.8	1.69	1.04	22.4	0.25	0.70	0.17	1.00	0.68	0.95	2.05
63.0	-40.6	49	SW-SM	11		120	125	6935.2	5156.8	1607.2	5339.0	2805.6	0.64	1	1	1.00	41.9	1.21	1.03	44.2	2.00	0.66	0.16	1.00	0.60	0.89	3.00
68.0	-45.6	37	SW-SM	11		120	125	7467.6	5699.2	1514.6	5964.0	3118.6	0.61	1	1	1.00	30.1	1.21	1.03	32.1	2.00	0.62	0.15	1.00	0.60	0.86	3.00
73.0	-50.6	54	SW-SM	11		120	125	8007.6	6229.2	1429.6	6589.0	3431.6	0.59	1	1	1.00	42.0	1.21	1.03	44.3	2.00	0.58	0.14	1.00	0.60	0.82	3.00

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. AL, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

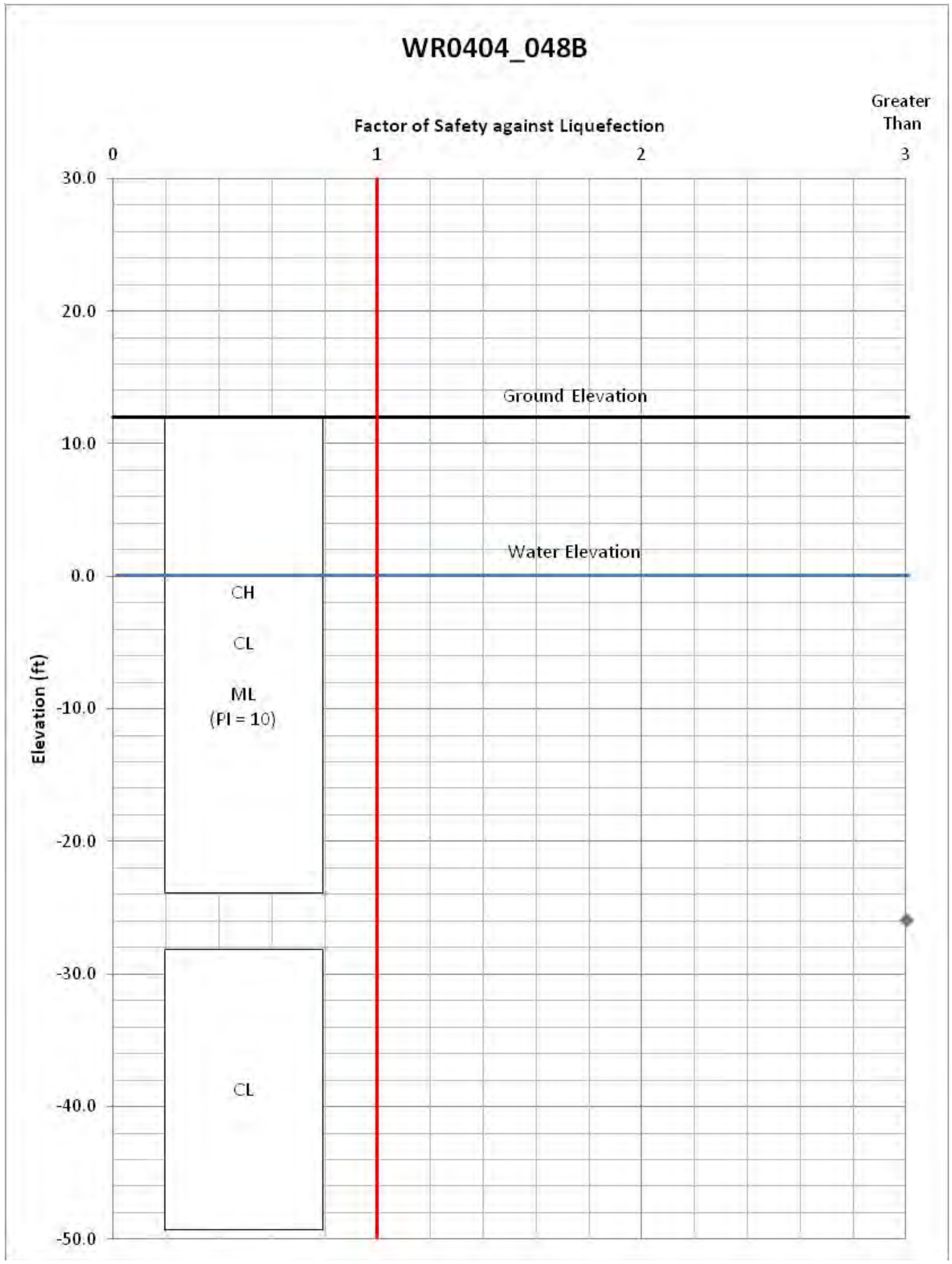


Fig. C-25. RD 404, Station 1108+07

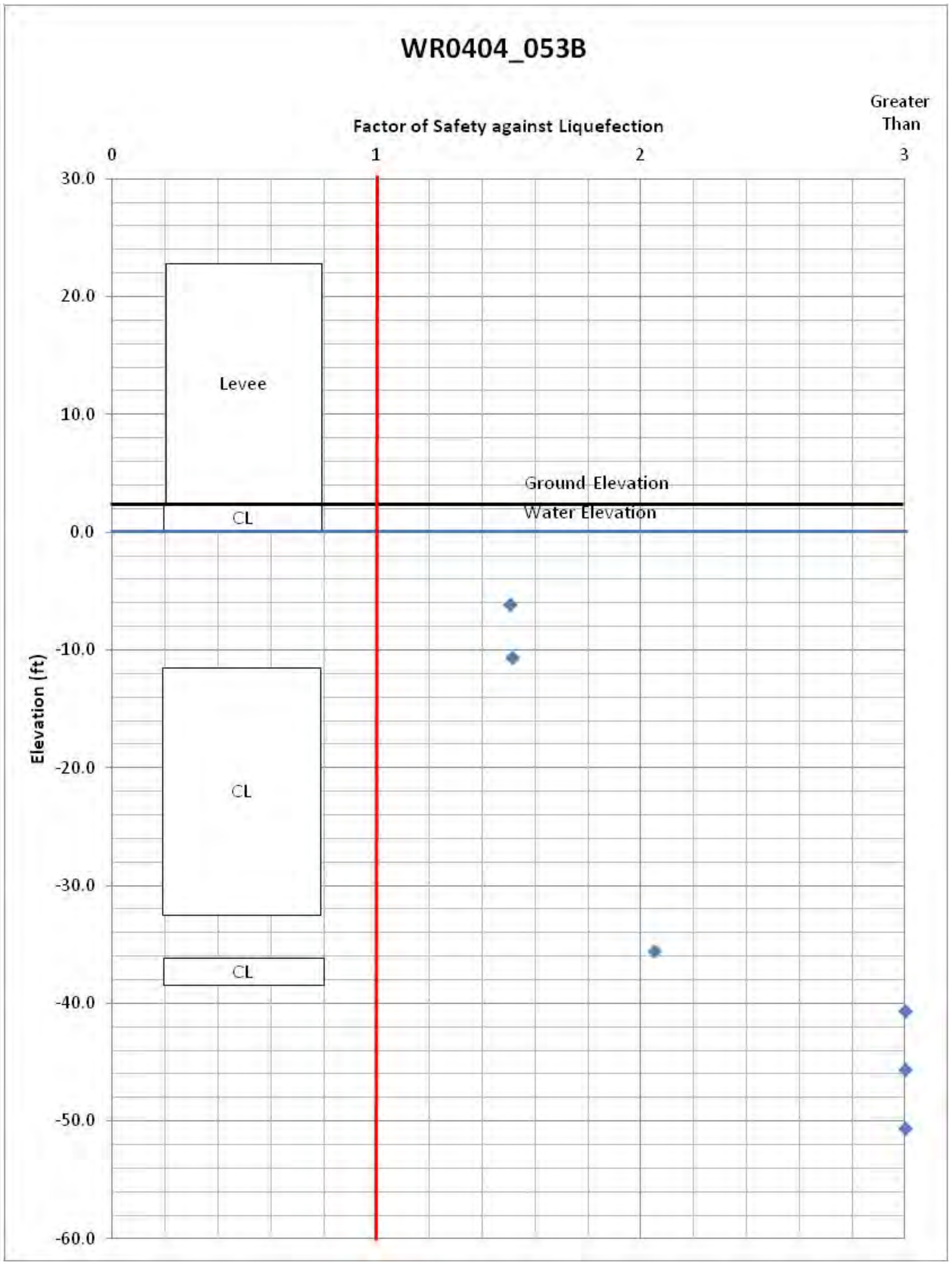
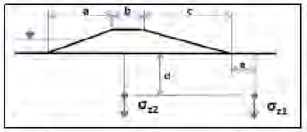


Fig. C-26. RD 404, Station 1087+77

LIQUERFACTION TRIGGERING ANALYSIS

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea	**Date:**	5/20/2013
Study Area:	RD 404	**Checked by:**		**Date:**	
Levee Station:	1070+28				
Boring Number:	WR0404_055B				

Input Parameters					
Embankment Crest Elevation (ft)	18.9 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	3.4 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.2 ft	Hammer Efficiency	80	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	0.0 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	17.5 ft
Crest Width, b (ft)	21.0 ft
Landside/Downstream Slope, c (ft)	155.3 ft
Dist. of Boring from Levee Toe [1] (ft)	-176.8 ft
Embankment Height, H (ft)	15.5 ft

Boring	WR0404_056B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
18.90 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
21.0	-2.1	4	SM/ML	49		120	125	2506.1	2506.1	1846.1	670.5	539.5	0.92	1	0.95	1.00	4.7	5.00	1.20	10.6	0.12	0.96	0.15	1.00	0.80	1.00	1.16
63.0	-44.1	21	SM	32		120	125	7152.1	5841.7	1242.6	5920.5	3168.7	0.60	1	1	1.00	16.9	4.83	1.17	24.6	0.28	0.66	0.16	1.00	0.70	0.89	2.35
69.0	-49.1	23	SP-SM	10		120	125	7728.1	6417.7	1193.6	6545.5	3481.7	0.57	1	1	1.00	17.6	0.87	1.02	18.9	0.20	0.62	0.15	1.00	0.70	0.86	1.71
75.0	-56.1	49	SP-SM	10		120	125	8541.9	7231.5	1132.4	7420.5	3919.9	0.54	1	1	1.00	35.3	0.87	1.02	37.0	2.00	0.56	0.14	1.00	0.60	0.78	3.00

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

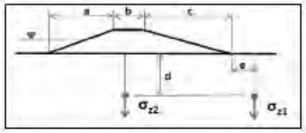
[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

LIQUERFACTION TRIGGERING ANALYSIS

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea	**Date:**	5/20/2013
Study Area:	RD 404	**Checked by:**		**Date:**	
Levee Station:	1042+70				
Boring Number:	WR0404_059B				

Input Parameters					
Embankment Crest Elevation (ft)	19.7 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	3.5 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.2 ft	Hammer Efficiency	80	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	0.0 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	21.5 ft
Crest Width, b (ft)	35.0 ft
Landside/Downstream Slope, c (ft)	16.6 ft
Dist. of Boring from Levee Toe [1] (ft)	-34.6 ft
Embankment Height, H (ft)	16.2 ft

Boring	WR0404_069B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
19.70 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
27.5	-7.8	17	ML	88		120	125	3270.6	2921.2	1886.6	1395.0	908.3	0.85	1	1	1.00	19.3	5.00	1.20	28.1	0.37	0.94	0.19	1.00	0.68	1.00	3.00
47.5	-27.8	62	SM	15		120	125	5364.6	3767.1	1480.6	3895.0	2160.3	0.75	1	1	1.00	62.0	2.50	1.05	67.4	2.00	0.79	0.18	1.00	0.60	0.99	3.00
52.0	-32.3	26	ML/CL	80		120	125	5831.9	4115.9	1385.4	4457.5	2442.0	0.72	1	1	1.00	24.9	5.00	1.20	34.8	2.00	0.75	0.18	1.00	0.64	0.95	3.00
57.5	-37.8	17	ML/CL	95		120	125	6411.8	4695.8	1277.8	5145.0	2786.3	0.67	1	1	1.00	15.2	5.00	1.20	23.3	0.26	0.71	0.17	1.00	0.72	0.93	2.14
67.5	-47.8	14	CL/ML	95		120	125	7493.6	5777.6	1109.6	6395.0	3412.3	0.61	1	1	1.00	11.3	5.00	1.20	18.6	0.20	0.62	0.15	1.00	0.76	0.89	1.74
71.0	-51.3	29	CL/ML	85		120	125	7880.1	6164.1	1058.6	6832.5	3631.4	0.59	1	1	1.00	22.7	5.00	1.20	32.2	2.00	0.60	0.15	1.00	0.66	0.83	3.00

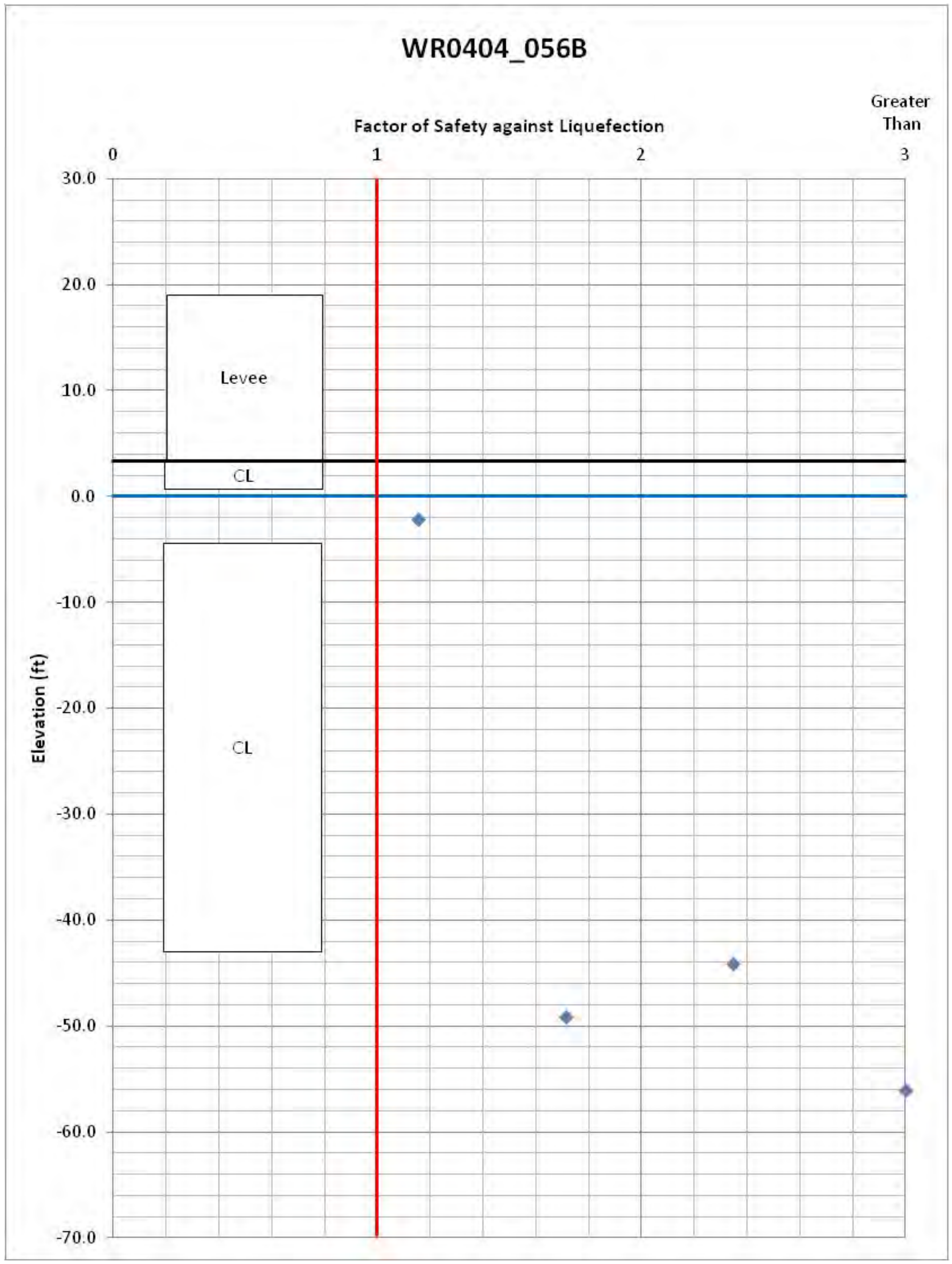


Fig. C-27. RD 404, Station 1070+28

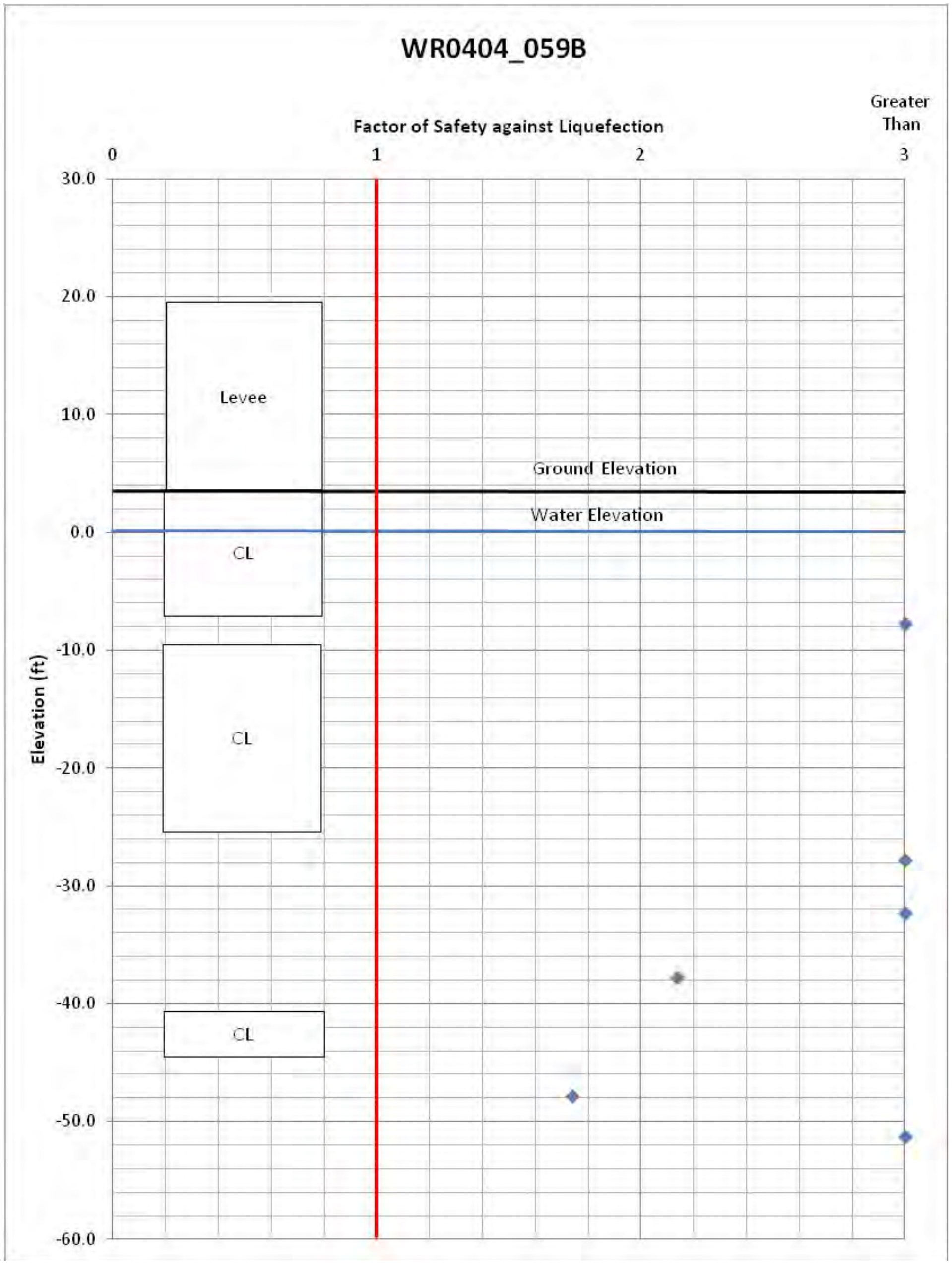


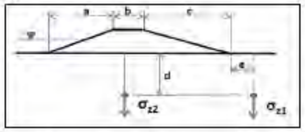
Fig. C-28. RD 404, Station 1042+70

LIQUEFACTION TRIGGERING ANALYSIS

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea	**Date:**	5/20/2013
Study Area:	RD 404	**Checked by:**		**Date:**	
Levee Station:	1028+00				
Boring Number:	WR0404_060B				

Input Parameters

Embankment Crest Elevation (ft)	21.9 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	10.9 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.2 ft	Hammer Efficiency	80	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	0.0 ft			120.0 pcf	



Surcharge Information

Waterside/Upstream Slope, a (ft)	12.1 ft
Crest Width, b (ft)	29.0 ft
Landside/Downstream Slope, c (ft)	132.0 ft
Dist. of Boring from Levee Toe [1] (ft)	-146.5 ft
Embankment Height, H (ft)	11.0 ft

Boring	WR0404_060B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
21.90 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
53.0	-31.1	25	SM	21		120	125	6125.6	4322.2	941.1	5195.5	3254.9	0.70	1	1	1.00	23.3	3.76	1.09	29.1	0.42	0.74	0.15	1.00	0.65	0.86	3.00
58.0	-36.1	32	SM	21		120	125	6706.5	4593.1	899.0	5830.5	3567.9	0.68	1	1	1.00	29.0	3.76	1.09	35.2	2.00	0.70	0.15	1.00	0.60	0.81	3.00
63.0	-41.1	50	SM	21		120	125	7295.2	4867.9	860.7	6445.5	3880.9	0.66	1	1	1.00	44.0	3.76	1.09	51.5	2.00	0.66	0.14	1.00	0.60	0.78	3.00
68.0	-46.1	46	SM	21		120	125	7885.3	5146.0	825.8	7070.5	4193.9	0.64	1	1	1.00	39.3	3.76	1.09	46.5	2.00	0.62	0.14	1.00	0.60	0.76	3.00
73.0	-51.1	18	CL/CH	94	Clay	120	125	8478.4	5427.0	793.9	7695.5	4506.9	0.62	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.58	0.13	1.00	0.60	0.74	#N/A

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

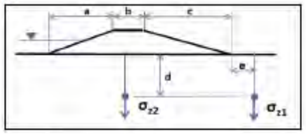
Updated April 2013

LIQUEFACTION TRIGGERING ANALYSIS

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea	**Date:**	5/21/2013
Study Area:	Calaveras River and Stockton Diverting Canal	**Checked by:**		**Date:**	
Levee Station:	6505+30				
Boring Number:	WR1614_017B				

Input Parameters

Embankment Crest Elevation (ft)	16.0 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	3.4 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	1.0 ft	Hammer Efficiency	85	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	3.4 ft			120.0 pcf	



Surcharge Information

Waterside/Upstream Slope, a (ft)	92.0 ft
Crest Width, b (ft)	208.0 ft
Landside/Downstream Slope, c (ft)	56.7 ft
Dist. of Boring from Levee Toe [1] (ft)	-158.6 ft
Embankment Height, H (ft)	12.6 ft

Boring	WR1614_017B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
16.00 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
50.5	-34.5	0	SM, MH	90		120	125	6224.2	4009.0	1498.7	4737.5	2372.5	0.73	1	1	1.00	0.0	5.00	1.20	5.0	0.07	0.76	0.20	1.00	0.80	0.98	0.33
58.0	-42.0	21	SM	15		120	125	7153.1	4469.9	1490.1	5675.0	2842.0	0.69	1	1	1.00	20.5	2.50	1.05	24.0	0.27	0.70	0.18	1.00	0.67	0.91	2.04
60.5	-44.5	27	SM	16		120	125	7462.2	4623.0	1486.7	5987.5	2998.5	0.68	1	1	1.00	25.9	2.77	1.05	30.0	2.00	0.68	0.18	1.00	0.63	0.88	3.00

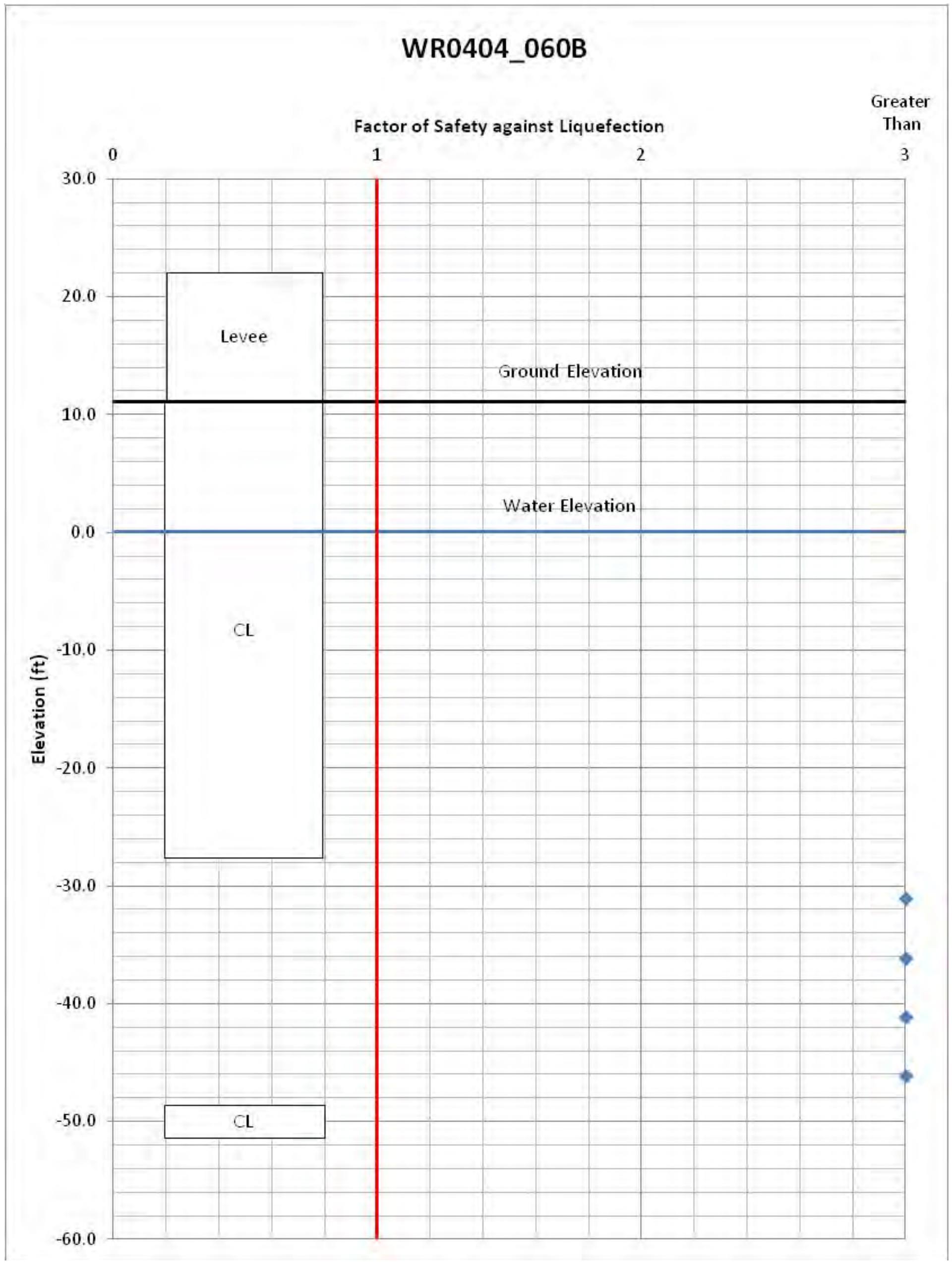


Fig. C-29. RD 404, Station 1028+00

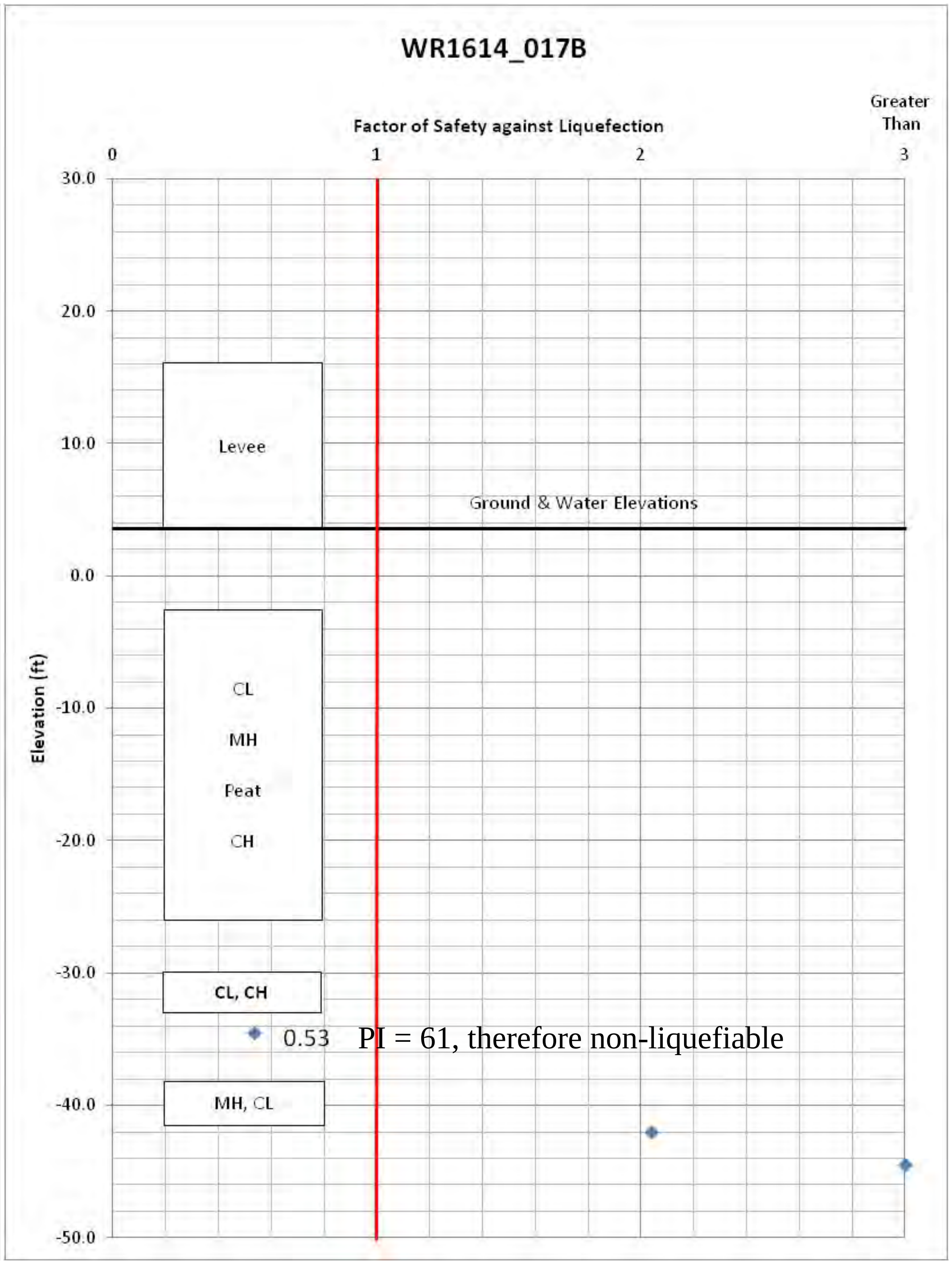


Fig. C-30. Calaveras River, Station 6505+30

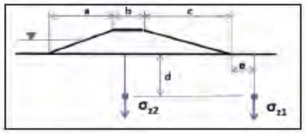
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Calaveras River and Stockton Diverting Canal
Levee Station: 3072+94
Boring Number: WR2074_016B

Prepared by: Vlad Perlea
Checked by:

Date: 5/21/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	16.4 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	3.0 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	13.4 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-1.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	-1.0 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	41.6 ft
Crest Width, b (ft)	46.0 ft
Landside/Downstream Slope, c (ft)	31.9 ft
Dist. of Boring from Levee Toe [1] (ft)	21.7 ft
Embankment Height, H (ft)	13.4 ft

Boring WR2074_016B
Boring on waterside or landside field
SPT Ground Elevation Used in Analysis
3.00 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
14.5	-11.5	7	ML	58		120	125	1814.9	1159.7	22.4	1792.5	1137.3	1.35	1	0.95	1.00	11.5	5.00	1.20	18.8	0.20	0.97	0.20	1.00	0.75	1.00	1.53
34.0	-31.0	21	SM	15		120	125	4365.0	3460.2	135.0	4230.0	2358.0	0.78	1	1	1.00	21.1	2.50	1.05	24.6	0.28	0.90	0.21	1.00	0.67	0.96	1.96
39.0	-36.0	27	SM	16		120	125	5021.7	4116.9	166.7	4855.0	2671.0	0.72	1	1	1.00	24.8	2.77	1.05	28.9	0.41	0.86	0.20	1.00	0.64	0.92	2.78
50.0	-47.0	21	CL	94	Clay	120	125	6459.3	5554.9	229.3	6230.0	3359.6	0.62	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.77	0.18	1.00	0.60	0.83	#N/A

NOTE
[1] "e" is the distance from landside toe, positive downstream and negative going upstream.
[2] Soil description may be used to estimate fines content where lab testing is not available.
Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.
Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.
[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.
[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

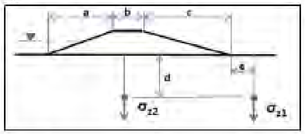
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Calaveras River and Stockton Diverting Canal
Levee Station: 3087+75
Boring Number: WCNBCR_010B

Prepared by: Vlad Perlea
Checked by:

Date: 5/21/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	14.7 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	1.6 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	13.1 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-1.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	-1.0 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	19.7 ft
Crest Width, b (ft)	27.0 ft
Landside/Downstream Slope, c (ft)	53.2 ft
Dist. of Boring from Levee Toe [1] (ft)	198.7 ft
Embankment Height, H (ft)	13.1 ft

Boring WCNBCR_010B
Boring on waterside or landside field
SPT Ground Elevation Used in Analysis
1.60 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
19.5	-17.9	13	SM	25		120	125	2424.6	1370.1	0.1	2424.5	1369.9	1.24	1	0.95	1.00	19.7	4.29	1.12	26.3	0.32	0.95	0.22	1.00	0.68	1.00	2.18
24.5	-22.9	19	SP-SM	8		120	125	3049.7	1832.9	0.2	3049.5	1682.9	1.07	1	0.95	1.00	24.9	0.30	1.01	25.5	0.30	0.94	0.22	1.00	0.64	1.00	2.04

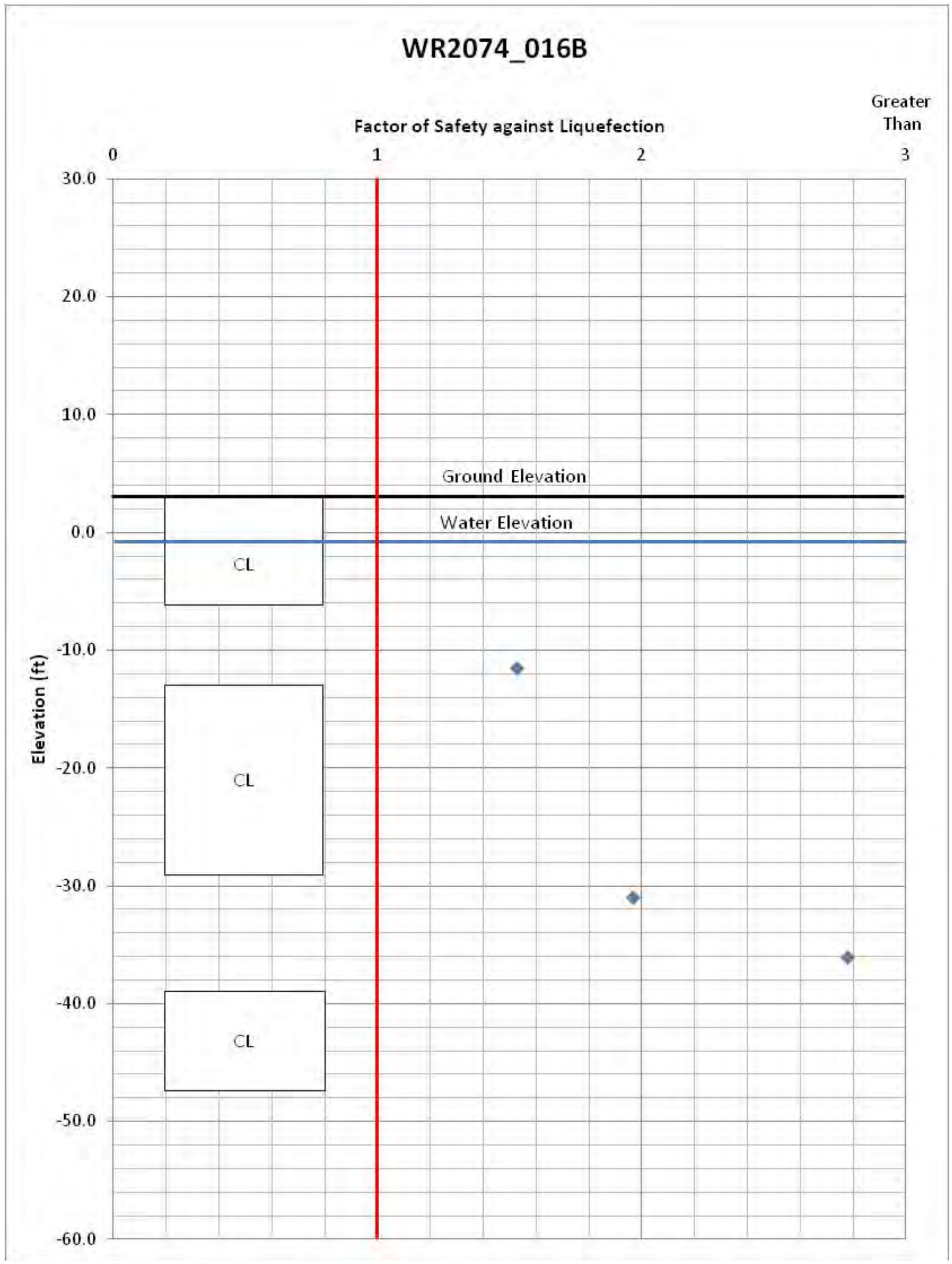


Fig. C-31. Calaveras River, Station 3072+94

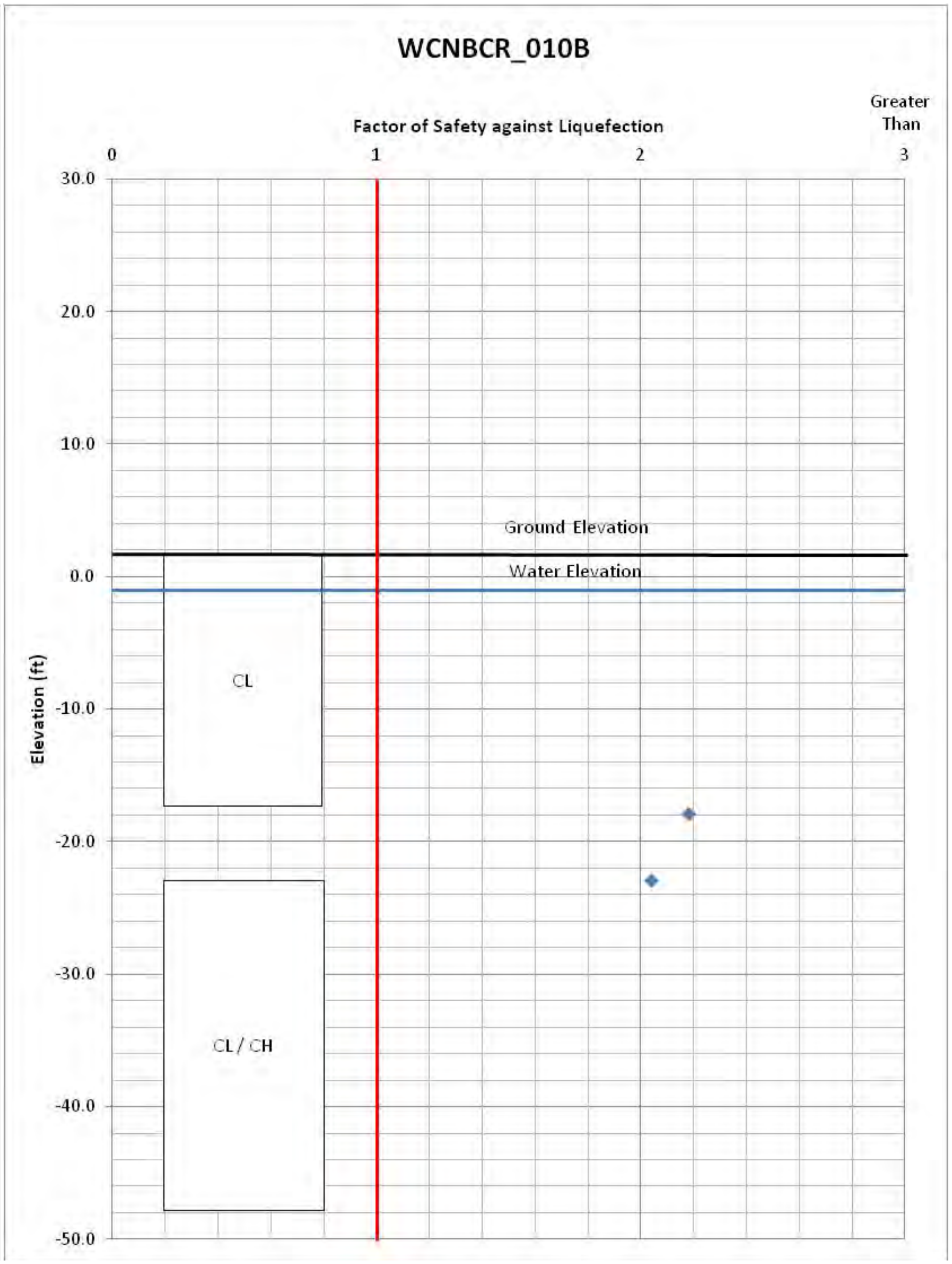


Fig. C-32. Calaveras River, Station 3087+75

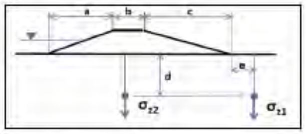
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Calaveras River and Stockton Diverting Canal
Levee Station: 6565+02
Boring Number: WR1614_018B

Prepared by: Vlad Perlea
Checked by:

Date: 5/21/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	19.5 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	1.4 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	1.0 ft	Hammer Efficiency	85	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	1.4 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	29.0 ft
Crest Width, b (ft)	75.0 ft
Landside/Downstream Slope, c (ft)	34.5 ft
Dist. of Boring from Levee Toe [1] (ft)	-88.9 ft
Embankment Height, H (ft)	18.1 ft

Boring	WR1614_018B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
19.50 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
41.5	-22.0	2	SP-SM	10		120	125	4975.0	3539.8	2052.0	2925.0	1464.8	0.77	1	1	1.00	2.2	0.87	1.02	3.1	0.06	0.84	0.22	1.00	0.80	1.00	0.41

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

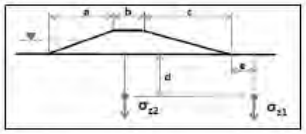
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Calaveras River and Stockton Diverting Canal
Levee Station: 3130+53
Boring Number: WCNBCR_011B

Prepared by: Vlad Perlea
Checked by:

Date: 5/21/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	19.0 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	7.3 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	11.7 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-1.5 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	-1.0 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	32.8 ft
Crest Width, b (ft)	18.0 ft
Landside/Downstream Slope, c (ft)	19.7 ft
Dist. of Boring from Levee Toe [1] (ft)	193.3 ft
Embankment Height, H (ft)	11.7 ft

Boring	WCNBCR_011B
Boring on waterside or landside field	
SPT Ground Elevation Used in Analysis	
7.30 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
6.0	1.3	12	ML	90	Unsaturated	120	125	720.0	720.0	0.0	720.0	720.0	1.70	1	0.8	1.00	20.9	5.00	1.20	30.1	n.a.	0.99	0.13	1.00	0.67	1.00	#N/A
10.0	-2.7	28	ML	90		120	125	1206.0	1131.1	0.0	1208.5	1102.4	1.37	1	0.85	1.00	41.8	5.00	1.20	55.1	2.00	0.98	0.14	1.00	0.60	1.00	3.00
15.0	-7.7	34	SM	45		120	125	1831.1	1456.7	0.1	1833.5	1415.4	1.21	1	0.95	1.00	50.0	5.00	1.20	65.0	2.00	0.97	0.16	1.00	0.60	1.00	3.00
20.0	-12.7	16	SM	20		120	125	2456.1	2081.7	0.1	2458.5	1728.4	1.01	1	0.95	1.00	19.7	3.61	1.08	24.8	0.29	0.95	0.18	1.00	0.68	1.00	2.46
25.0	-17.7	17	SM	20		120	125	3081.3	2706.9	0.3	3083.5	2041.4	0.88	1	0.95	1.00	18.3	3.61	1.08	23.4	0.26	0.94	0.18	1.00	0.69	1.00	2.14
30.0	-22.7	21	SM	20		120	125	3706.4	3332.0	0.4	3708.5	2354.4	0.80	1	1	1.00	21.5	3.61	1.08	26.8	0.33	0.93	0.19	1.00	0.67	0.96	2.53
45.0	-37.7	17	CL	94	Clay	120	125	5582.4	5208.0	1.4	5583.5	3293.4	0.64	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.81	0.18	1.00	0.60	0.84	#N/A

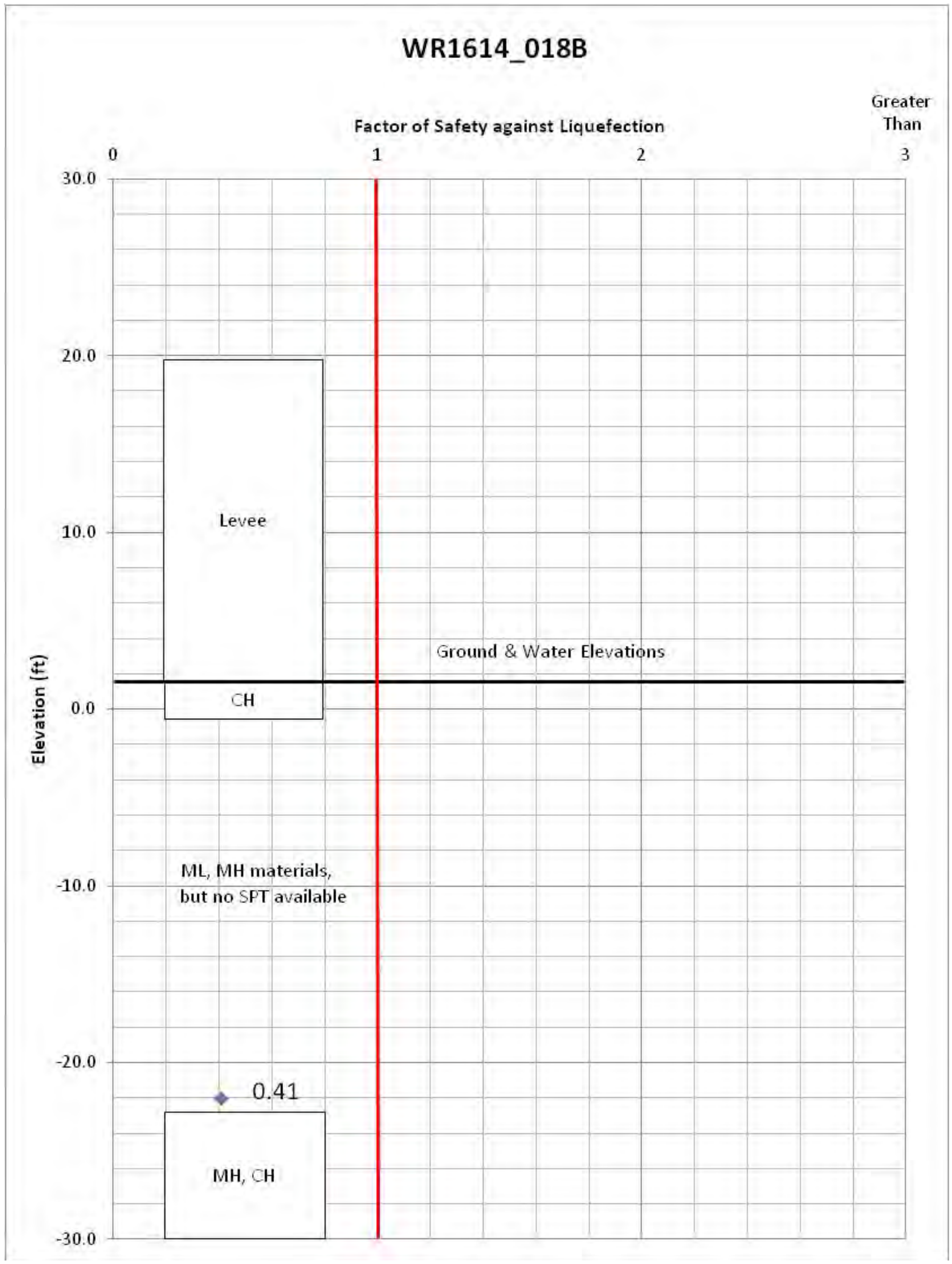


Fig. C-33. Calaveras River, Station 6565+02

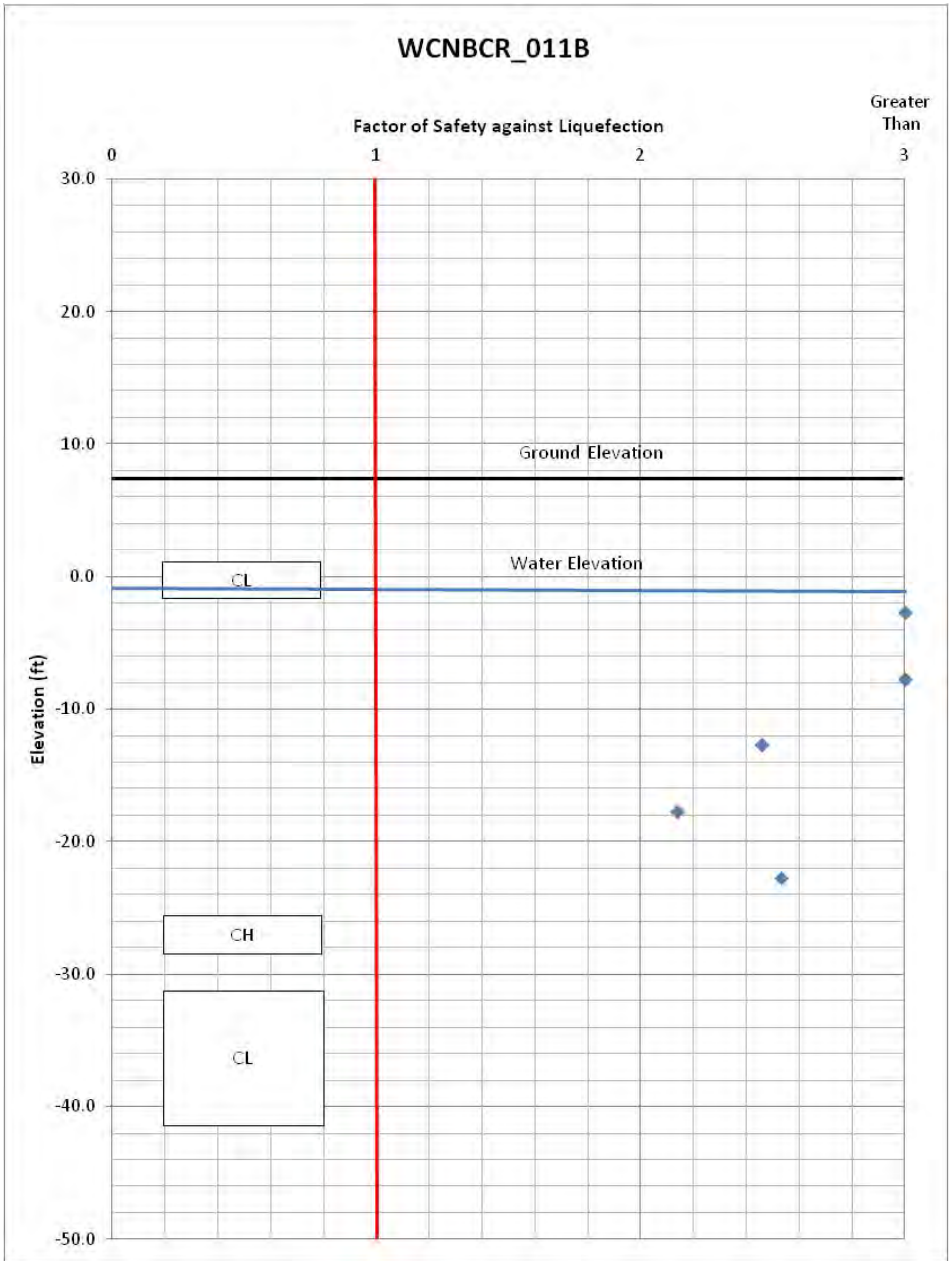


Fig. C-34. Calaveras River, Station 3130+53

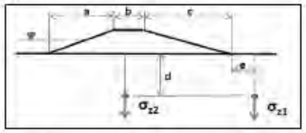
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Calaveras River and Stockton Diverting Canal
Levee Station: 3156+02
Boring Number: WCNBCR_012B

Prepared by: Vlad Perlea
Checked by:

Date: 5/21/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	23.1 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	13.6 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-1.5 ft	Hammer Efficiency	85	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	-1.0 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	24.9 ft
Crest Width, b (ft)	7.0 ft
Landside/Downstream Slope, c (ft)	20.1 ft
Dist. of Boring from Levee Toe [1] (ft)	-20.9 ft
Embankment Height, H (ft)	9.5 ft

Boring WCNBCR_012B
Boring on the crest
SPT Ground Elevation Used in Analysis
23.10 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
37.0	-13.9	5	SM	20		120	125	3970.0	3196.3	608.0	3364.5	2559.5	0.81	1	1	1.00	5.8	3.61	1.08	9.8	0.11	0.87	0.15	1.00	0.80	0.96	1.08
44.0	-20.9	10	SM	40		120	125	4756.1	3547.5	521.1	4239.5	2997.7	0.77	1	1	1.00	10.9	5.00	1.20	16.1	0.19	0.82	0.15	1.00	0.76	0.92	1.78

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.
[2] Soil description may be used to estimate fines content where lab testing is not available.
Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.
Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.
[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.
[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

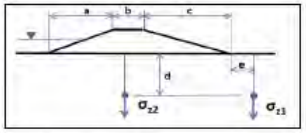
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Calaveras River and Stockton Diverting Canal
Levee Station: 6669+40
Boring Number: WR1614_019B

Prepared by: Vlad Perlea
Checked by:

Date: 5/21/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	25.4 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	15.4 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	10.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	1.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	4.0 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	21.5 ft
Crest Width, b (ft)	11.0 ft
Landside/Downstream Slope, c (ft)	28.1 ft
Dist. of Boring from Levee Toe [1] (ft)	22.5 ft
Embankment Height, H (ft)	10.0 ft

Boring WR1614_019B
Boring on waterside or landside field
SPT Ground Elevation Used in Analysis
15.40 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
6.0	9.4	43	SM	40	Unsaturated	120	125	721.4	721.4	1.4	720.0	720.0	1.70	1	0.8	1.00	75.0	5.00	1.20	95.1	n.a.	0.99	0.13	1.00	0.60	1.00	#N/A
12.0	3.4	8	ML	81		120	125	1449.5	1449.5	9.5	1443.0	1405.6	1.21	1	0.85	1.00	10.5	5.00	1.20	17.7	0.19	0.97	0.13	1.00	0.76	1.00	2.17
17.0	-1.6	8	MH	100		120	125	2075.7	1913.5	22.7	2068.0	1718.6	1.05	1	0.95	1.00	10.3	5.00	1.20	17.3	0.18	0.96	0.15	1.00	0.77	1.00	1.84
22.0	-6.6	14	ML	60		120	125	2718.2	2343.8	40.2	2693.0	2031.6	0.95	1	0.95	1.00	16.2	5.00	1.20	24.5	0.28	0.95	0.16	1.00	0.71	1.00	2.58
27.0	-11.6	0	ML	53		120	125	3362.9	2988.5	59.9	3318.0	2344.6	0.84	1	1	1.00	0.0	5.00	1.20	5.0	0.07	0.94	0.17	1.00	0.80	0.98	0.61

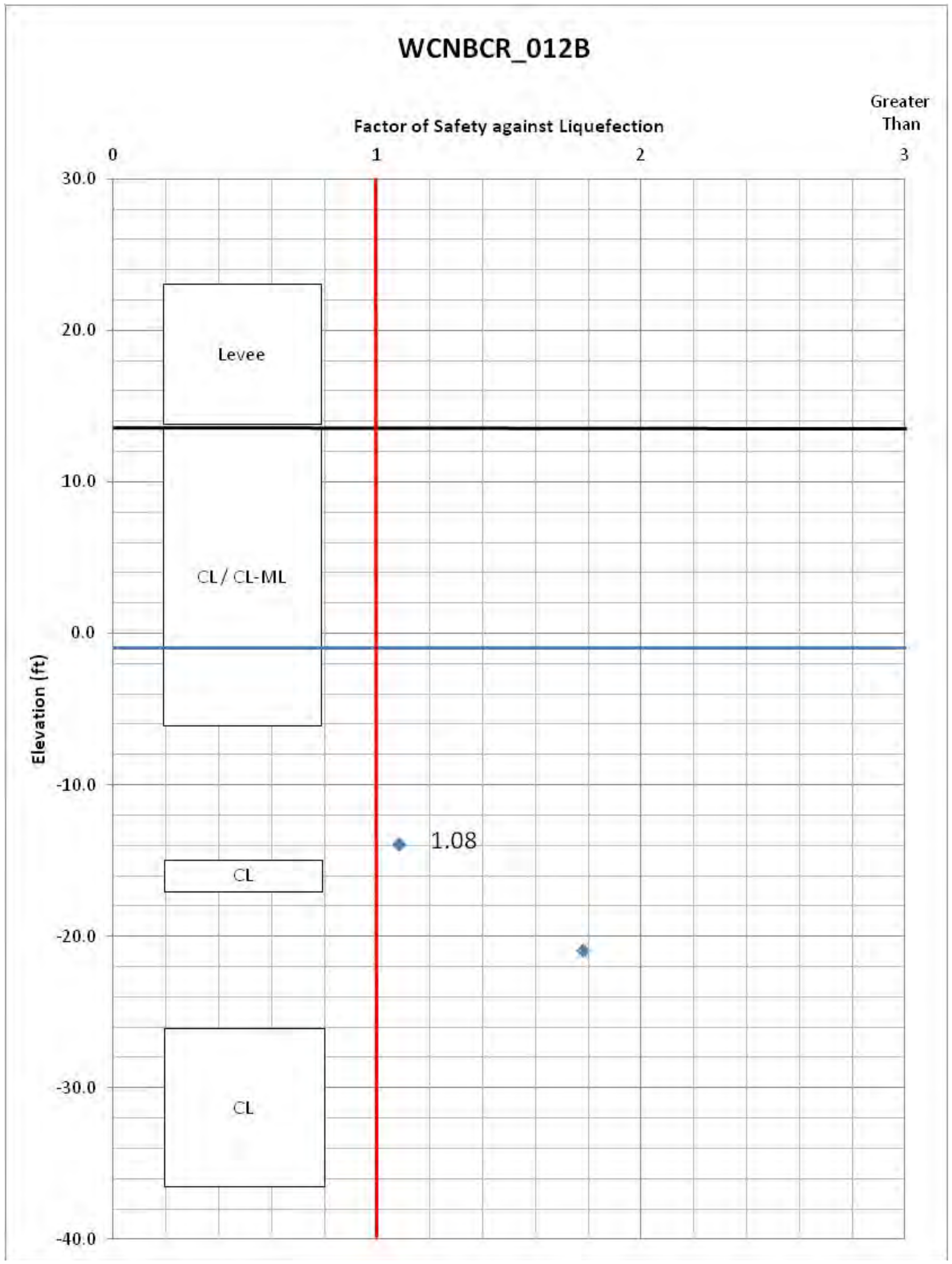


Fig. C-35. Calaveras River, Station 3156+02

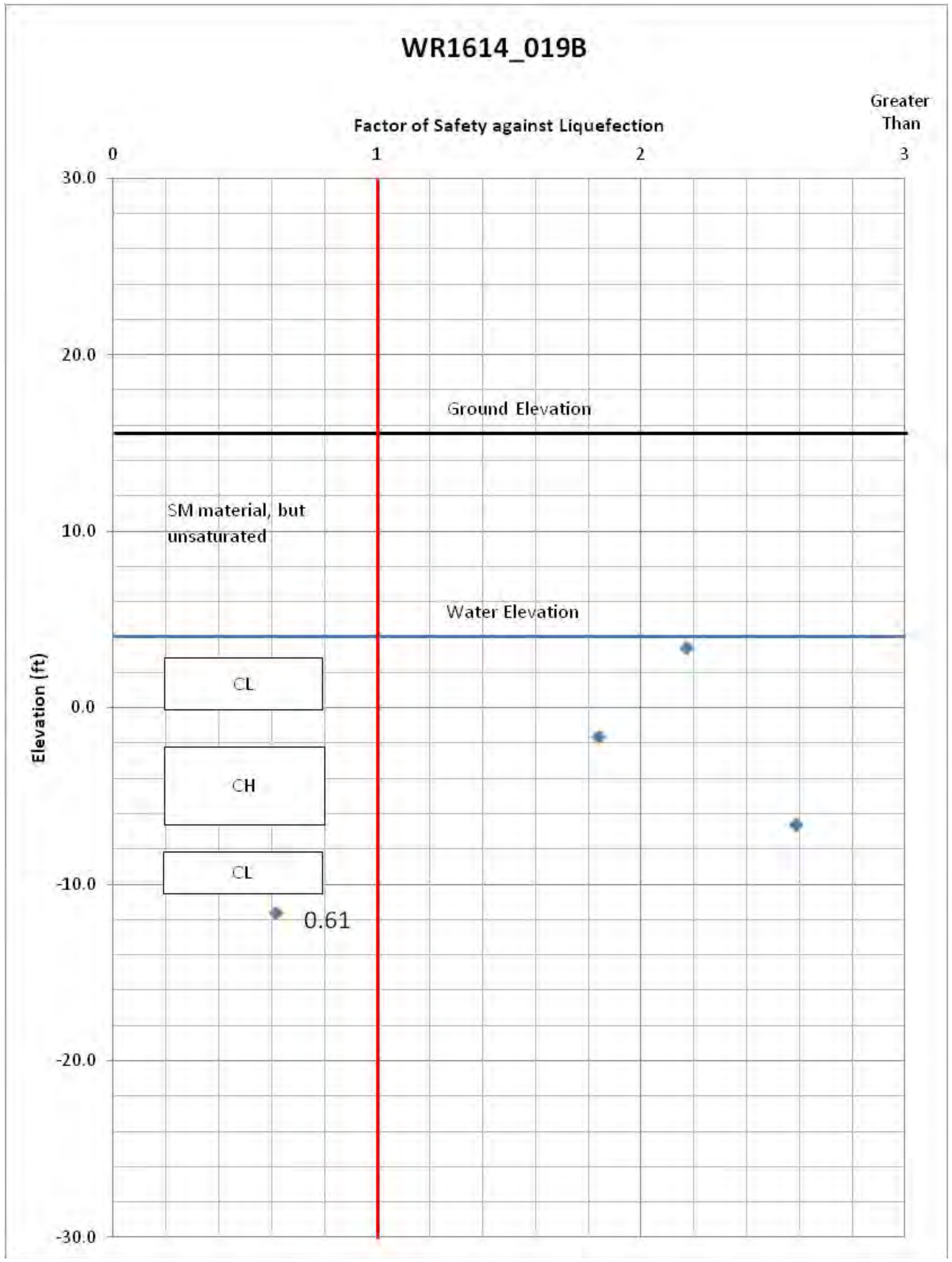


Fig. C-36. Calaveras River, Station 6669+40

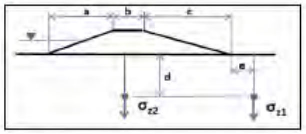
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Calaveras River and Stockton Diverting Canal
Levee Station: 3238+00
Boring Number: WCNBCR_013B

Prepared by: Vlad Perlea
Checked by:

Date: 5/21/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	25.5 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	14.4 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-1.5 ft	Hammer Efficiency	85	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	-1.0 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	83.3 ft
Crest Width, b (ft)	21.0 ft
Landside/Downstream Slope, c (ft)	21.1 ft
Dist. of Boring from Levee Toe [1] (ft)	-31.6 ft
Embankment Height, H (ft)	11.1 ft

Boring	WCNBCR_013B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
25.50 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
24.0	1.5	15	SM	45	unsaturated	120	125	2803.4	2803.4	1255.4	2880.0	2880.0	0.87	1	0.95	1.00	17.5	5.00	1.20	26.0	n.a.	0.94	0.12	1.00	0.70	0.91	#N/A

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for linter (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

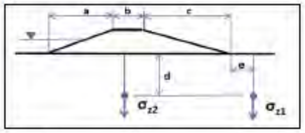
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Stockton Diverting Canal
Levee Station: 5762+29
Boring Number: WCSBCR_004B

Prepared by: Vlad Perlea
Checked by:

Date: 7/22/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	32.8 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	20.8 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.18
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-4.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	3.0 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	30.0 ft
Crest Width, b (ft)	25.0 ft
Landside/Downstream Slope, c (ft)	33.6 ft
Dist. of Boring from Levee Toe [1] (ft)	-46.1 ft
Embankment Height, H (ft)	12.0 ft

Boring	WCSBCR_004B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
32.80 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
6.0	26.8	10	ML	60		120	125	720.0	720.0	0.0	Embankment	Embankment	1.70	1	0.8	1.00	17.5	5.00	1.20	25.9	0.31	0.99	#N/A	1.00	0.70	#N/A	#N/A
13.0	19.8	7	ML	70		120	125	1559.9	1559.9	1439.9	1560.0	1560.0	1.16	1	0.95	1.00	9.9	5.00	1.20	16.9	0.18	0.97	0.11	1.00	0.77	1.00	2.38
16.0	16.8	6	ML	62		120	125	1916.6	1916.6	1436.5	1920.0	1920.0	1.05	1	0.95	1.00	7.7	5.00	1.20	14.2	0.15	0.96	0.11	1.00	0.79	1.00	2.03
46.0	-13.2	27	ML	62		120	125	5168.7	4794.3	1042.7	4161.0	3150.1	0.66	1	1	1.00	23.0	5.00	1.20	32.6	2.00	0.80	0.12	1.00	0.65	0.87	3.00

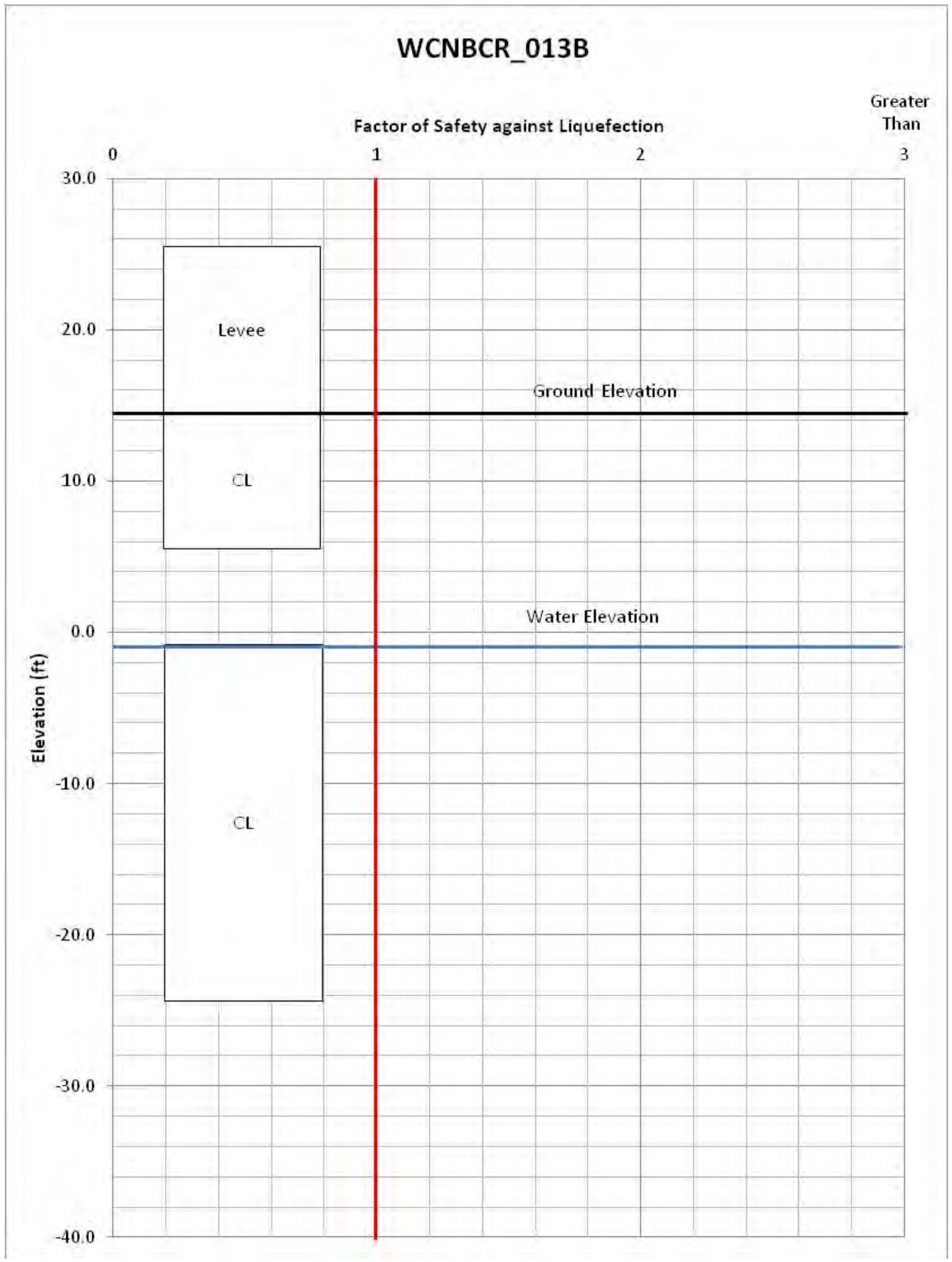


Fig. C-37. Calaveras River, Station 3238+00

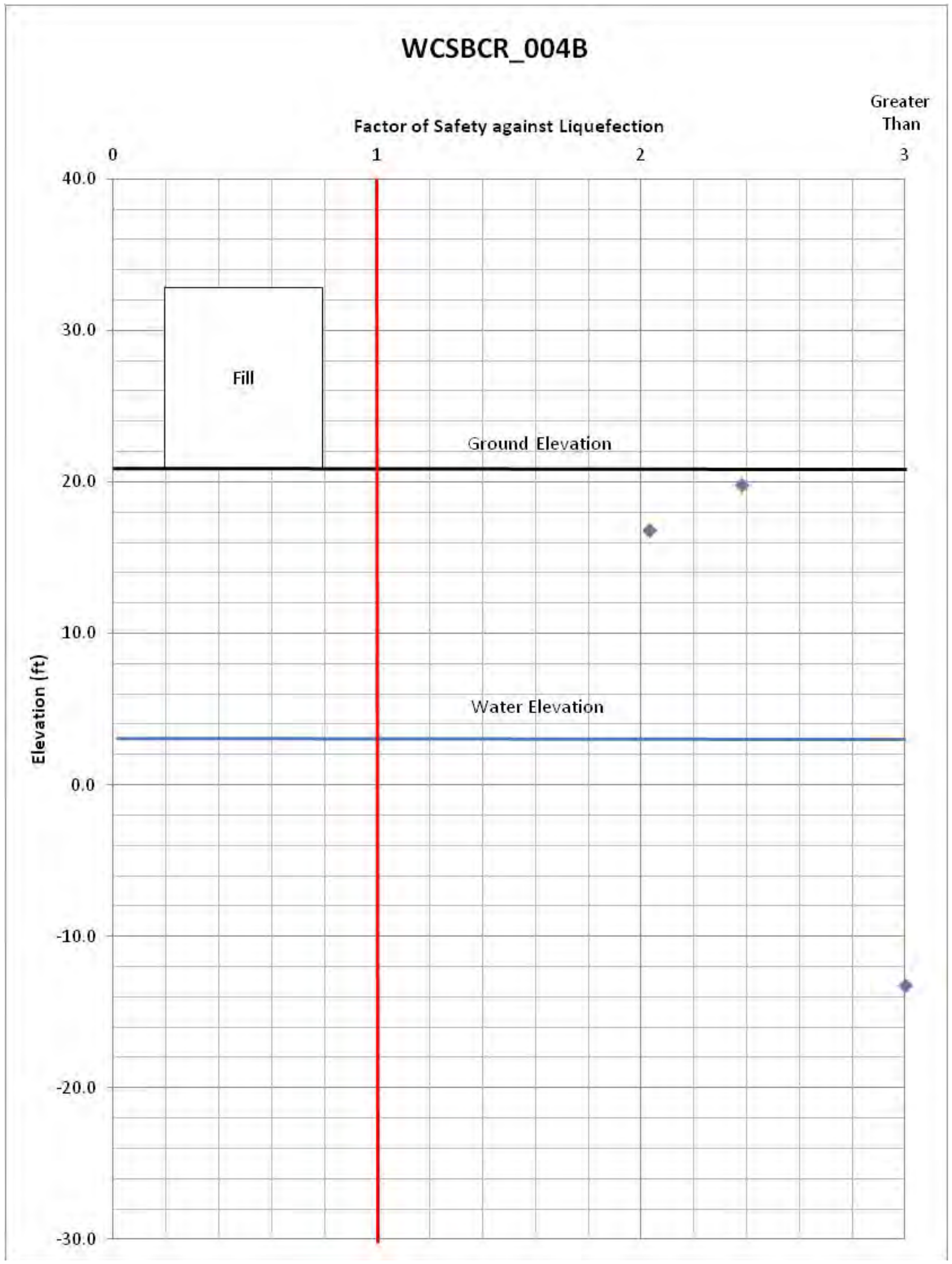


Fig. C-38. Calaveras River, Station 6762+29

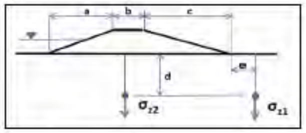
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Stockton Diverting Canal
Levee Station: 811+98
Boring Number: WCSBDC_001B

Prepared by: Vlad Perlea
Checked by:

Date: 7/22/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	42.8 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	24.8 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.18
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4		
Groundwater Elevation during Drilling (ft)	-5.2 ft	Hammer Efficiency	84.5	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	24.8 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	41.4 ft
Crest Width, b (ft)	31.0 ft
Landside/Downstream Slope, c (ft)	41.4 ft
Dist. of Boring from Levee Toe [1] (ft)	-56.9 ft
Embankment Height, H (ft)	18.0 ft

Boring	**WCSBDC_001B**
Boring on the crest	
SPT Ground Elevation Used in Analysis	
42.80 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
26.0	16.8	20	ML	77		120	125	3101.3	3101.3	2141.3	1000.0	500.8	0.83	1	1	1.00	23.3	5.00	1.20	32.9	2.00	0.94	0.22	1.00	0.65	1.00	3.00
31.0	11.8	29	SM	15		120	125	3654.8	3654.8	2094.8	1625.0	813.8	0.76	1	1	1.00	31.1	2.50	1.05	35.1	2.00	0.92	0.22	1.00	0.60	1.00	3.00
36.0	6.8	31	ML	80		120	125	4183.7	4183.7	2023.7	2250.0	1126.8	0.71	1	1	1.00	31.0	5.00	1.20	42.3	2.00	0.88	0.21	1.00	0.60	1.00	3.00
41.0	1.8	42	ML	80		120	125	4697.4	4697.4	1937.4	2875.0	1439.8	0.67	1	1	1.00	39.7	5.00	1.20	52.6	2.00	0.84	0.20	1.00	0.60	1.00	3.00
46.0	-3.2	46	ML	80		120	125	5204.1	5204.1	1844.1	3500.0	1752.8	0.64	1	1	1.00	41.3	5.00	1.20	54.6	2.00	0.80	0.19	1.00	0.60	1.00	3.00

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1976) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

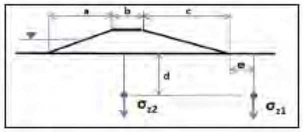
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Stockton Diverting Canal
Levee Station: 883+93
Boring Number: WCSBDC_005B

Prepared by: Vlad Perlea
Checked by:

Date: 7/22/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	39.2 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	24.2 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.18
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	6		
Groundwater Elevation during Drilling (ft)	-5.8 ft	Hammer Efficiency	83.5	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	24.2 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	34.5 ft
Crest Width, b (ft)	31.0 ft
Landside/Downstream Slope, c (ft)	34.5 ft
Dist. of Boring from Levee Toe [1] (ft)	-50.0 ft
Embankment Height, H (ft)	15.0 ft

Boring	**WCSBDC_006B**
Boring on the crest	
SPT Ground Elevation Used in Analysis	
39.20 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
26.0	13.2	15	ML	87		120	125	3078.1	3078.1	1758.1	1375.0	688.6	0.83	1.05	1	1.00	18.2	5.00	1.20	26.8	0.33	0.94	0.22	1.00	0.69	1.00	2.29
31.0	8.2	16	ML	86		120	125	3618.2	3618.2	1698.2	2000.0	1001.6	0.76	1.05	1	1.00	17.9	5.00	1.20	26.5	0.32	0.92	0.22	1.00	0.69	1.00	2.26
36.0	3.2	34	ML	60		120	125	4141.0	4141.0	1621.0	2625.0	1314.6	0.71	1.05	1	1.00	35.5	5.00	1.20	47.6	2.00	0.88	0.21	1.00	0.60	1.00	3.00
42.5	-3.3	33	ML	73		120	125	4809.4	4809.4	1509.4	3437.5	1721.5	0.66	1.05	1	1.00	32.0	5.00	1.20	43.4	2.00	0.83	0.19	1.00	0.60	1.00	3.00
46.0	-6.8	54	ML	60		120	125	5173.3	5110.9	1448.3	3875.0	1940.6	0.64	1.05	1	1.00	50.8	5.00	1.20	65.9	2.00	0.80	0.19	1.00	0.60	1.00	3.00

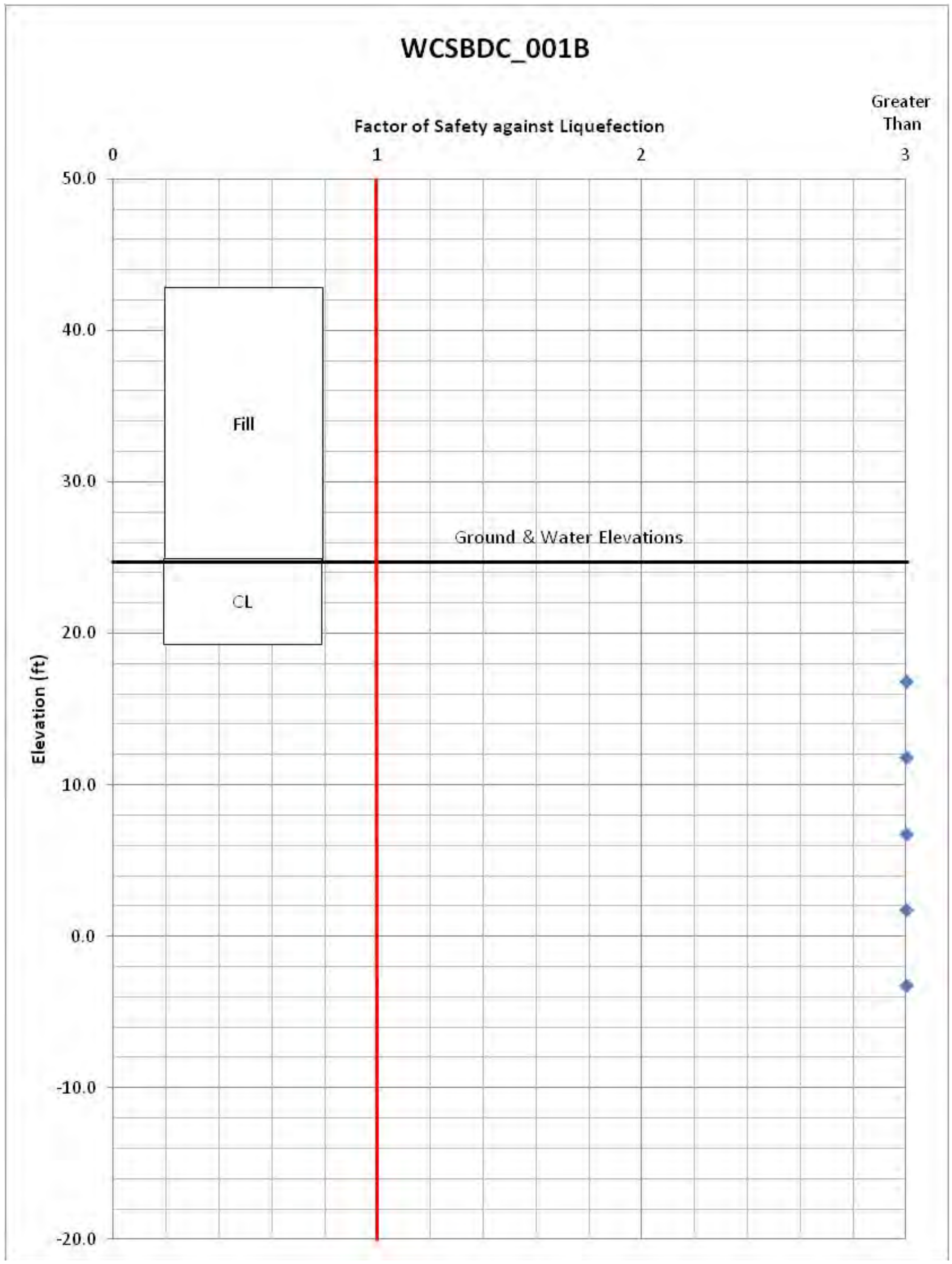


Fig. C-39. Stockton Diverting Canal, Station 811+98

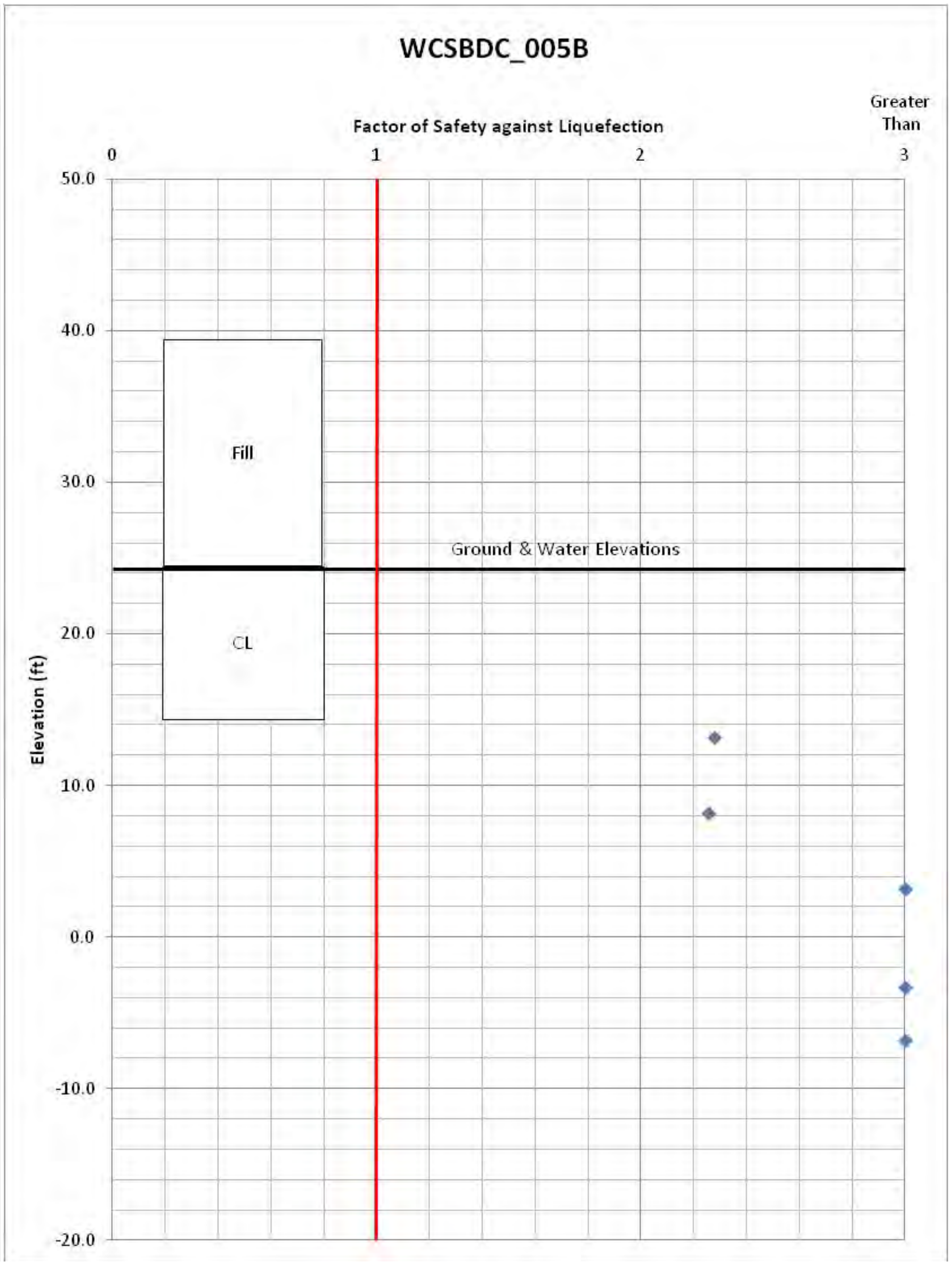


Fig. C-40. Stockton Diverting Canal, Station 883+93

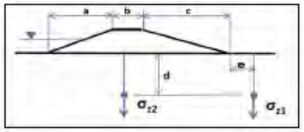
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Stockton Diverting Canal
Levee Station: 940+82
Boring Number: WCSBDC_008B

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	42.4 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	27.4 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.18
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	5		
Groundwater Elevation during Drilling (ft)	-2.6 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	27.4 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	34.5 ft
Crest Width, b (ft)	31.0 ft
Landside/Downstream Slope, c (ft)	34.5 ft
Dist. of Boring from Levee Toe [1] (ft)	-50.0 ft
Embankment Height, H (ft)	15.0 ft

Boring WCSBDC_008B
Boring on the crest
SPT Ground Elevation Used in Analysis
42.40 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
27.0	15.4	12	SM/ML	82		120	125	3188.0	3188.0	1748.0	1500.0	751.2	0.81	1	1	1.00	12.5	5.00	1.20	20.1	0.22	0.94	0.22	1.00	0.74	1.00	1.49
32.0	10.4	13	SM	30		120	125	3723.8	3723.8	1683.8	2125.0	1064.2	0.75	1	1	1.00	12.6	4.71	1.15	19.2	0.21	0.91	0.21	1.00	0.74	1.00	1.45
37.0	5.4	21	ML	77		120	125	4244.3	4244.3	1604.3	2750.0	1377.2	0.71	1	1	1.00	19.0	5.00	1.20	27.8	0.36	0.87	0.20	1.00	0.68	1.00	2.68
42.0	0.4	45	ML	77		120	125	4758.1	4758.1	1518.1	3375.0	1690.2	0.67	1	1	1.00	38.5	5.00	1.20	51.2	2.00	0.83	0.19	1.00	0.60	1.00	3.00
47.0	-4.6	17	ML	66		120	125	5281.0	5156.2	1431.0	4000.0	2003.2	0.64	1	1	1.00	14.0	5.00	1.20	21.8	0.24	0.79	0.18	1.00	0.73	1.00	1.94
52.0	-9.6	16	SM	34		120	125	5821.4	5384.6	1346.4	4625.0	2316.2	0.63	1	1	1.00	12.9	4.93	1.19	20.2	0.22	0.75	0.18	1.00	0.74	0.98	1.82

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

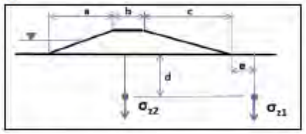
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Calaveras River and Stockton Diverting Canal
Levee Station: 978+49
Boring Number: WCSBDC_013B

Prepared by: Vlad Perlea
Checked by:

Date: 5/21/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	45.3 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	33.9 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.18
Height below Crest of Embankment (ft)	11.4 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	30.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	33.0 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	25.3 ft
Crest Width, b (ft)	31.0 ft
Landside/Downstream Slope, c (ft)	25.3 ft
Dist. of Boring from Levee Toe [1] (ft)	9.6 ft
Embankment Height, H (ft)	11.4 ft

Boring WCSBDC_013B
Boring on waterside or landside field
SPT Ground Elevation Used in Analysis
33.90 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
6.0	27.9	37	SM	45		120	125	740.9	609.9	10.4	745.5	427.3	1.70	1	0.8	1.00	64.6	5.00	1.20	82.5	2.00	0.99	0.20	1.00	0.60	1.00	3.00
13.0	20.9	26	ML	100		120	125	1667.1	1292.7	61.6	1620.5	865.5	1.28	1	0.95	1.00	40.6	5.00	1.20	53.7	2.00	0.97	0.21	1.00	0.60	1.00	3.00
18.0	15.9	29	ML	88		120	125	2339.9	1965.5	109.4	2245.5	1178.5	1.04	1	0.95	1.00	36.7	5.00	1.20	49.0	2.00	0.96	0.21	1.00	0.60	1.00	3.00
23.0	10.9	24	ML	90		120	125	3011.8	2637.4	156.3	2870.5	1491.5	0.90	1	0.95	1.00	26.2	5.00	1.20	36.5	2.00	0.95	0.21	1.00	0.63	1.00	3.00
33.0	0.9	27	CL	94	Clay	120	125	4339.9	3965.5	234.4	4120.5	2117.5	0.73	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.91	0.21	1.00	0.60	1.00	#N/A

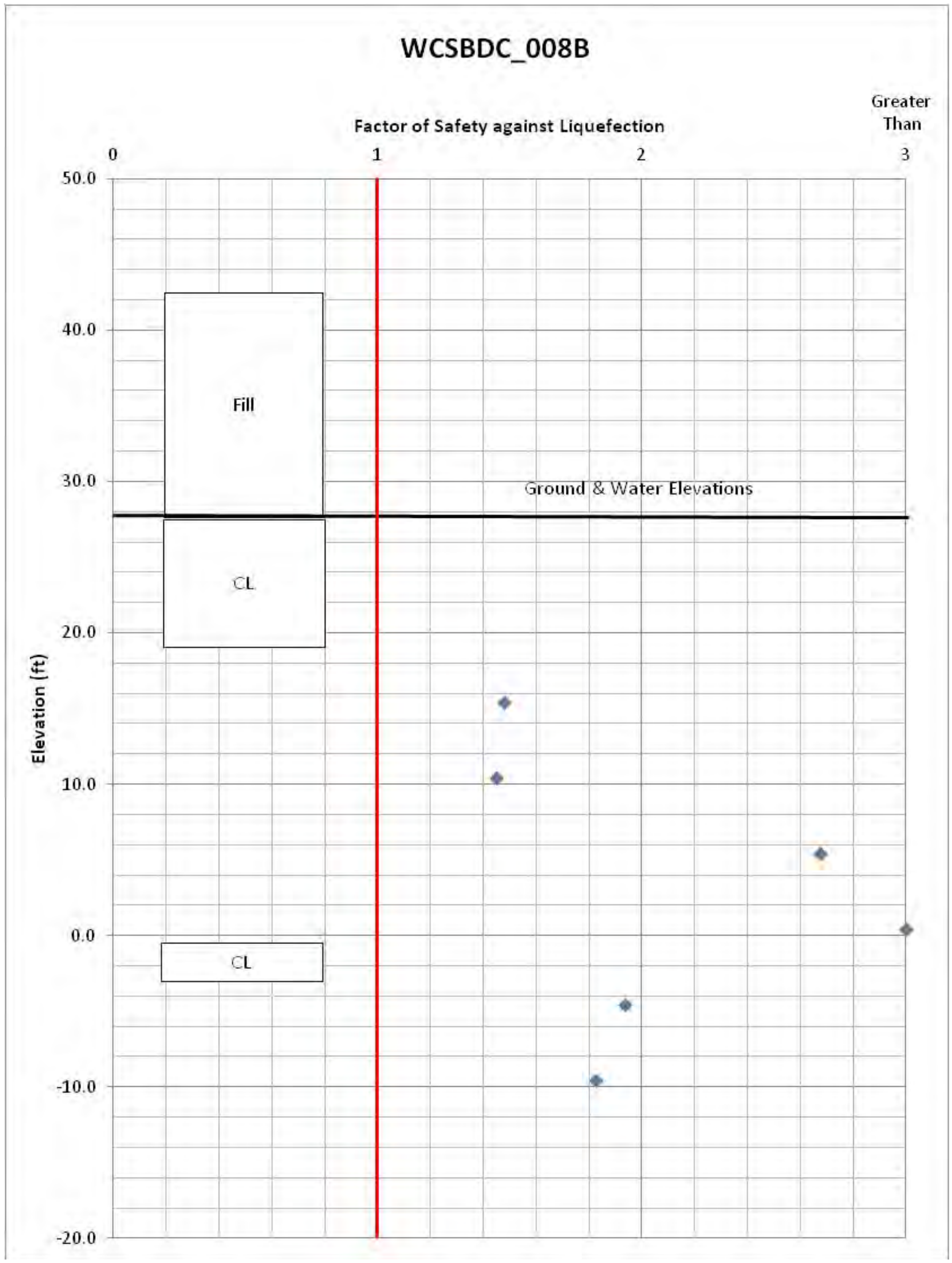


Fig. C-41. Stockton Diverting Canal, Station 940+82

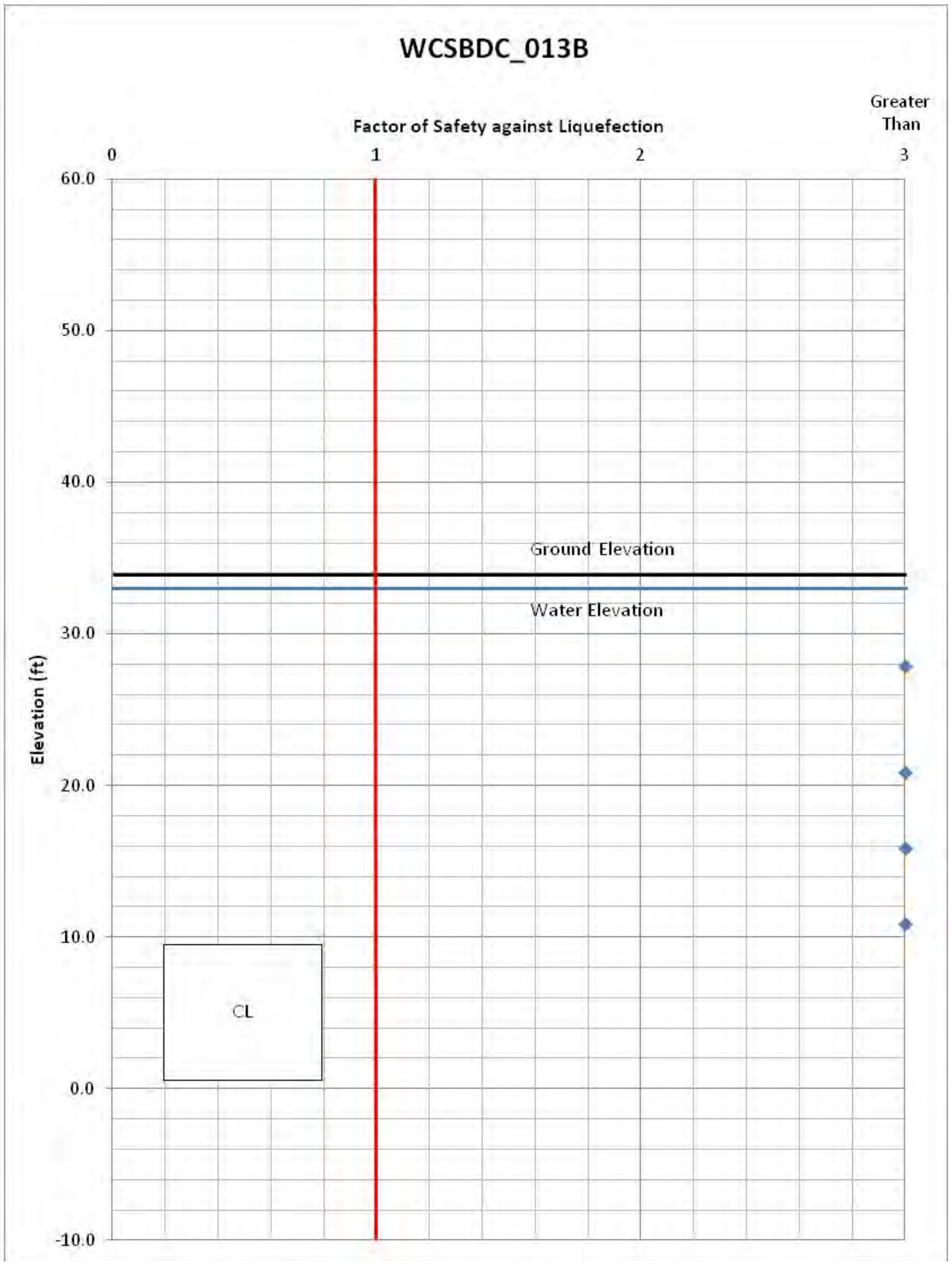


Fig. C-42. Stockton Diverting Canal, Station 978+49

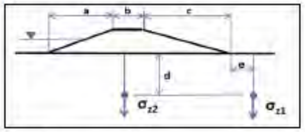
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Calaveras River and Stockton Diverting Canal
Levee Station: 1029+16
Boring Number: WCSBDC_014B

Prepared by: Vlad Perlea
Checked by:

Date: 5/21/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	51.3 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	35.7 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.18
Height below Crest of Embankment (ft)	15.6 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	30.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	35.0 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	26.6 ft
Crest Width, b (ft)	8.0 ft
Landside/Downstream Slope, c (ft)	28.2 ft
Dist. of Boring from Levee Toe [1] (ft)	63.6 ft
Embankment Height, H (ft)	15.6 ft

Boring	WCSBDC_014B
Boring on waterside or landside field	
SPT Ground Elevation Used in Analysis	
35.70 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
6.0	29.7	21	ML	74		120	125	721.6	702.9	0.1	746.5	415.8	1.70	1	0.8	1.00	36.7	5.00	1.20	49.0	2.00	0.99	0.21	1.00	0.60	1.00	3.00
12.0	23.7	20	SM / ML	68		120	125	1472.6	1098.2	1.1	1496.5	791.4	1.39	1	0.85	1.00	30.3	5.00	1.20	41.3	2.00	0.97	0.22	1.00	0.60	1.00	3.00
17.0	18.7	22	SM	13		120	125	2099.5	1725.1	3.0	2121.5	1104.4	1.11	1	0.95	1.00	29.7	1.89	1.04	32.7	2.00	0.96	0.22	1.00	0.60	1.00	3.00
32.0	3.7	91	GW-GM	10		120	125	3987.9	3613.5	16.4	3996.5	2043.4	0.77	1	1	1.00	89.4	0.87	1.02	92.2	2.00	0.91	0.21	1.00	0.60	1.00	3.00

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

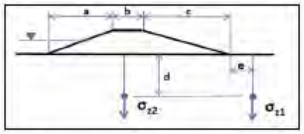
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Mormon Slough South
Levee Station: 2527+95
Boring Number: WCSBMS_003B

Prepared by: Vlad Perlea
Checked by:

Date: 5/21/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	50.4 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	44.6 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.165
Height below Crest of Embankment (ft)	5.8 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	40.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	44.0 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	23.8 ft
Crest Width, b (ft)	15.0 ft
Landside/Downstream Slope, c (ft)	50.5 ft
Dist. of Boring from Levee Toe [1] (ft)	10.2 ft
Embankment Height, H (ft)	5.8 ft

Boring	WCSBMS_003B
Boring on waterside or landside field	
SPT Ground Elevation Used in Analysis	
44.60 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
13.0	31.6	20	SM/ML	70		120	125	1617.8	1093.6	15.8	1622.0	848.2	1.39	1	0.95	1.00	33.9	5.00	1.20	45.7	2.00	0.97	0.20	1.00	0.60	1.00	3.00

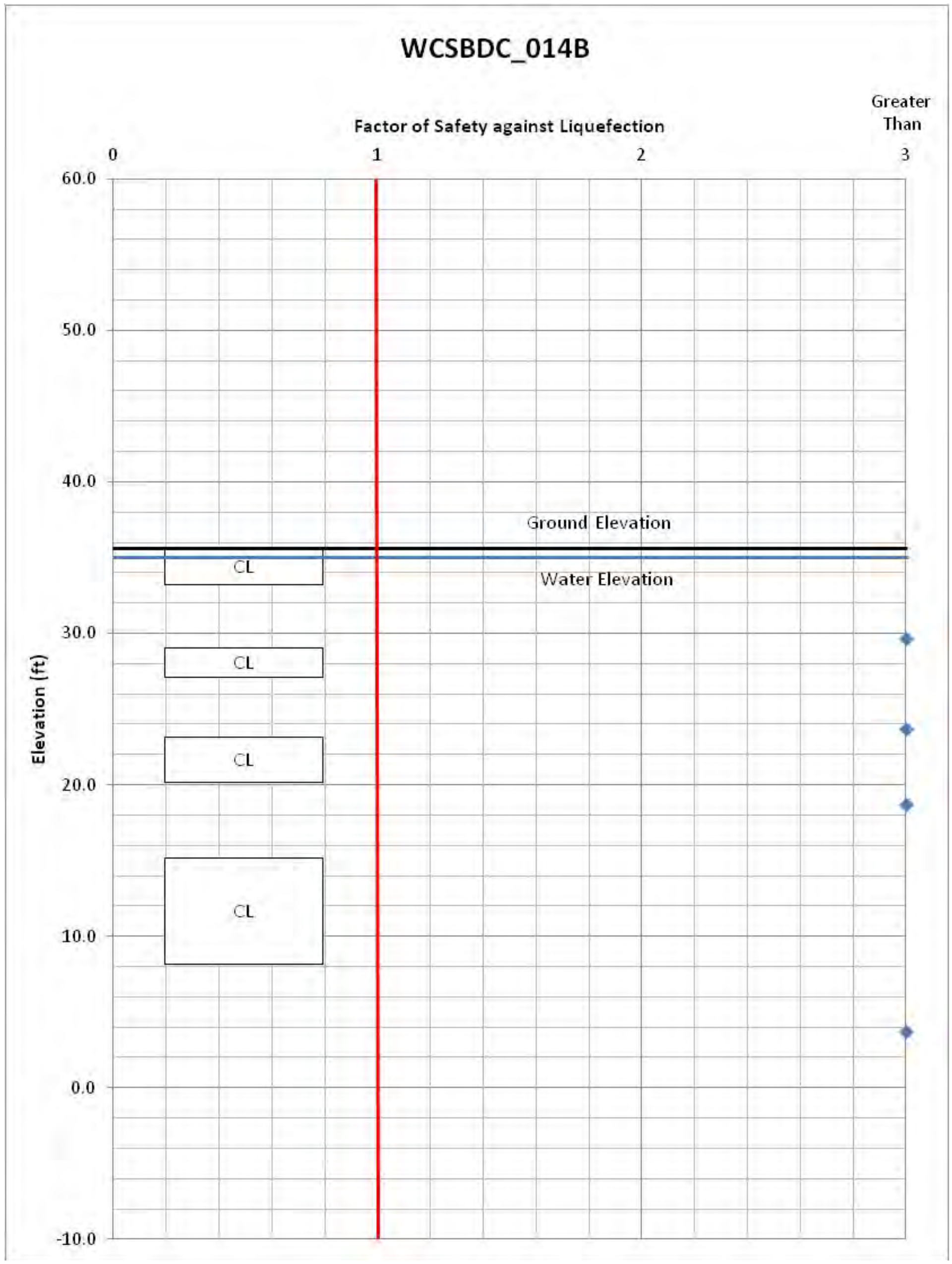


Fig. C-43. Stockton Diverting Canal, Station 1029+16

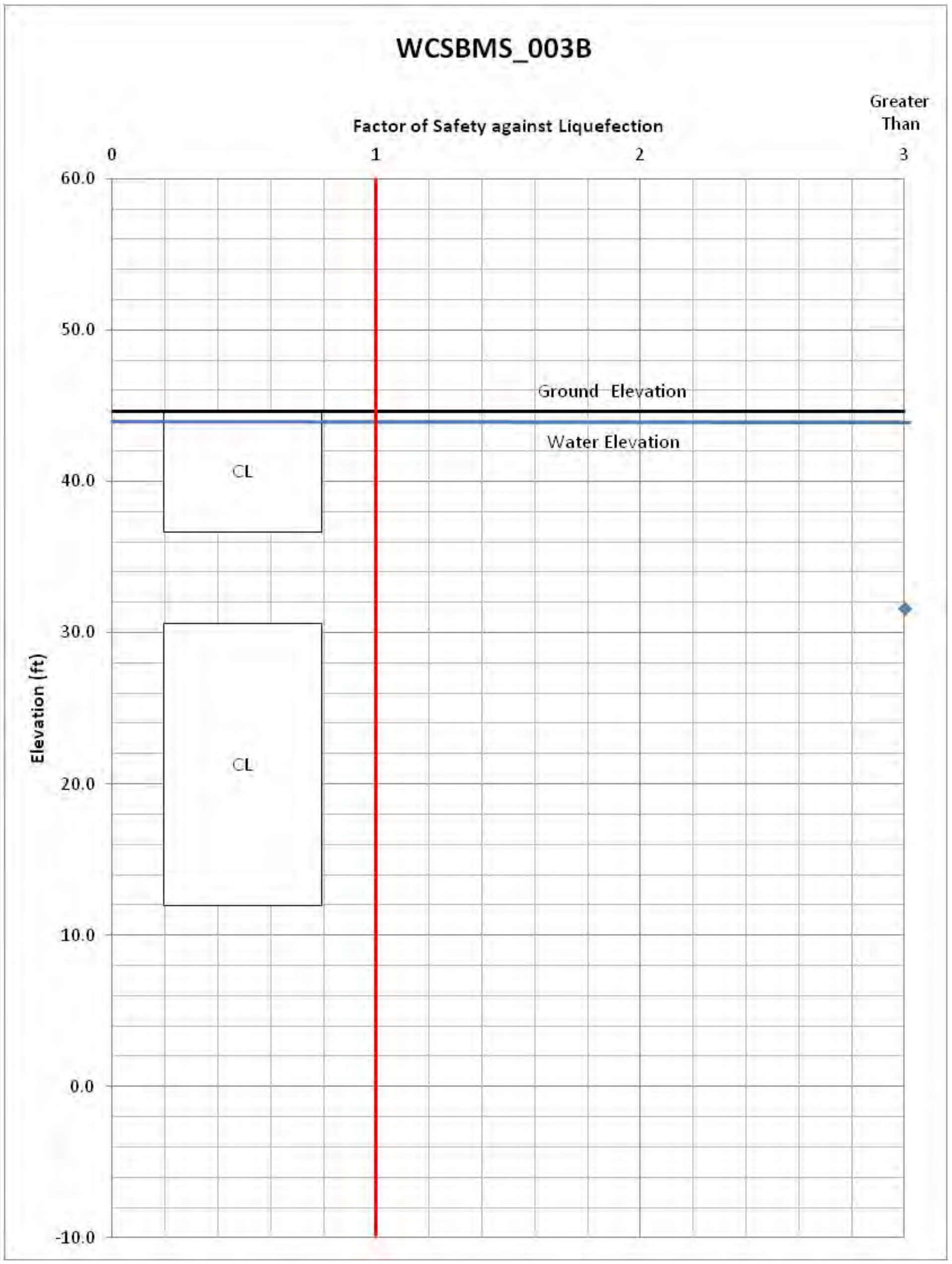


Fig. C-44. Mormon Slough, Station 2527+95

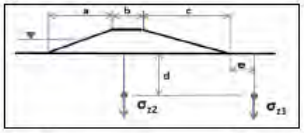
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Stockton Diverting Canal
Levee Station: 2583+28
Boring Number: WCSBMS_002B

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	56.4 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	51.4 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.165
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	5.75		
Groundwater Elevation during Drilling (ft)	-2.6 ft	Hammer Efficiency	84.5	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	51.4 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	16.0 ft
Crest Width, b (ft)	21.0 ft
Landside/Downstream Slope, c (ft)	9.0 ft
Dist. of Boring from Levee Toe [1] (ft)	-19.5 ft
Embankment Height, H (ft)	5.0 ft

Boring **WCSBMS_002B**
Boring on the crest
SPT Ground Elevation Used in Analysis
56.35 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
13.0	43.4	29	ML	77		120	125	1532.7	1532.7	566.7	1006.3	503.9	1.17	1.05	0.95	1.00	47.9	5.00	1.20	62.4	2.00	0.97	0.21	1.00	0.60	1.00	3.00
16.0	40.4	40	ML	77		120	125	1864.6	1864.6	538.6	1381.3	691.7	1.07	1.05	0.95	1.00	59.9	5.00	1.20	76.8	2.00	0.96	0.21	1.00	0.60	1.00	3.00
33.0	23.4	46	ML	64		120	125	3728.5	3728.5	362.5	3506.3	1755.9	0.75	1.05	1	1.00	51.2	5.00	1.20	66.5	2.00	0.91	0.19	1.00	0.60	1.00	3.00
38.0	18.4	40	ML	65		120	125	4290.0	4290.0	324.0	4131.3	2068.9	0.70	1.05	1	1.00	41.5	5.00	1.20	54.9	2.00	0.86	0.19	1.00	0.60	1.00	3.00
48.0	8.4	52	SM	49		120	125	5430.6	5430.6	264.6	5381.3	2694.9	0.62	1.05	1	1.00	48.0	5.00	1.20	62.6	2.00	0.78	0.17	1.00	0.60	0.91	3.00

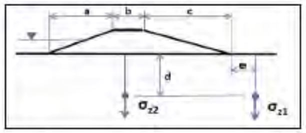
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Brookside
Levee Station: 117+51
Boring Number: WR2074_003M

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	14.0 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	4.5 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	79	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	3.2 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	23.8 ft
Crest Width, b (ft)	18.2 ft
Landside/Downstream Slope, c (ft)	20.0 ft
Dist. of Boring from Levee Toe [1] (ft)	-29.1 ft
Embankment Height, H (ft)	9.5 ft

Boring **WR2074_003M**
Boring on the crest
SPT Ground Elevation Used in Analysis
14.00 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
31.5	-17.5	6	SM	25		120	125	3577.8	2610.6	860.3	2743.5	1451.8	0.90	1	1	1.00	7.1	4.29	1.12	12.2	0.13	0.92	0.23	1.00	0.80	1.00	0.89
36.0	-22.0	8	SP-SM	10		120	125	4066.2	2818.2	786.2	3306.0	1733.5	0.87	1	1	1.00	9.1	0.87	1.02	10.2	0.11	0.88	0.22	1.00	0.78	1.00	0.79

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for inter (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

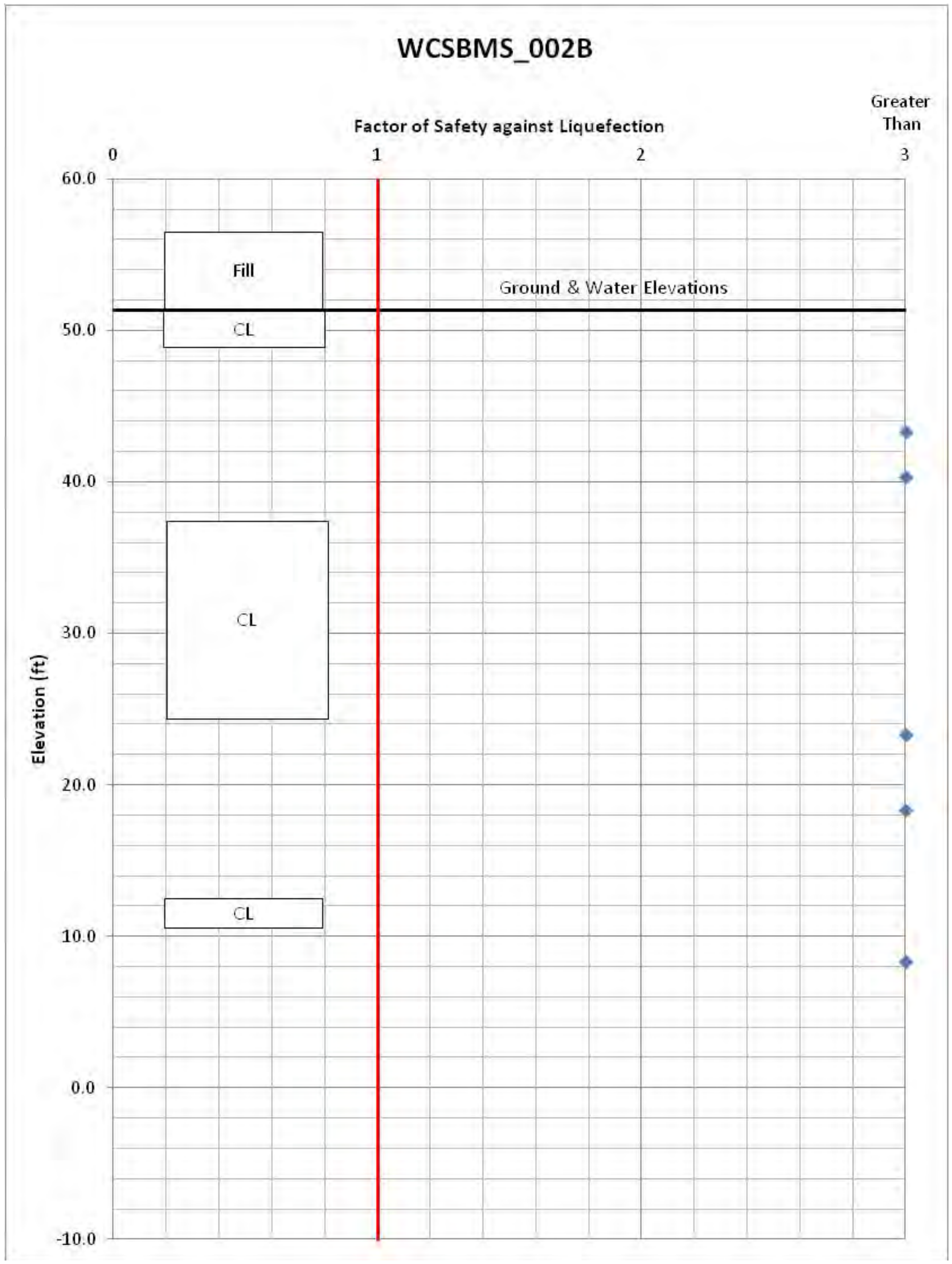


Fig. C-45. Mormon Slough, Station 2583+28

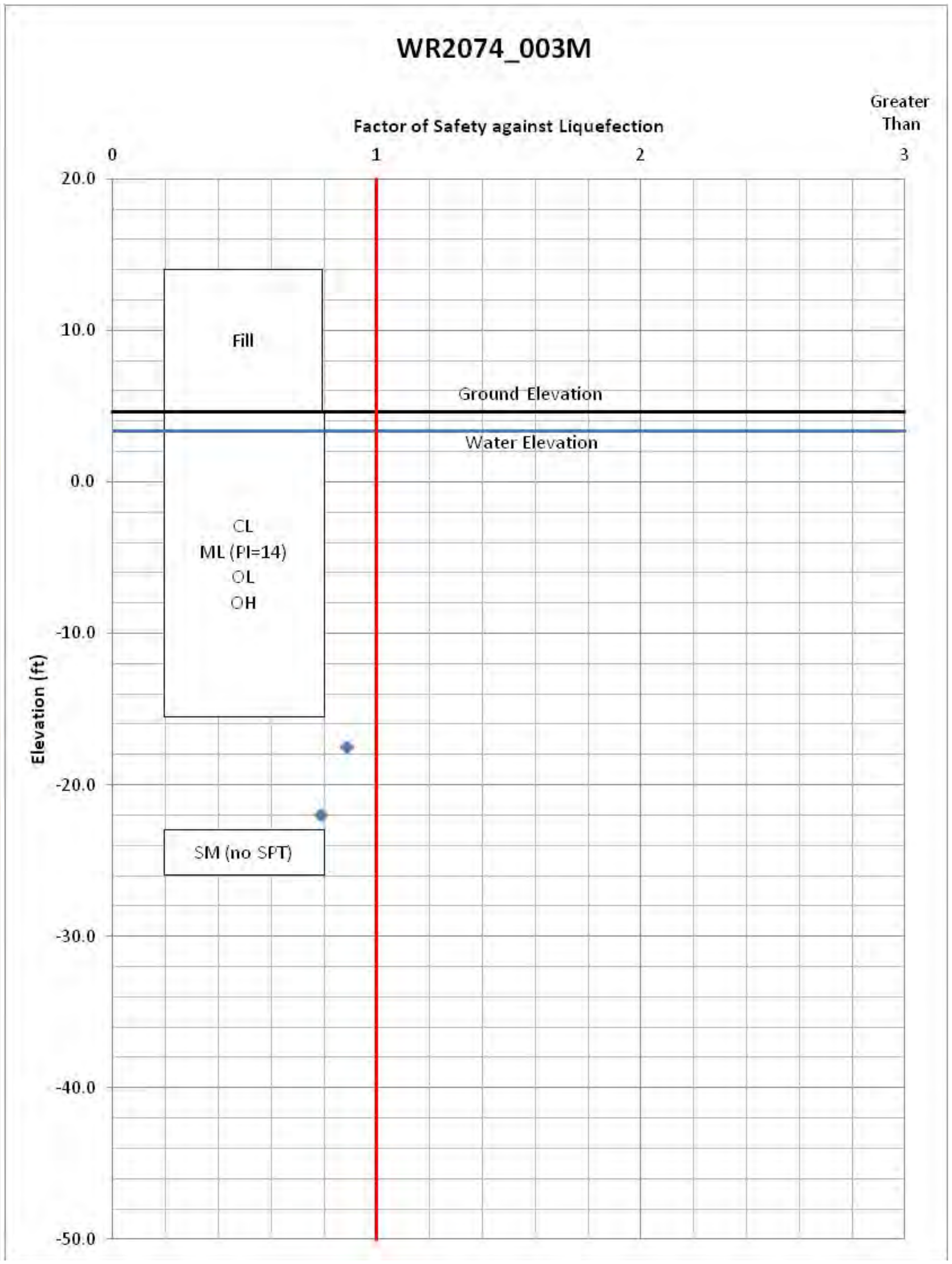


Fig. C-46. Brookside, Station 117+51

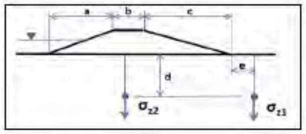
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Brookside
River Section: 118+02
Boring Number: WR2074_009B

Prepared by: Vlad Perlea
Checked by:

Date: 5/31/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	14.6 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	7.1 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	0.0 ft	Hammer Efficiency	79	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	1.1 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	70.1 ft
Crest Width, b (ft)	15.8 ft
Landside/Downstream Slope, c (ft)	48.9 ft
Dist. of Boring from Levee Toe [1] (ft)	-56.8 ft
Embankment Height, H (ft)	7.5 ft

Boring WR2074_009B
Boring on the crest
SPT Ground Elevation Used in Analysis
14.60 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
36.0	-21.4	10	SM	19		120	125	4262.7	2927.4	735.7	3532.5	2128.5	0.85	1	1	1.00	11.2	3.43	1.07	15.4	0.16	0.88	0.19	1.00	0.76	1.00	1.30
41.0	-26.4	11	SP	4		120	125	4850.6	3203.2	698.6	4157.5	2441.5	0.81	1	1	1.00	11.8	0.00	1.00	11.8	0.13	0.84	0.19	1.00	0.75	0.97	1.00
46.0	-31.4	6	SP-SM	10		120	125	5440.1	3480.8	663.1	4782.5	2754.5	0.78	1	1	1.00	6.2	0.87	1.02	7.2	0.09	0.80	0.18	1.00	0.80	0.95	0.70

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1976) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

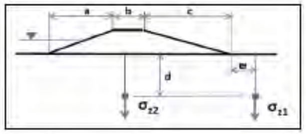
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Brookside
Levee Station: 133+44
Boring Number: WR2074_010B

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	15.2 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	-0.6 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-0.8 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	-0.6 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	67.9 ft
Crest Width, b (ft)	15.2 ft
Landside/Downstream Slope, c (ft)	41.1 ft
Dist. of Boring from Levee Toe [1] (ft)	-48.7 ft
Embankment Height, H (ft)	15.8 ft

Boring WR2074_010B
Boring on the crest
SPT Ground Elevation Used in Analysis
15.20 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
41.0	-25.8	24	SM	34		120	125	4716.4	3156.4	1561.4	3156.0	1583.5	0.82	1	1	1.00	25.2	4.93	1.19	34.9	2.00	0.84	0.22	1.00	0.64	1.00	3.00
48.5	-33.3	16	SM	19		120	125	5526.5	3498.5	1434.0	4093.5	2053.0	0.78	1	1	1.00	16.0	3.43	1.07	20.6	0.22	0.78	0.20	1.00	0.71	1.00	1.65
56.0	-40.8	21	SM	21		120	125	6347.2	3851.2	1317.2	5031.0	2522.5	0.74	1	1	1.00	20.0	3.78	1.09	25.5	0.30	0.72	0.19	1.00	0.68	0.94	2.30
61.0	-45.8	22	ML	65		120	125	6900.9	4342.5	1245.9	5656.0	2835.5	0.70	1	1	1.00	19.7	5.00	1.20	28.6	0.39	0.68	0.18	1.00	0.68	0.91	3.00

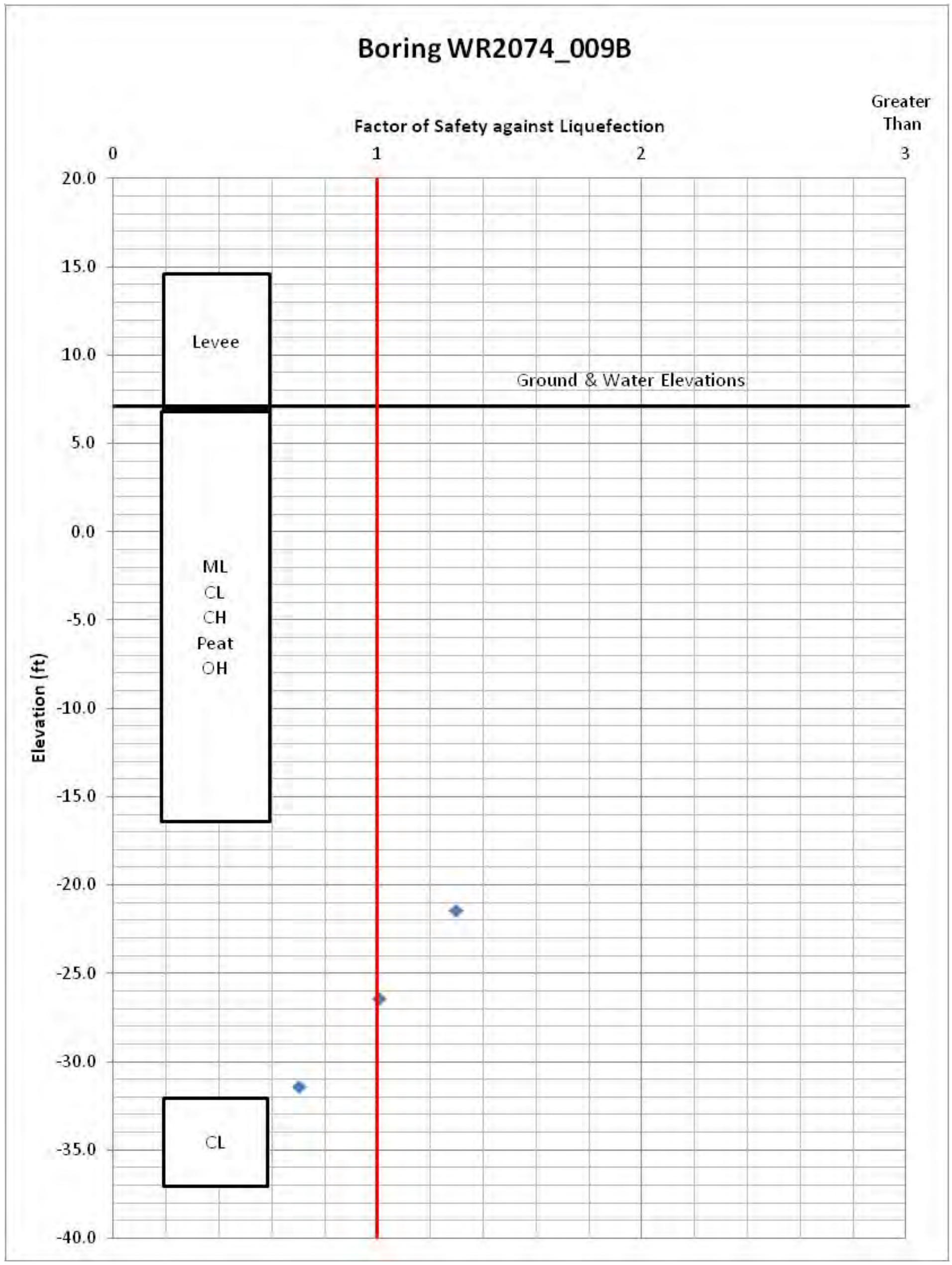


Fig. C-47. Brookside, Station 118+02

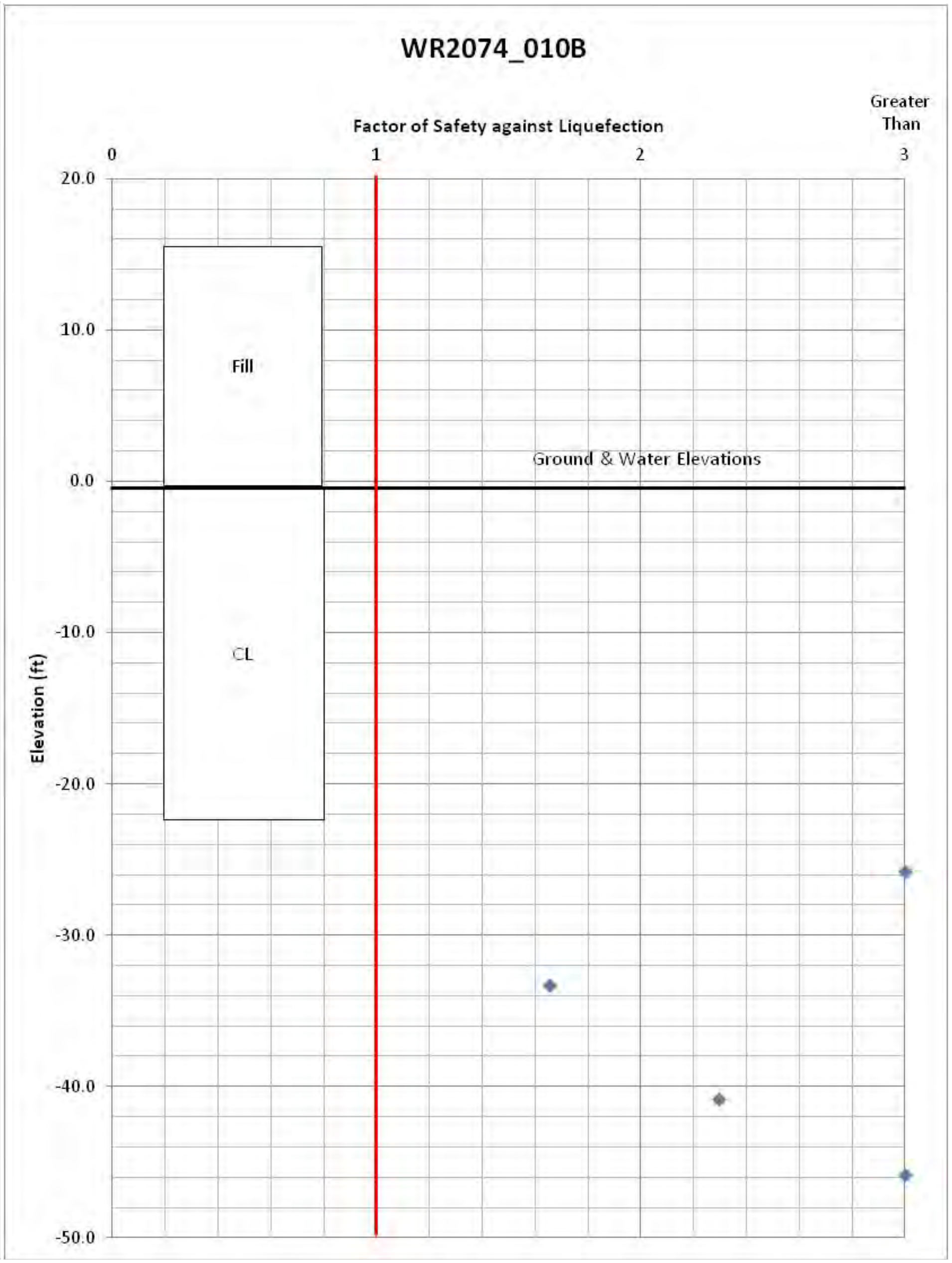


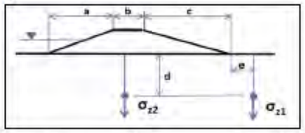
Fig. C-48. Brookside, Station 133+44

LIQUERFACTION TRIGGERING ANALYSIS

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea	**Date:**	5/31/2013
Study Area:	Brookside	**Checked by:**		**Date:**	
River Section:	133+82				
Boring Number:	WR2074_007B				

Input Parameters

Embankment Crest Elevation (ft)	15.2 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	5.5 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	5.2 ft	Hammer Efficiency	79	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	5.5 ft			120.0 pcf	



Surcharge Information

Waterside/Upstream Slope, a (ft)	41.7 ft
Crest Width, b (ft)	15.2 ft
Landside/Downstream Slope, c (ft)	25.2 ft
Dist. of Boring from Levee Toe [1] (ft)	-32.8 ft
Embankment Height, H (ft)	9.7 ft

Boring	WR2074_007B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
15.20 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
26.5	-11.3	8	SC	20		120	125	3095.9	2066.3	997.4	2100.0	1051.7	1.01	1	1	1.00	10.7	3.61	1.08	15.1	0.16	0.94	0.24	1.00	0.76	1.00	0.99
31.5	-16.3	23	SM	49		120	125	3643.8	2302.2	920.3	2725.0	1364.7	0.96	1	1	1.00	29.0	5.00	1.20	39.8	2.00	0.92	0.24	1.00	0.60	1.00	3.00
36.5	-21.3	8	SC	20		120	125	4196.2	2542.6	847.7	3350.0	1677.7	0.91	1	1	1.00	9.6	3.61	1.08	14.0	0.15	0.88	0.23	1.00	0.77	1.00	0.99

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

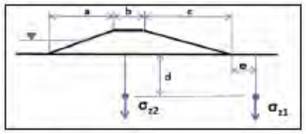
Updated April 2013

LIQUERFACTION TRIGGERING ANALYSIS

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea	**Date:**	7/23/2013
Study Area:	Brookside	**Checked by:**		**Date:**	
Levee Station:	160+48				
Boring Number:	WR2074_011B				

Input Parameters

Embankment Crest Elevation (ft)	17.6 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	0.6 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-0.8 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	0.6 ft			120.0 pcf	



Surcharge Information

Waterside/Upstream Slope, a (ft)	61.2 ft
Crest Width, b (ft)	20.5 ft
Landside/Downstream Slope, c (ft)	35.7 ft
Dist. of Boring from Levee Toe [1] (ft)	-46.0 ft
Embankment Height, H (ft)	17.0 ft

Boring	WR2074_011B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
17.60 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
44.5	-26.9	13	SM	47		120	125	5099.6	3471.0	1669.1	3437.5	1721.5	0.78	1	1	1.00	13.0	5.00	1.20	20.6	0.22	0.81	0.21	1.00	0.74	1.00	1.59
47.0	-29.4	42	SW-SM	12		120	125	5364.7	3580.1	1621.7	3750.0	1878.0	0.77	1	1	1.00	41.4	1.55	1.03	44.3	2.00	0.79	0.21	1.00	0.60	1.00	3.00
49.5	-31.9	33	SW-SM	12		120	125	5630.9	3690.2	1575.4	4062.5	2034.5	0.76	1	1	1.00	32.1	1.55	1.03	34.6	2.00	0.77	0.20	1.00	0.60	1.00	3.00
52.0	-34.4	25	SW-SM	12		120	125	5898.2	3801.6	1530.2	4375.0	2191.0	0.75	1	1	1.00	23.9	1.55	1.03	26.2	0.32	0.75	0.19	1.00	0.65	0.99	2.43
54.5	-36.9	20	SW-SM	12		120	125	6166.9	3914.3	1486.4	4687.5	2347.5	0.74	1	1	1.00	18.9	1.55	1.03	21.0	0.23	0.73	0.19	1.00	0.69	0.97	1.75

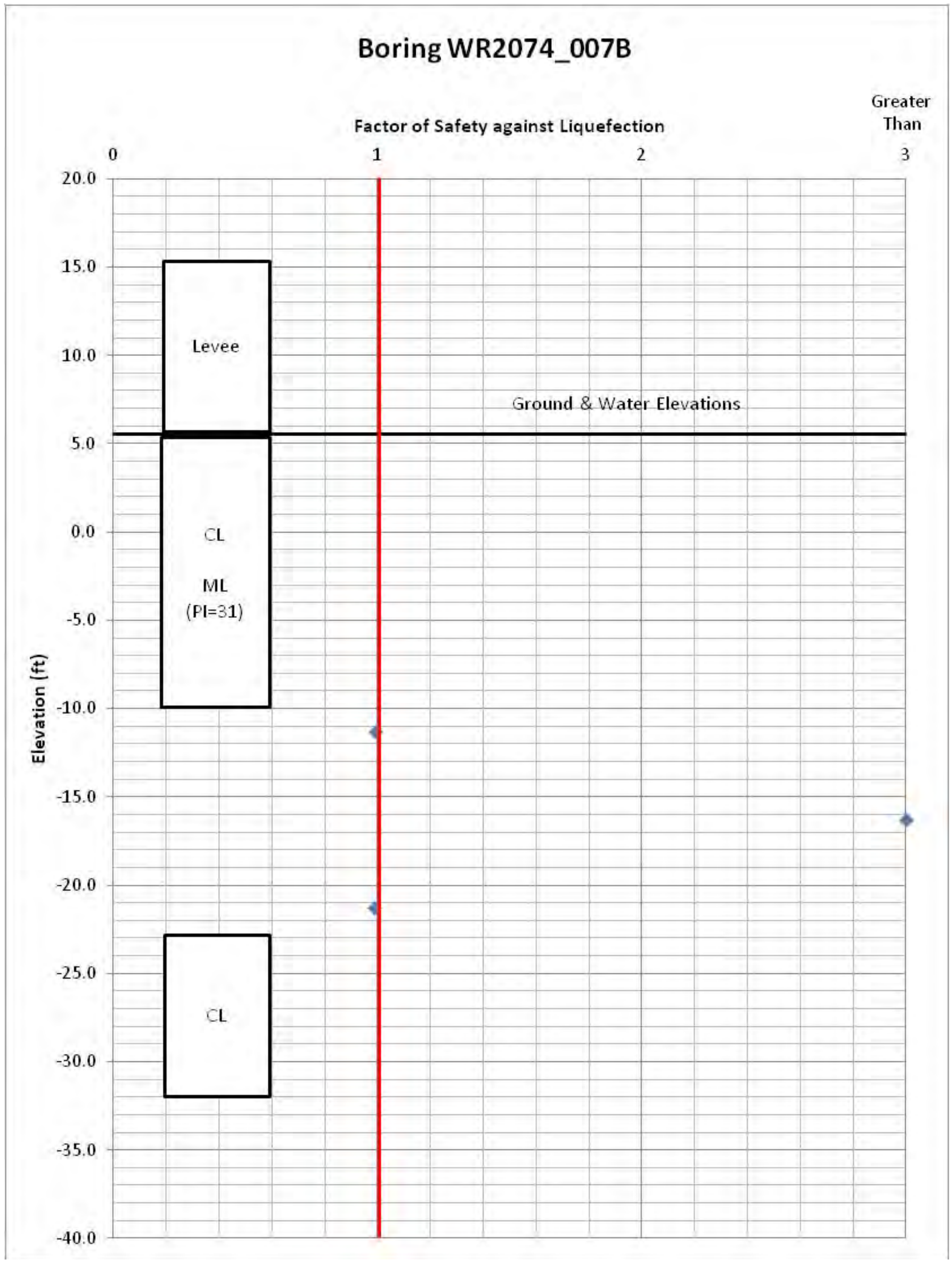


Fig. C-49. Brookside, Station 133+82

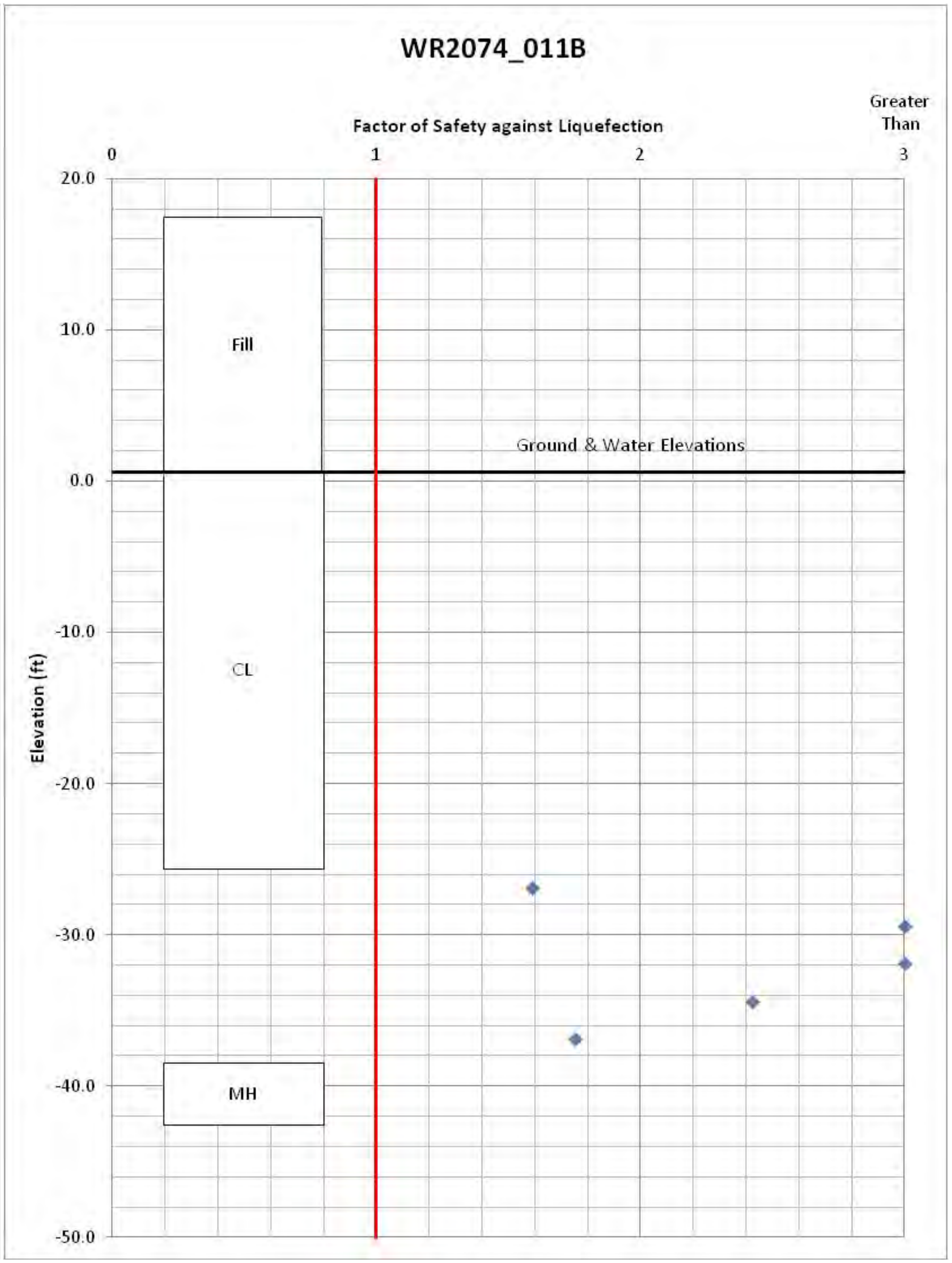


Fig. C-50. Brookside, Station 160+48

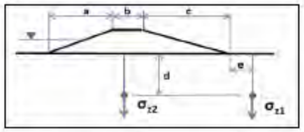
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Brookside
River Section: 185+78
Boring Number: WR2074_008B

Prepared by: Vlad Perlea
Checked by:

Date: 5/31/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	17.6 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	0.8 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	0.0 ft	Hammer Efficiency	79	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	0.8 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	55.4 ft
Crest Width, b (ft)	15.8 ft
Landside/Downstream Slope, c (ft)	38.6 ft
Dist. of Boring from Levee Toe[1] (ft)	-46.5 ft
Embankment Height, H (ft)	16.8 ft

Boring WR2074_008B
Boring on the crest
SPT Ground Elevation Used in Analysis
17.60 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
26.0	-8.4	9	CL	56	Clay	120	125	3097.1	2572.9	1951.1	1150.0	575.9	0.91	1	1	1.00	n.a.	5.00	1.20	n.a.	2.00	0.94	0.24	1.00	0.60	1.00	#N/A
41.0	-23.4	13	SM	41		120	125	4678.0	3217.9	1657.0	3025.0	1514.9	0.81	1	1	1.00	13.9	5.00	1.20	21.7	0.24	0.84	0.22	1.00	0.73	1.00	1.63
46.0	-28.4	15	SP-SM	9		120	125	5202.8	3590.4	1556.8	3650.0	1827.9	0.77	1	1	1.00	15.2	0.56	1.02	16.0	0.17	0.80	0.21	1.00	0.72	1.00	1.23
51.0	-33.4	14	ML	80		120	125	5733.0	4110.6	1462.0	4275.0	2140.9	0.72	1	1	1.00	13.2	5.00	1.20	20.9	0.23	0.76	0.20	1.00	0.74	1.00	1.72
61.0	-43.4	24	SM	45		120	125	6813.1	5190.7	1292.1	5525.0	2766.9	0.64	1	1	1.00	20.2	5.00	1.20	29.2	0.42	0.68	0.18	1.00	0.68	0.92	3.00
66.0	-48.4	26	SP-SM	8		120	125	7363.2	5740.8	1217.2	6150.0	3079.9	0.61	1	1	1.00	20.8	0.30	1.01	21.3	0.23	0.64	0.17	1.00	0.67	0.88	1.87

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method: answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

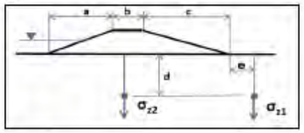
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Brookside
Levee Station: 217+77
Boring Number: WR2074_012B

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	13.4 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	0.9 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.6 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	0.9 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	50.0 ft
Crest Width, b (ft)	33.0 ft
Landside/Downstream Slope, c (ft)	41.3 ft
Dist. of Boring from Levee Toe[1] (ft)	-57.8 ft
Embankment Height, H (ft)	12.5 ft

Boring WR2074_012B
Boring on the crest
SPT Ground Elevation Used in Analysis
13.40 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
38.5	-25.1	15	SM	39		120	125	4563.4	3159.4	1330.9	3250.0	1627.6	0.82	1	1	1.00	15.8	5.00	1.20	23.9	0.27	0.86	0.22	1.00	0.71	1.00	1.83
41.0	-27.6	23	SP-SM	7		120	125	4846.0	3286.0	1301.0	3562.5	1784.1	0.80	1	1	1.00	23.7	0.12	1.01	24.0	0.27	0.84	0.22	1.00	0.65	1.00	1.88

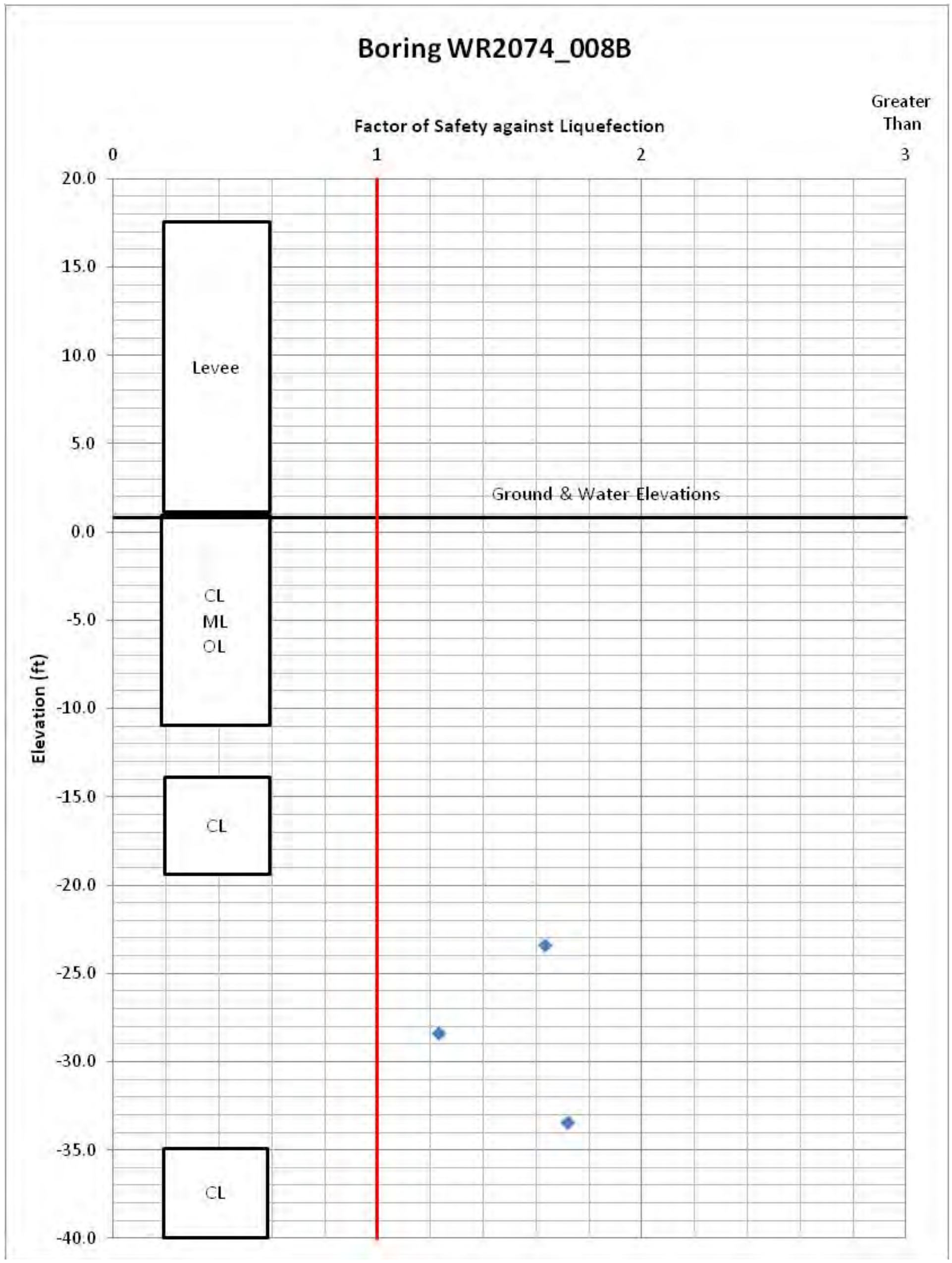


Fig. C-51. Brookside, Station 185+70

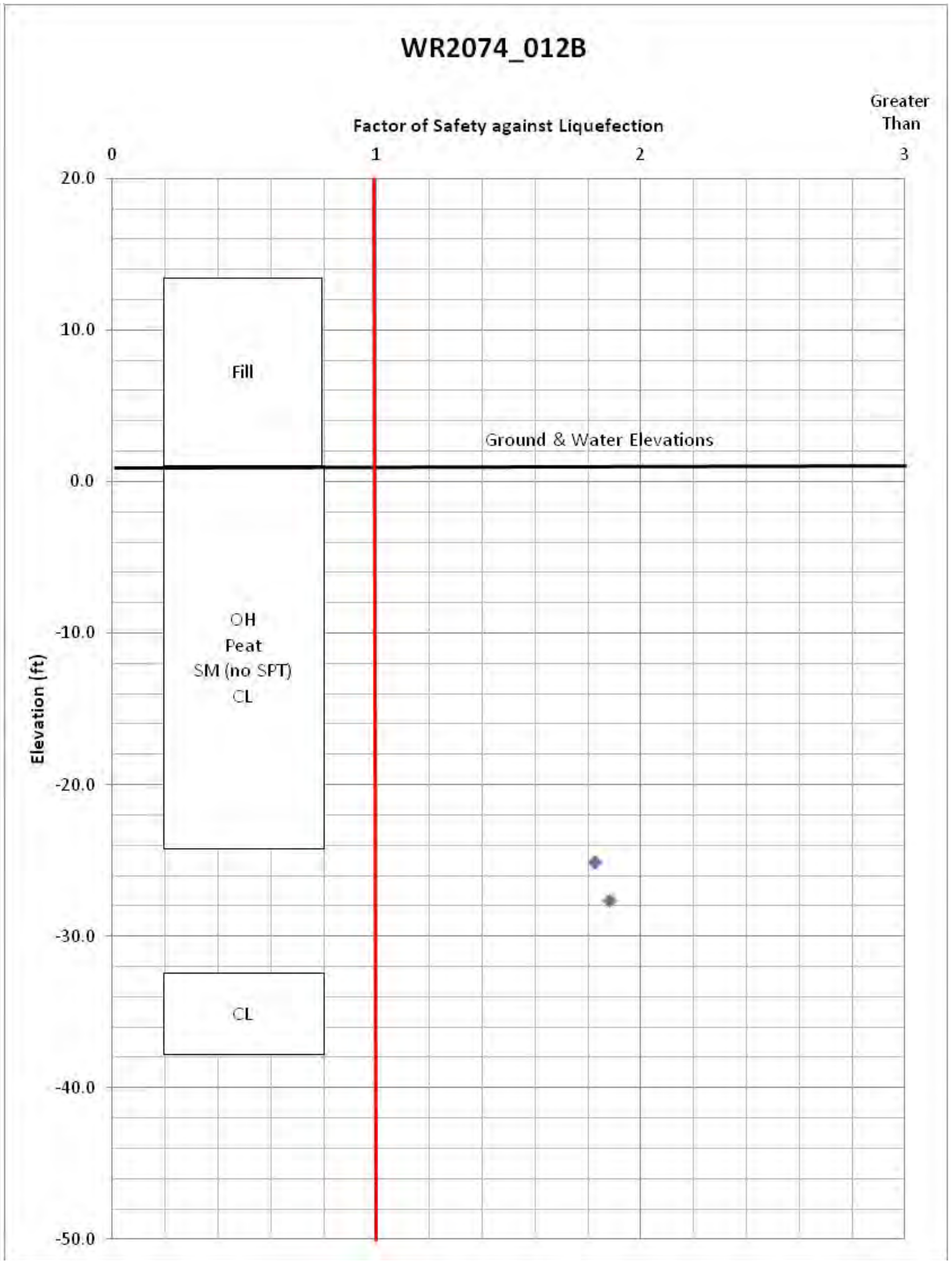


Fig. C-52. Brookside, Station 217+77

LIQUERFACTION TRIGGERING ANALYSIS

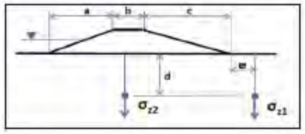
Project: Lower San Joaquin
Study Area: Brookside
Levee Station: 247+31
Boring Number: WR2074_013B

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters

Embankment Crest Elevation (ft)	13.9 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	-1.1 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-1.1 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	-1.1 ft			120.0 pcf	



Surcharge Information

Waterside/Upstream Slope, a (ft)	49.5 ft
Crest Width, b (ft)	17.0 ft
Landside/Downstream Slope, c (ft)	37.5 ft
Dist. of Boring from Levee Toe [1] (ft)	-46.0 ft
Embankment Height, H (ft)	15.0 ft

Boring	WR2074_013B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
13.90 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
31.0	-17.1	25	SM	14		120	125	3631.1	2632.7	1631.1	2000.0	1001.6	0.90	1	1	1.00	28.6	2.20	1.04	32.2	2.00	0.92	0.24	1.00	0.60	1.00	3.00

NOTE
[1] "e" is the distance from landside toe, positive downstream and negative going upstream.
[2] Soil description may be used to estimate fines content where lab testing is not available.
Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.
Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.
[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.
[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for linter (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

LIQUERFACTION TRIGGERING ANALYSIS

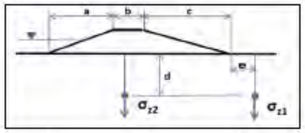
Project: Lower San Joaquin
Study Area: Brookside
Levee Station: 248+41
Boring Number: WR2074_005M

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters

Embankment Crest Elevation (ft)	13.9 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	2.4 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.1 ft	Hammer Efficiency	79	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	2.4 ft			120.0 pcf	



Surcharge Information

Waterside/Upstream Slope, a (ft)	48.3 ft
Crest Width, b (ft)	20.0 ft
Landside/Downstream Slope, c (ft)	23.0 ft
Dist. of Boring from Levee Toe [1] (ft)	-33.0 ft
Embankment Height, H (ft)	11.5 ft

Boring	WR2074_005M
Boring on the crest	
SPT Ground Elevation Used in Analysis	
13.90 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
21.0	-7.1	16	SP-SM	11		120	125	2496.9	2184.9	1331.9	1187.5	594.7	0.98	1	0.95	1.00	19.7	1.21	1.03	21.4	0.23	0.95	0.25	1.00	0.68	1.00	1.42
26.0	-12.1	27	SM	27		120	125	3052.5	2428.5	1262.5	1812.5	907.7	0.93	1	1	1.00	33.2	4.48	1.13	42.0	2.00	0.94	0.24	1.00	0.60	1.00	3.00
31.5	-17.6	16	SP-SM	6		120	125	3651.0	2683.8	1173.5	2500.0	1252.0	0.89	1	1	1.00	18.7	0.03	1.00	18.8	0.20	0.92	0.24	1.00	0.69	1.00	1.27
41.5	-27.6	17	SM	46		120	125	4742.7	3432.3	1015.2	3750.0	1878.0	0.79	1	1	1.00	17.6	5.00	1.20	26.1	0.32	0.84	0.22	1.00	0.70	1.00	2.18
46.0	-32.1	21	SM	13		120	125	5241.0	3930.6	951.0	4312.5	2159.7	0.73	1	1	1.00	20.3	1.89	1.04	22.9	0.26	0.80	0.21	1.00	0.67	0.99	1.84

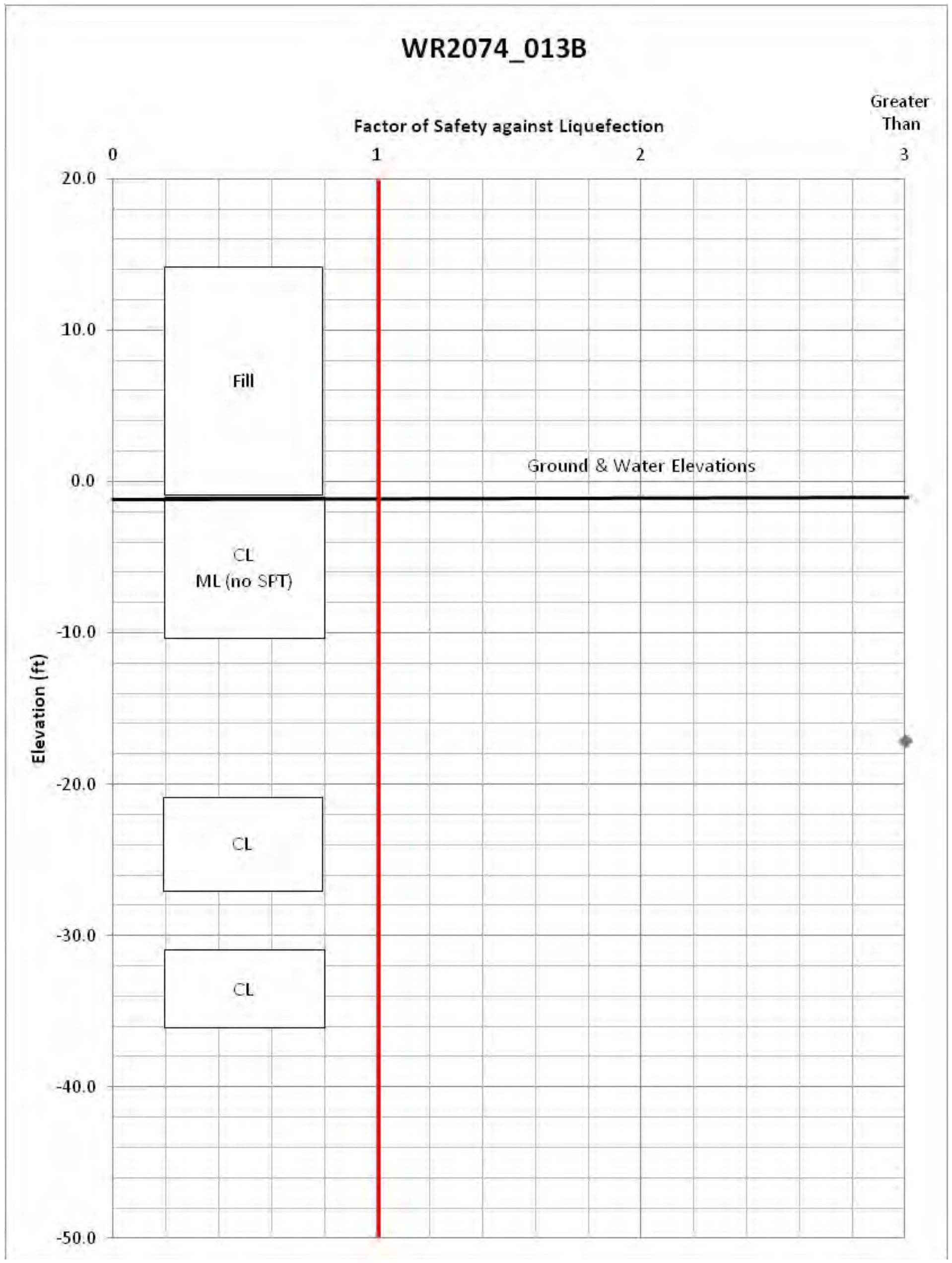


Fig. C-53. Brookside, Station 247+31

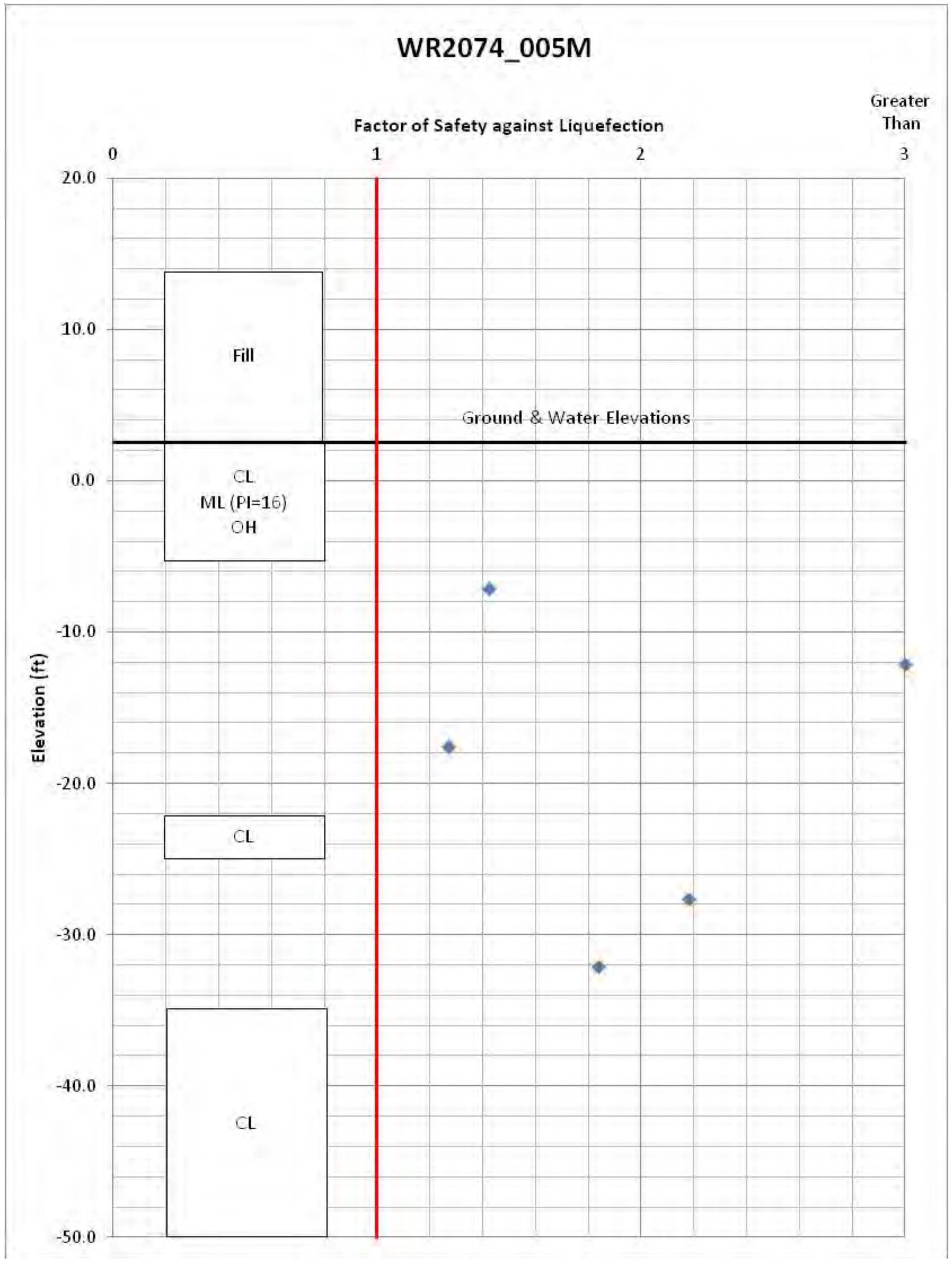


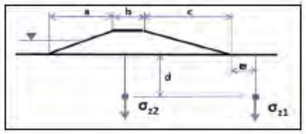
Fig. C-54. Brookside, Station 248+41

LIQUERFACTION TRIGGERING ANALYSIS

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea
Study Area:	Lincoln Village	**Checked by:**	
Levee Station:	5+23	**Date:**	7/23/2013
Boring Number:	WR1608_002B	**Date:**	

Input Parameters

Embankment Crest Elevation (ft)	13.4 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	5.4 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	5.4 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	26.4 ft
Crest Width, b (ft)	9.0 ft
Landside/Downstream Slope, c (ft)	23.2 ft
Dist. of Boring from Levee Toe [1] (ft)	-27.7 ft
Embankment Height, H (ft)	8.0 ft

Boring	WR1608_002B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
13.40 ft	

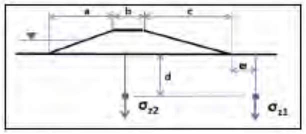
Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Decoription [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
36.5	-23.1	46	SW-SM	5		120	125	4084.8	2768.1	559.3	3562.5	1784.1	0.87	1	1	1.00	51.6	0.00	1.00	51.6	2.00	0.88	0.23	1.00	0.60	1.00	3.00

LIQUERFACTION TRIGGERING ANALYSIS

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea
Study Area:	Lincoln Village	**Checked by:**	
Levee Station:	43+57	**Date:**	7/23/2013
Boring Number:	WR1608_002M	**Date:**	

Input Parameters

Embankment Crest Elevation (ft)	13.4 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	3.3 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	10.1 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	84	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	3.3 ft				120.0 pcf



Surcharge Information	
Waterside/Upstream Slope, a (ft)	31.7 ft
Crest Width, b (ft)	16.0 ft
Landside/Downstream Slope, c (ft)	38.4 ft
Dist. of Boring from Levee Toe [1] (ft)	5.0 ft
Embankment Height, H (ft)	10.1 ft

Boring	WR1608_002M
Boring on waterside or landside field	
SPT Ground Elevation Used in Analysis	
3.30 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Decoription [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
16.5	-13.2	18	SM	16		120	125	2131.3	1432.4	95.3	2062.5	1032.9	1.22	1	0.95	1.00	29.1	2.77	1.05	33.4	2.00	0.96	0.25	1.00	0.60	1.00	3.00
21.0	-17.7	7	SP-SM	27		120	125	2727.5	1747.8	129.0	2625.0	1314.6	1.10	1	0.95	1.00	10.2	4.48	1.13	16.1	0.17	0.95	0.25	1.00	0.77	1.00	1.04
26.5	-23.2	8	SP-SM	8		120	125	3451.6	2422.0	165.6	3312.5	1658.9	0.93	1	1	1.00	10.5	0.30	1.01	10.9	0.12	0.94	0.24	1.00	0.77	1.00	0.75

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

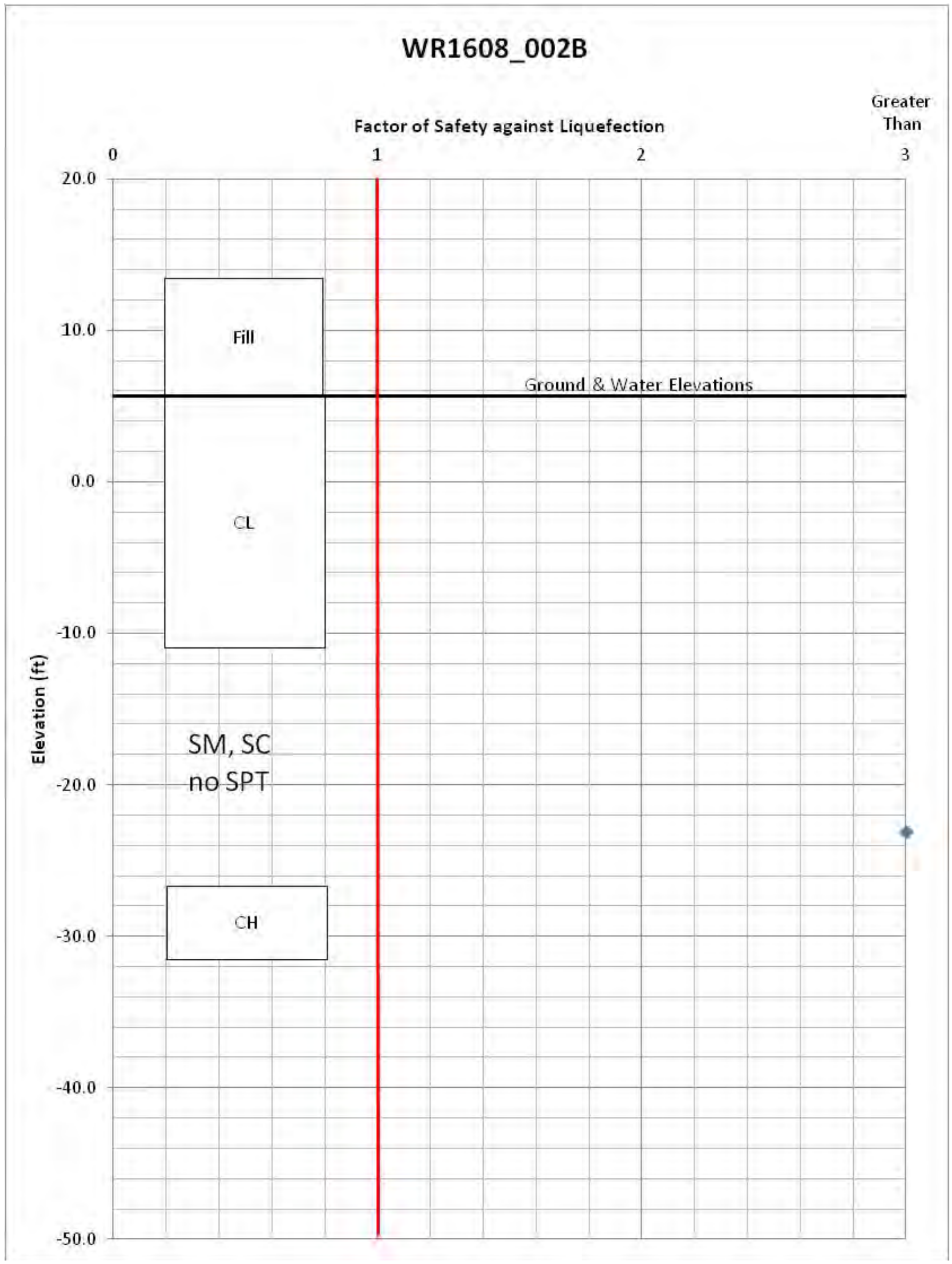


Fig. C-55. Lincoln Village, Station 5+23

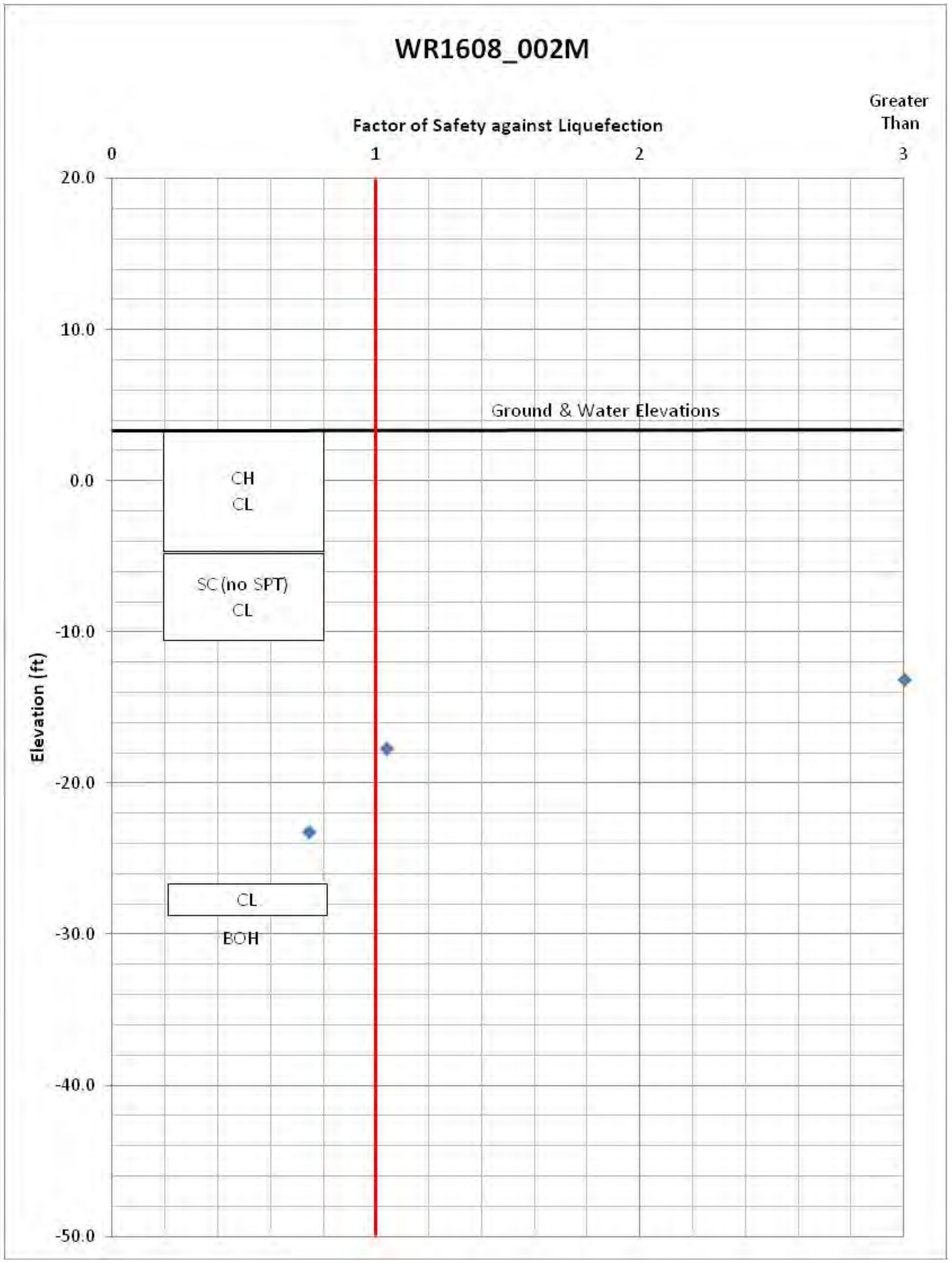


Fig. C-56. Lincoln Village, Station 43+00

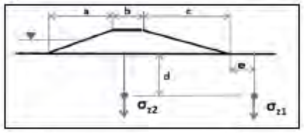
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Lincoln Village
Levee Station: 43+58
Boring Number: WR1608_001M

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	13.3 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	3.3 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	84	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	3.3 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	33.0 ft
Crest Width, b (ft)	16.0 ft
Landside/Downstream Slope, c (ft)	40.0 ft
Dist. of Boring from Levee Toe[1] (ft)	-48.0 ft
Embankment Height, H (ft)	10.0 ft

Boring **WR1608_001M**
Boring on the crest
SPT Ground Elevation Used in Analysis
13.30 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
31.0	-17.7	7	SW-SM	31		120	125	3588.8	2609.1	990.3	2625.0	1314.6	0.90	1	1	1.00	8.8	4.77	1.16	15.0	0.16	0.92	0.24	1.00	0.78	1.00	1.01
36.5	-23.2	13	SW-SM	7		120	125	4198.4	2875.5	912.4	3312.5	1658.9	0.86	1	1	1.00	15.6	0.12	1.01	15.9	0.17	0.88	0.23	1.00	0.71	1.00	1.11

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

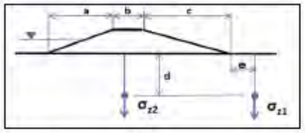
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Lincoln Village
Levee Station: 50+79
Boring Number: WR1608_004B

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	13.4 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	5.4 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	5.4 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	26.4 ft
Crest Width, b (ft)	9.0 ft
Landside/Downstream Slope, c (ft)	20.0 ft
Dist. of Boring from Levee Toe[1] (ft)	-24.5 ft
Embankment Height, H (ft)	8.0 ft

Boring **WR1608_004B**
Boring on the crest
SPT Ground Elevation Used in Analysis
13.40 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
24.0	-10.6	30	SP	4		120	125	2692.2	2155.5	729.2	2000.0	1001.6	0.99	1	0.95	1.00	36.5	0.00	1.00	36.5	2.00	0.94	0.25	1.00	0.60	1.00	3.00
26.5	-13.1	28	SP	4		120	125	2961.4	2268.8	685.9	2312.5	1158.1	0.97	1	1	1.00	34.7	0.00	1.00	34.7	2.00	0.94	0.24	1.00	0.60	1.00	3.00
29.0	-15.6	26	SP	4		120	125	3233.6	2384.9	645.6	2625.0	1314.6	0.94	1	1	1.00	31.2	0.00	1.00	31.2	2.00	0.93	0.24	1.00	0.60	1.00	3.00
32.0	-18.6	19	SM	28		120	125	3564.1	2528.2	601.1	3000.0	1502.4	0.91	1	1	1.00	22.5	4.56	1.14	30.2	2.00	0.91	0.24	1.00	0.66	1.00	3.00
34.5	-21.1	14	SM	28		120	125	3842.8	2650.9	567.3	3312.5	1658.9	0.89	1	1	1.00	16.4	4.56	1.14	23.2	0.26	0.89	0.23	1.00	0.71	1.00	1.69
37.0	-23.6	15	SP-SM	7		120	125	4124.2	2776.4	536.2	3625.0	1815.4	0.87	1	1	1.00	16.6	0.12	1.01	16.8	0.18	0.87	0.23	1.00	0.70	1.00	1.19
39.5	-26.1	13	SP-SM	7		120	125	4408.3	2910.7	507.8	3937.5	1971.9	0.85	1	1	1.00	13.8	0.12	1.01	14.0	0.15	0.85	0.22	1.00	0.73	1.00	1.02

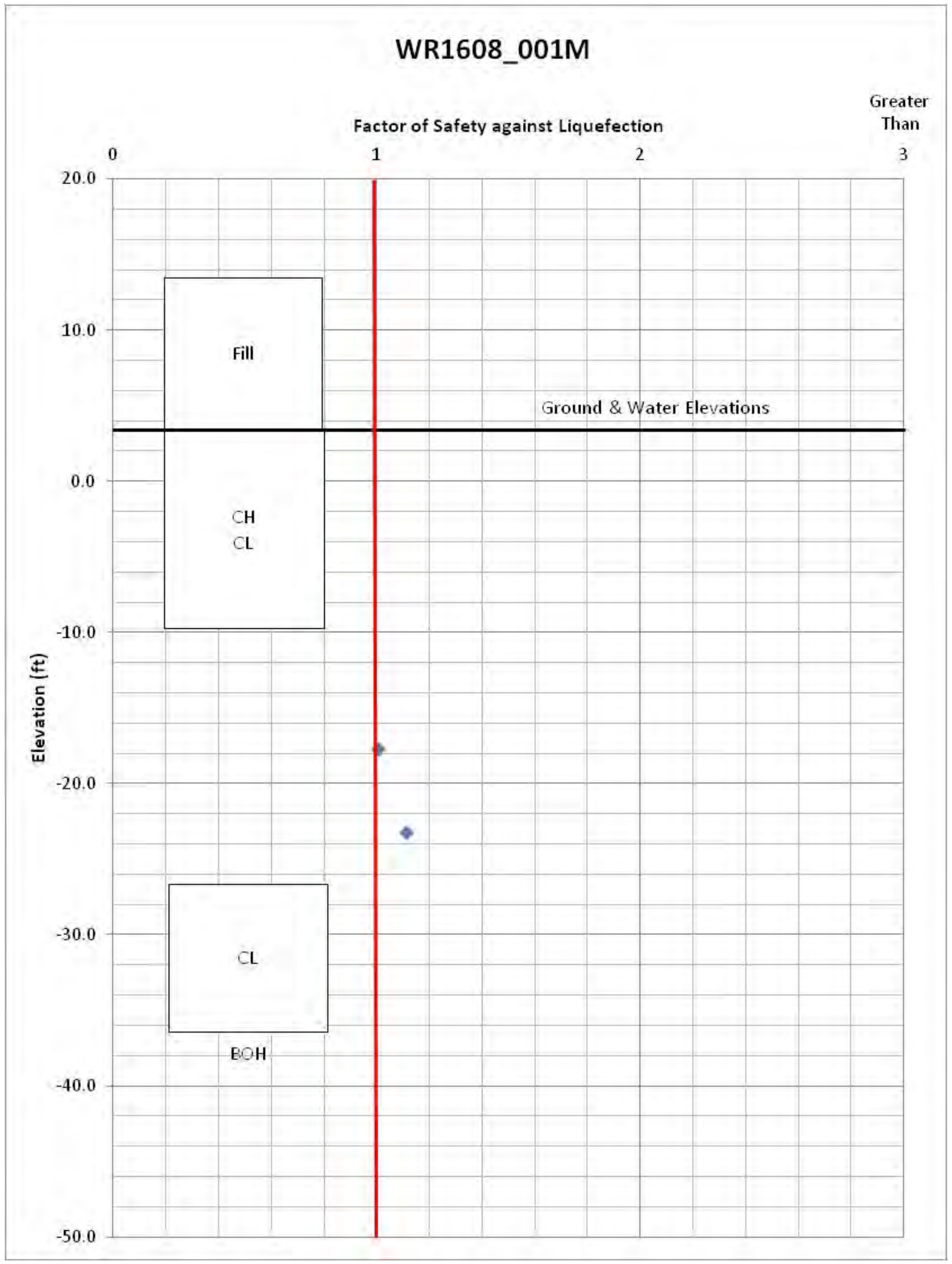
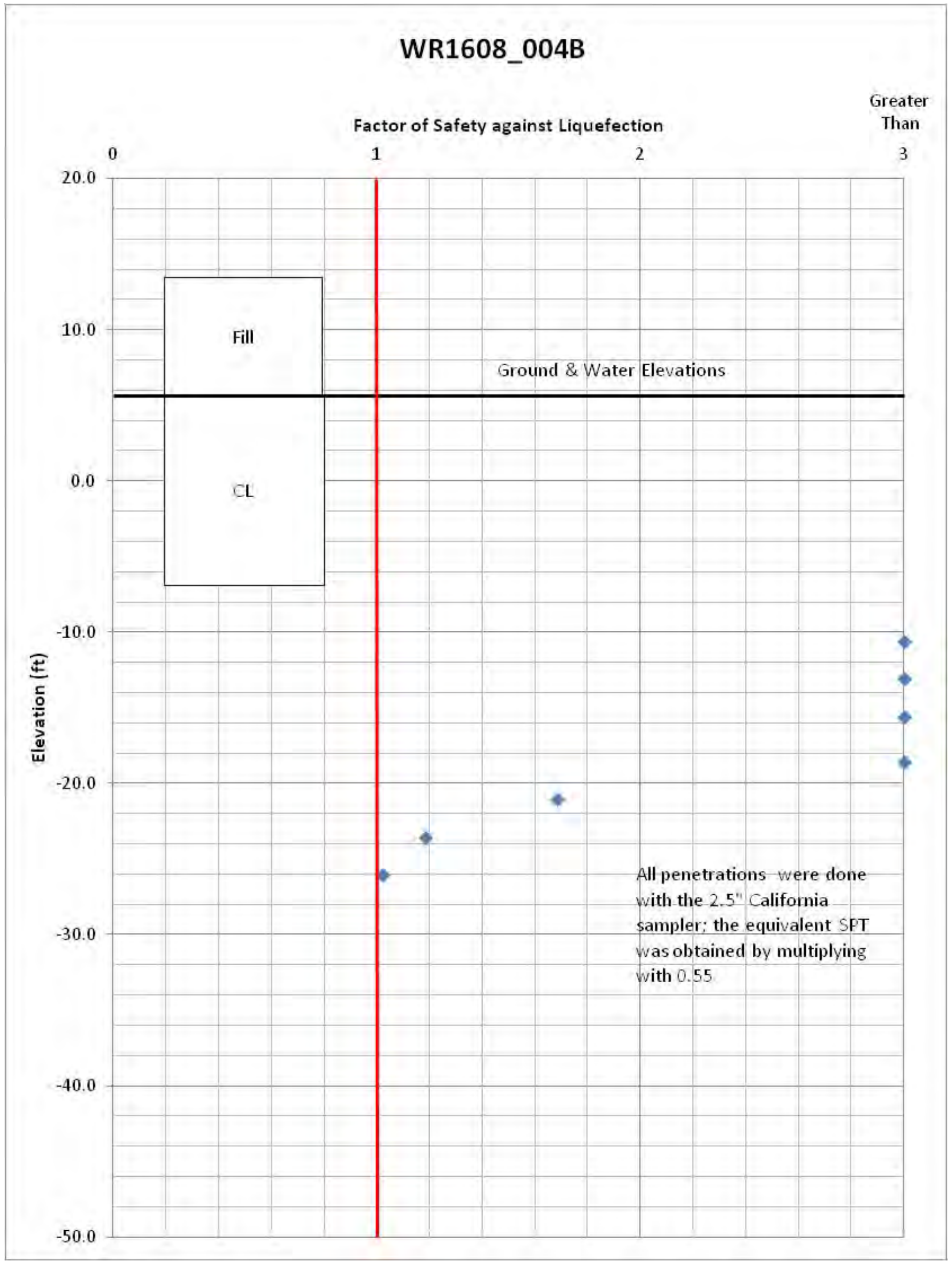


Fig. C-57. Lincoln Village, Station 43+58



All penetrations were done with the 2.5" California sampler; the equivalent SPT was obtained by multiplying with 0.55

Fig. C-58. Lincoln Village, Station 50+79

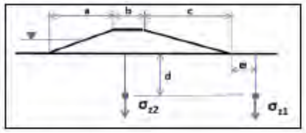
LIQUEFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Lincoln Village
Levee Station: 89+65
Boring Number: WR1608_004M

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	13.5 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	5.7 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	84	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	5.7 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	25.5 ft
Crest Width, b (ft)	22.0 ft
Landside/Downstream Slope, c (ft)	34.0 ft
Dist. of Boring from Levee Toe [1] (ft)	-45.0 ft
Embankment Height, H (ft)	7.8 ft

Boring	**WR1608_004M**
Boring on the crest	
SPT Ground Elevation Used in Analysis	
13.50 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
26.0	-12.5	15	SC	17		120	125	3054.7	2399.5	818.2	2275.0	1139.3	0.94	1	1	1.00	19.7	3.01	1.06	23.9	0.27	0.94	0.24	1.00	0.68	1.00	1.67

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

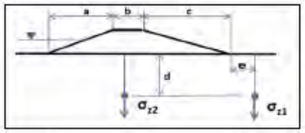
LIQUEFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Lincoln Village
Levee Station: 89+67
Boring Number: WR1608_003M

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	13.3 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	4.8 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	84	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	4.8 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	25.5 ft
Crest Width, b (ft)	22.0 ft
Landside/Downstream Slope, c (ft)	34.0 ft
Dist. of Boring from Levee Toe [1] (ft)	-45.0 ft
Embankment Height, H (ft)	8.5 ft

Boring	**WR1608_003M**
Boring on the crest	
SPT Ground Elevation Used in Analysis	
13.30 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao&Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
26.0	-12.7	19	SC	42		120	125	3053.8	2386.1	900.3	2187.5	1095.5	0.94	1	1	1.00	25.0	5.00	1.20	35.1	2.00	0.94	0.24	1.00	0.64	1.00	3.00

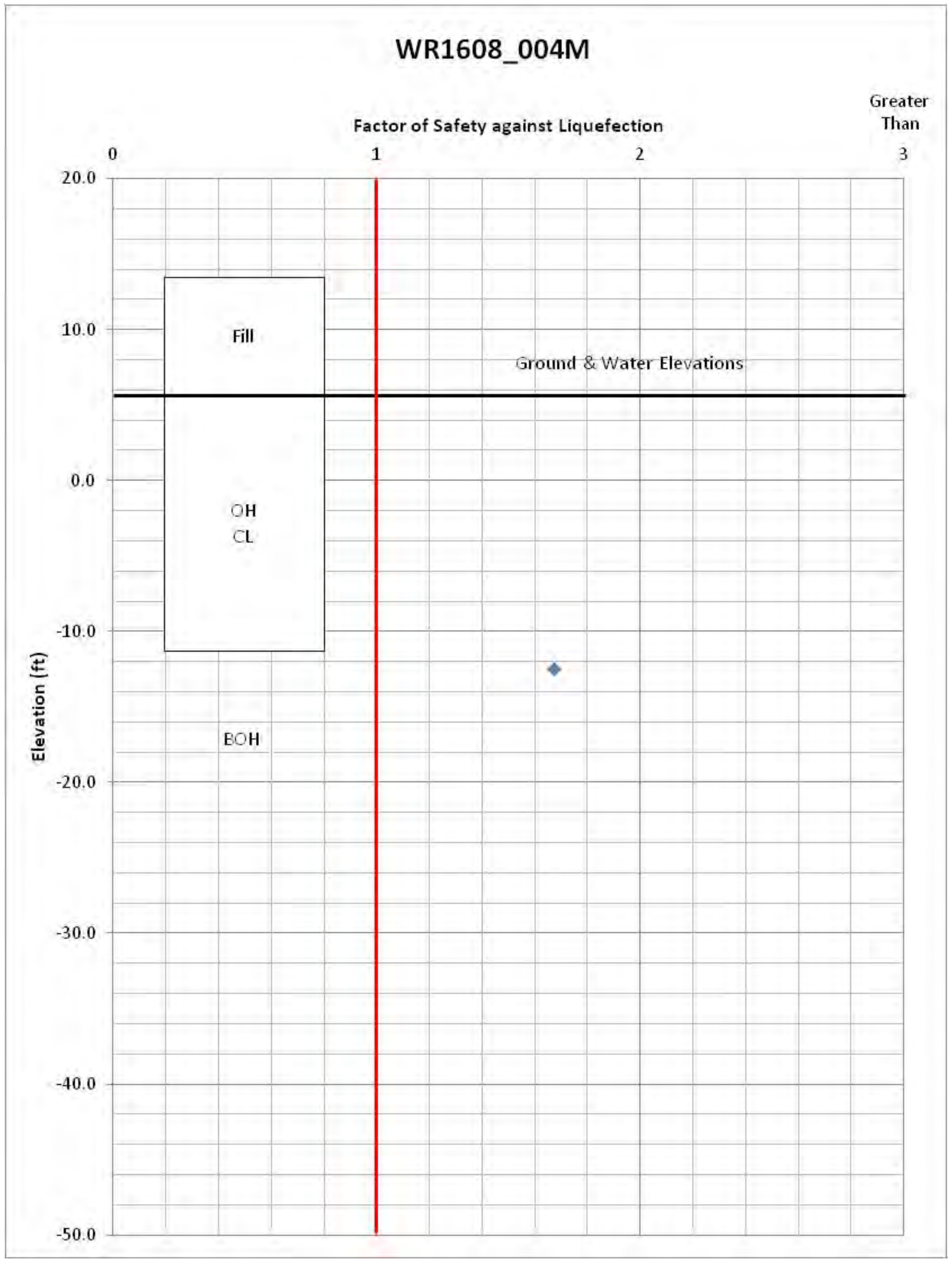


Fig. C-59. Lincoln Village, Station 89+65

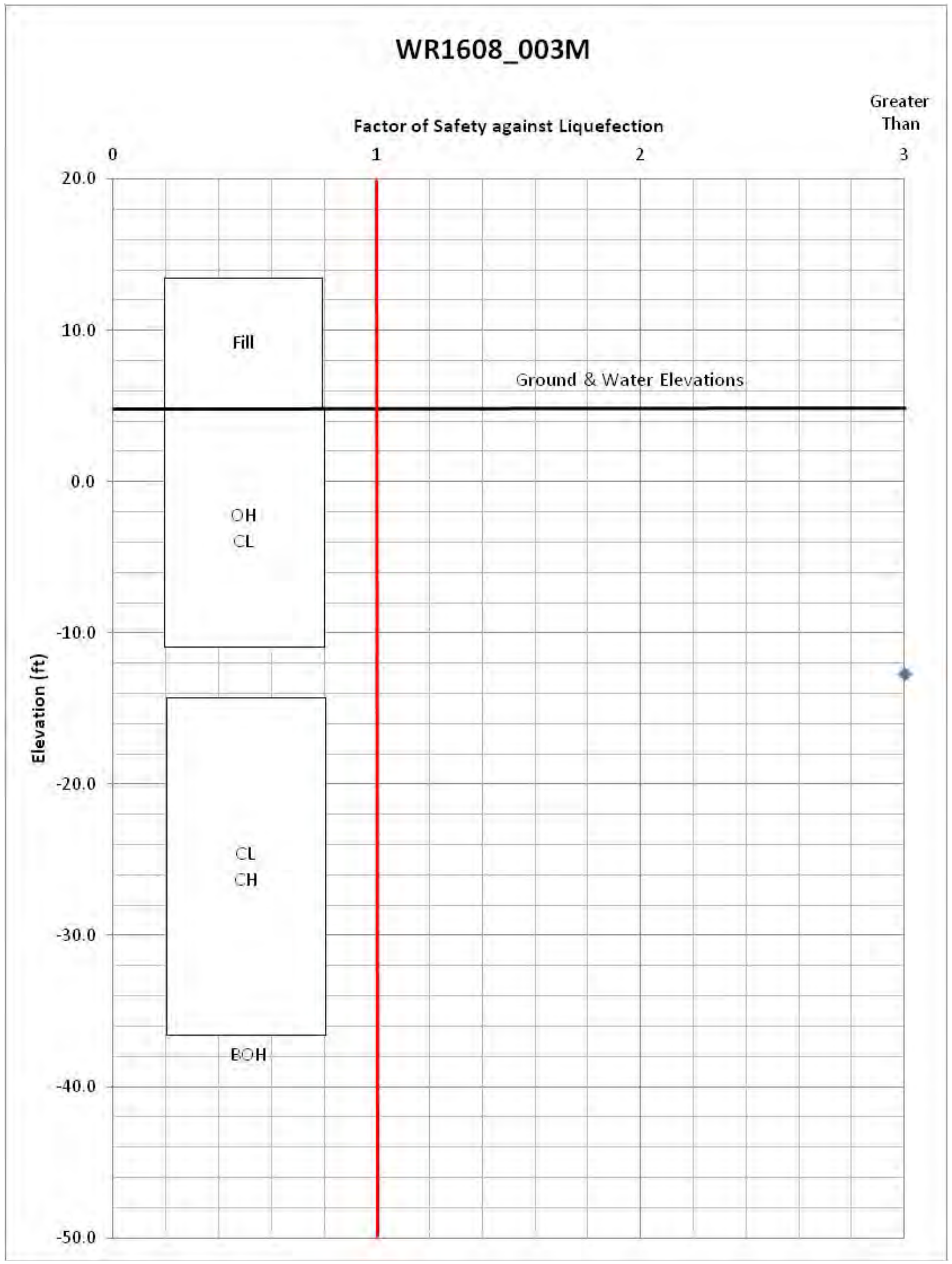


Fig. C-60. Lincoln Village, Station 89+67

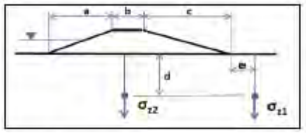
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Lincoln Village
Levee Station: 109+90
Boring Number: WR1608_008B

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	13.0 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	1.0 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	1.0 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	39.6 ft
Crest Width, b (ft)	60.0 ft
Landside/Downstream Slope, c (ft)	30.0 ft
Dist. of Boring from Levee Toe [1] (ft)	-60.0 ft
Embankment Height, H (ft)	12.0 ft

Boring	WR1608_008B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
13.00 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
27.0	-14.0	10	SP-SM	10		120	125	3277.9	2529.1	1417.9	1875.0	939.0	0.91	1	1	1.00	12.3	0.87	1.02	13.4	0.14	0.94	0.24	1.00	0.75	1.00	0.99
29.5	-16.5	25	ML	60		120	125	3579.3	2674.5	1406.8	2187.5	1095.5	0.89	1	1	1.00	28.5	5.00	1.20	39.2	2.00	0.93	0.24	1.00	0.60	1.00	3.00
33.5	-20.5	20	SM	33		120	125	4056.7	2902.3	1384.2	2687.5	1345.9	0.85	1	1	1.00	21.9	4.88	1.18	30.7	2.00	0.90	0.23	1.00	0.66	1.00	3.00
37.5	-24.5	45	SP-SM	11		120	125	4528.7	3124.7	1356.2	3187.5	1596.3	0.82	1	1	1.00	47.5	1.21	1.03	50.0	2.00	0.87	0.23	1.00	0.60	1.00	3.00

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

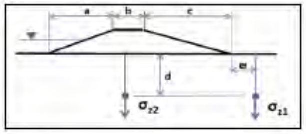
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Lincoln Village
Levee Station: 150+00
Boring Number: WR1608_013B

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	12.7 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	3.2 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	3.2 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	31.4 ft
Crest Width, b (ft)	8.0 ft
Landside/Downstream Slope, c (ft)	23.8 ft
Dist. of Boring from Levee Toe [1] (ft)	-27.8 ft
Embankment Height, H (ft)	9.5 ft

Boring	WR1608_013B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
12.70 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
23.0	-10.3	16	SP-SM	16		120	125	2598.0	2080.1	936.5	1687.5	845.1	1.01	1	0.95	1.00	19.6	2.77	1.05	23.4	0.26	0.95	0.25	1.00	0.68	1.00	1.61
28.5	-15.8	20	SP-SM	7		120	125	3179.1	2318.0	830.1	2375.0	1189.4	0.96	1	1	1.00	25.0	0.12	1.01	25.3	0.30	0.93	0.24	1.00	0.64	1.00	1.84
31.0	-18.3	15	SP-SM	7		120	125	3447.4	2430.3	785.9	2687.5	1345.9	0.93	1	1	1.00	18.4	0.12	1.01	18.7	0.20	0.92	0.24	1.00	0.69	1.00	1.25
33.5	-20.8	18	SP-SM	4		120	125	3718.6	2545.5	744.6	3000.0	1502.4	0.91	1	1	1.00	20.6	0.00	1.00	20.6	0.22	0.90	0.23	1.00	0.67	1.00	1.43

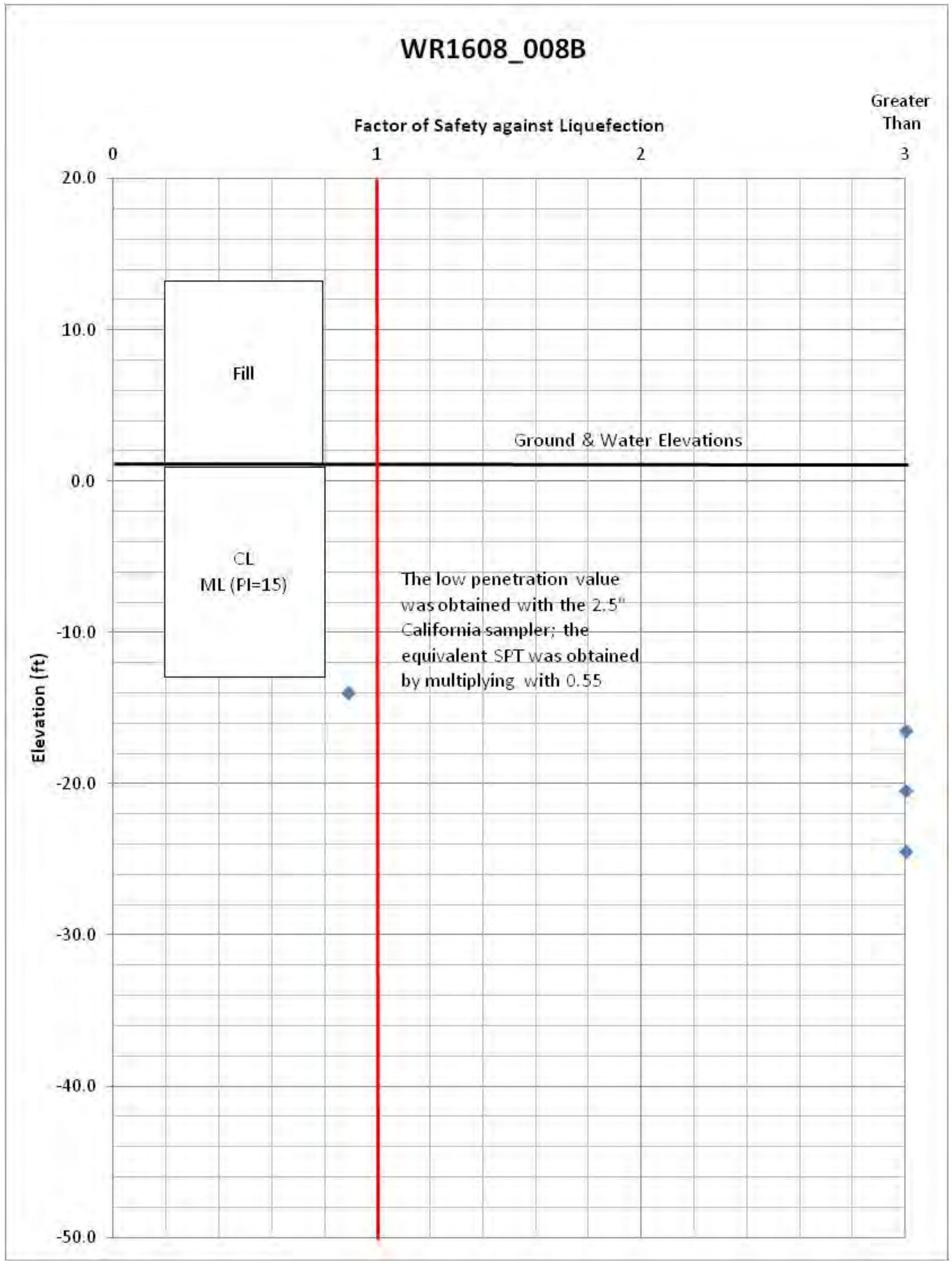


Fig. C-61. Lincoln Village, Station 109+90

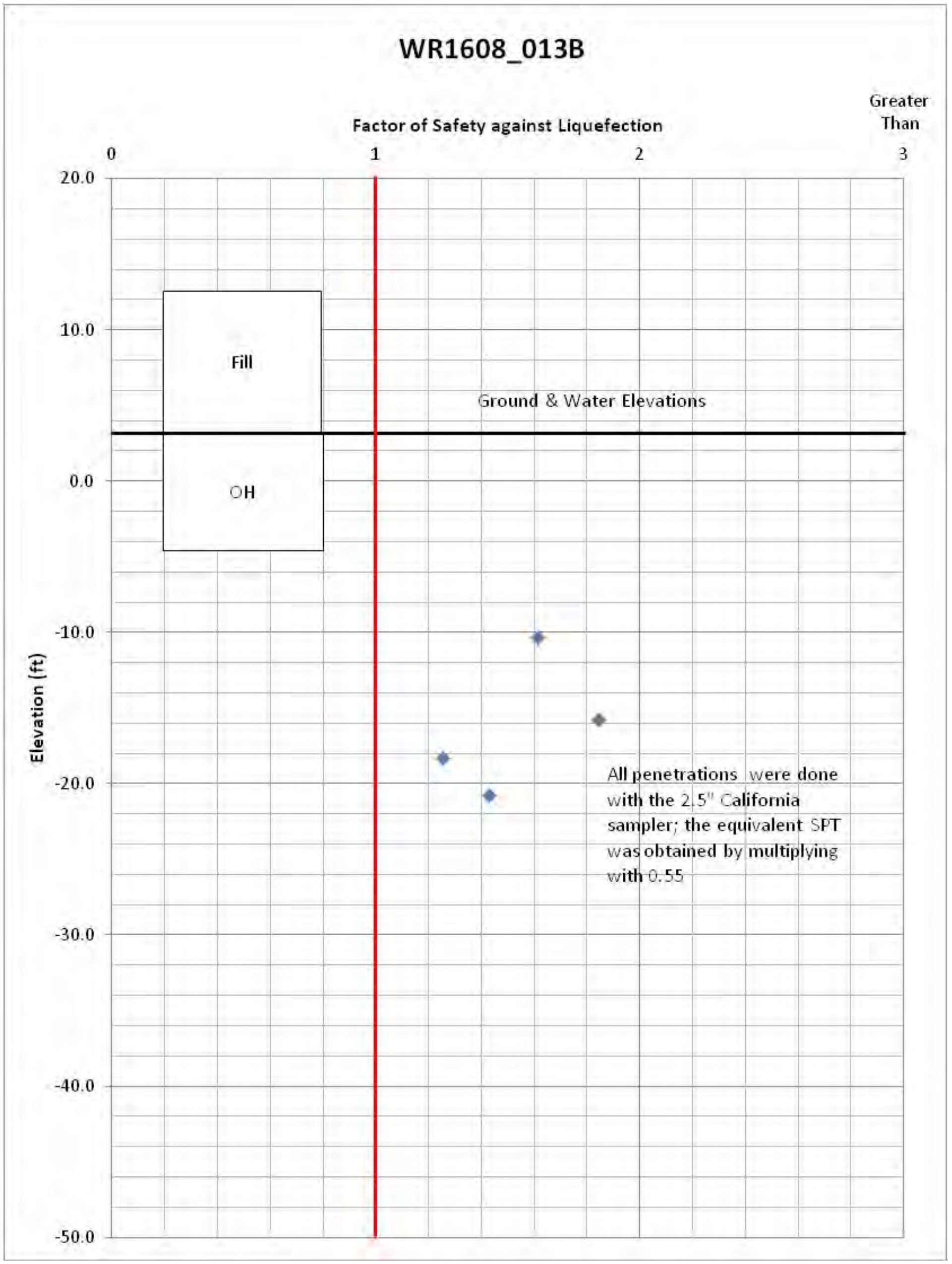


Fig. C-62. Lincoln Village, Station 150+00

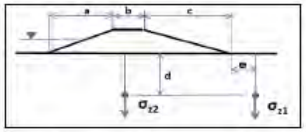
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Lincoln Village
Levee Station: 159+20
Boring Number: WR1608_001B

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	13.1 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	3.1 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	84	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	3.1 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	25.0 ft
Crest Width, b (ft)	9.0 ft
Landside/Downstream Slope, c (ft)	50.0 ft
Dist. of Boring from Levee Toe [1] (ft)	-54.5 ft
Embankment Height, H (ft)	10.0 ft

Boring **WR1608_001B**
Boring on the crest
SPT Ground Elevation Used in Analysis
13.05 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
31.0	-18.0	17	SM	15		120	125	3510.4	2515.1	910.6	2628.4	1314.9	0.92	1	1	1.00	21.8	2.50	1.05	25.4	0.30	0.92	0.24	1.00	0.66	1.00	1.88
36.5	-23.5	14	SM	39		120	125	4115.7	2777.3	828.5	3315.9	1659.2	0.87	1	1	1.00	17.1	5.00	1.20	25.5	0.30	0.88	0.23	1.00	0.70	1.00	1.99
41.5	-28.5	13	SP-SM	12		120	125	4674.6	3024.1	762.4	3940.9	1972.2	0.84	1	1	1.00	15.2	1.55	1.03	17.3	0.18	0.84	0.22	1.00	0.72	1.00	1.27

NOTE
[1] "e" is the distance from landside toe, positive downstream and negative going upstream.
[2] Soil description may be used to estimate fines content where lab testing is not available.
Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.
Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.
[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.
[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

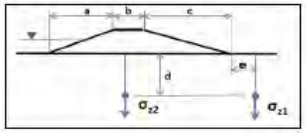
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Lincoln Village
Levee Station: 159+41
Boring Number: WR1608_009B

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	13.4 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	4.1 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	4.1 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	28.1 ft
Crest Width, b (ft)	14.0 ft
Landside/Downstream Slope, c (ft)	37.4 ft
Dist. of Boring from Levee Toe [1] (ft)	-44.4 ft
Embankment Height, H (ft)	9.4 ft

Boring **WR1608_009B**
Boring on the crest
SPT Ground Elevation Used in Analysis
13.40 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
33.5	-20.1	12	ML	63		120	125	3833.4	2703.9	844.9	3022.1	1512.0	0.88	1	1	1.00	13.1	5.00	1.20	20.7	0.22	0.90	0.23	1.00	0.74	1.00	1.43
36.0	-22.6	13	ML	63		120	125	4110.9	2825.4	809.9	3334.6	1668.5	0.87	1	1	1.00	14.7	5.00	1.20	22.6	0.25	0.88	0.23	1.00	0.72	1.00	1.64
38.5	-25.1	12	ML	63		120	125	4390.0	2948.5	776.5	3647.1	1825.0	0.85	1	1	1.00	12.5	5.00	1.20	20.0	0.22	0.86	0.22	1.00	0.74	1.00	1.45
41.0	-27.6	15	SM	11		120	125	4670.8	3073.3	744.8	3959.6	1981.5	0.83	1	1	1.00	15.8	1.21	1.03	17.4	0.19	0.84	0.22	1.00	0.71	1.00	1.27
43.5	-30.1	17	SM	13		120	125	4953.3	3199.8	714.8	4272.1	2138.0	0.81	1	1	1.00	17.2	1.89	1.04	19.7	0.21	0.82	0.21	1.00	0.70	1.00	1.49

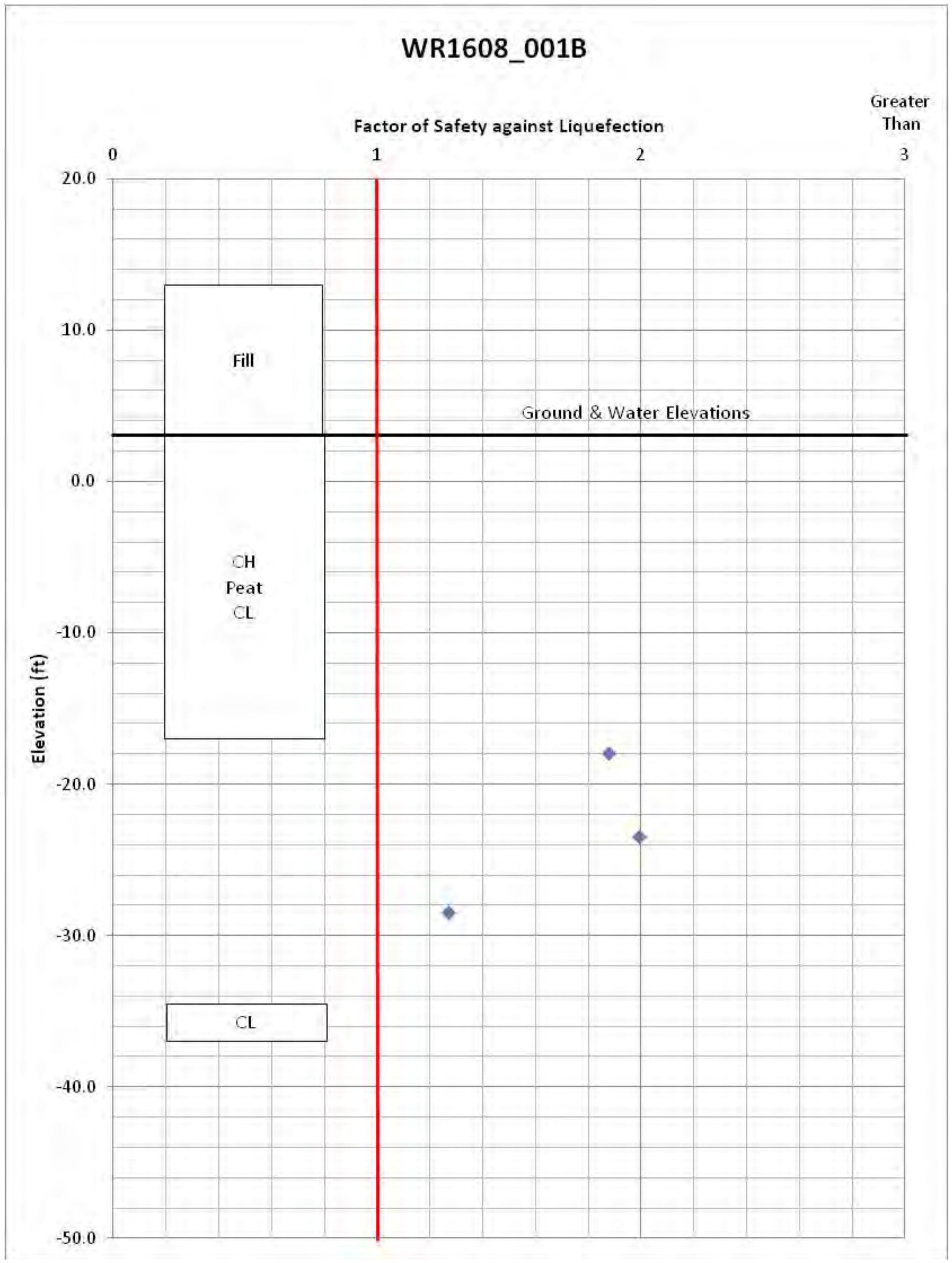
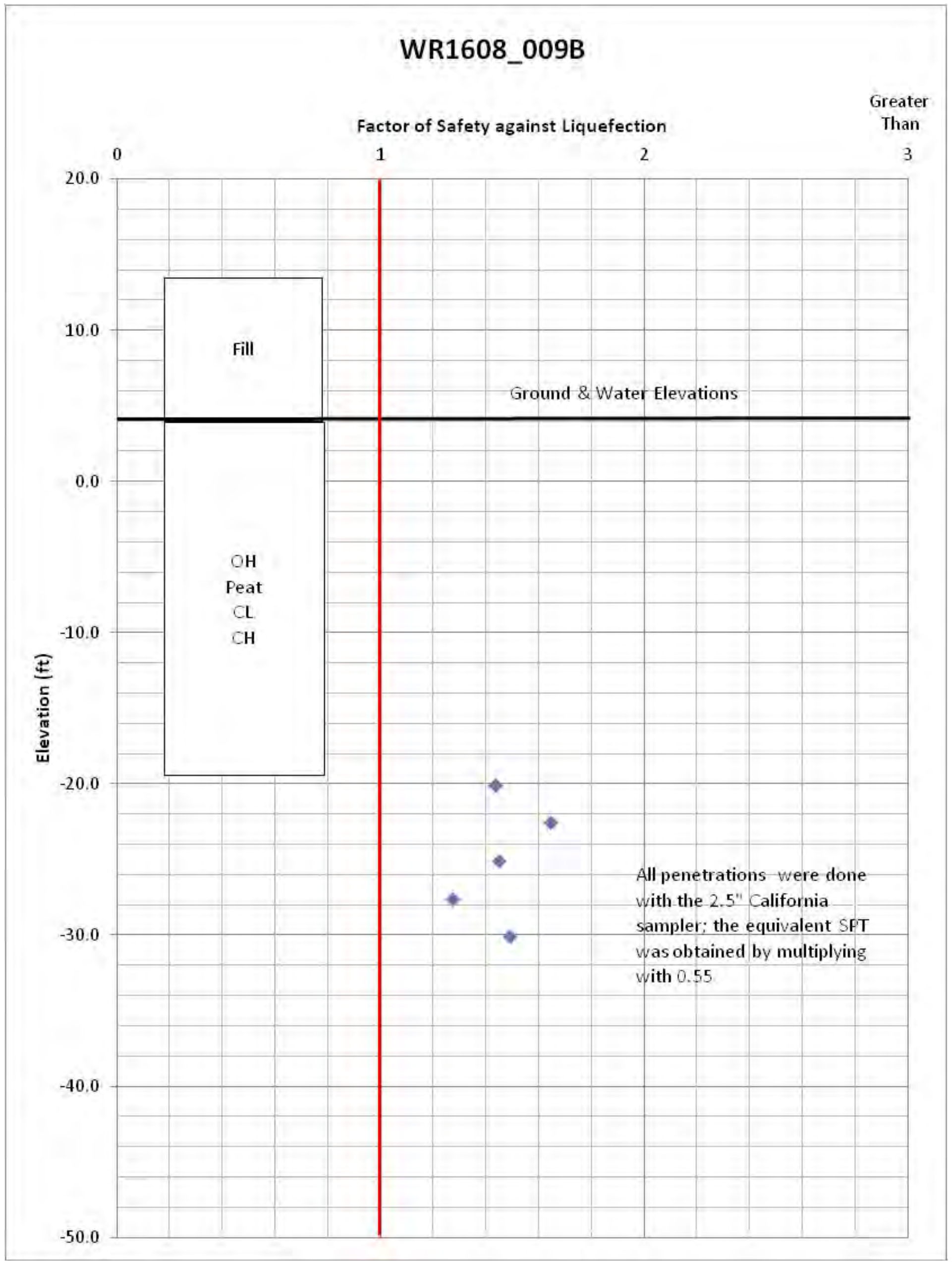


Fig. C-63. Lincoln Village, Station 159+20



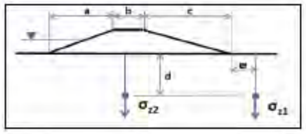
All penetrations were done with the 2.5" California sampler; the equivalent SPT was obtained by multiplying with 0.55

Fig. C-64. Lincoln Village, Station 159+41

LIQUERFACTION TRIGGERING ANALYSIS

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea
Study Area:	Lincoln Village	**Checked by:**	
Levee Station:	159+48	**Date:**	7/23/2013
Boring Number:	WR1608_010B	**Date:**	

Input Parameters					
Embankment Crest Elevation (ft)	13.4 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	3.7 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	9.7 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	3.7 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	29.1 ft
Crest Width, b (ft)	9.0 ft
Landside/Downstream Slope, c (ft)	38.8 ft
Dist. of Boring from Levee Toe[1] (ft)	5.0 ft
Embankment Height, H (ft)	9.7 ft

Boring **WR1608_010B**
Boring on waterside or landside field
SPT Ground Elevation Used in Analysis
3.70 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
21.0	-17.3	8	SP-SM	10		120	125	2717.2	1762.5	120.7	2625.0	1314.6	1.10	1	0.95	1.00	10.7	0.87	1.02	11.8	0.13	0.95	0.25	1.00	0.76	1.00	0.79
24.0	-20.3	11	SM	23		120	125	3111.1	1969.2	139.6	3000.0	1502.4	1.04	1	0.95	1.00	13.9	4.06	1.10	19.4	0.21	0.94	0.25	1.00	0.73	1.00	1.27
30.0	-26.3	34	SM	12		120	125	3893.7	2583.3	172.2	3750.0	1878.0	0.91	1	1	1.00	39.5	1.55	1.03	42.3	2.00	0.93	0.24	1.00	0.60	1.00	3.00
33.0	-29.3	31	SM	8		120	125	4282.3	2971.9	185.8	4125.0	2065.8	0.84	1	1	1.00	33.6	0.30	1.01	34.3	2.00	0.91	0.23	1.00	0.60	1.00	3.00
36.0	-32.3	26	SM	8		120	125	4669.2	3358.8	197.7	4500.0	2253.6	0.79	1	1	1.00	26.5	0.30	1.01	27.1	0.34	0.88	0.23	1.00	0.63	0.98	2.19

NOTE

[1] "e" is the distance from landside toe, positive downstream and negative going upstream.

[2] Soil description may be used to estimate fines content where lab testing is not available.

Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.

Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.

[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.

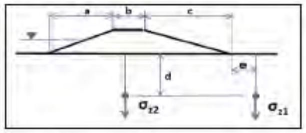
[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

LIQUERFACTION TRIGGERING ANALYSIS

Project:	Lower San Joaquin	**Prepared by:**	Vlad Perlea
Study Area:	Lincoln Village	**Checked by:**	
Levee Station:	164+99	**Date:**	7/23/2013
Boring Number:	WR1608_011B	**Date:**	

Input Parameters					
Embankment Crest Elevation (ft)	13.6 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	3.6 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	77	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	3.6 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	33.0 ft
Crest Width, b (ft)	14.0 ft
Landside/Downstream Slope, c (ft)	29.0 ft
Dist. of Boring from Levee Toe[1] (ft)	-36.0 ft
Embankment Height, H (ft)	10.0 ft

Boring **WR1608_011B**
Boring on the crest
SPT Ground Elevation Used in Analysis
13.60 ft

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR^3	K_α	f parameter	K_σ	FS against Liquefaction
24.5	-10.9	18	ML	38		120	125	2836.1	2260.8	1051.6	1812.5	907.7	0.96	1	0.95	1.00	21.3	5.00	1.20	30.6	2.00	0.94	0.24	1.00	0.67	1.00	3.00
27.0	-13.4	8	ML	38		120	125	3106.2	2394.8	1009.2	2125.0	1064.2	0.94	1	1	1.00	10.0	5.00	1.20	16.9	0.18	0.94	0.24	1.00	0.77	1.00	1.11
29.5	-15.9	11	ML	70		120	125	3376.5	2509.1	967.0	2437.5	1220.7	0.92	1	1	1.00	13.0	5.00	1.20	20.6	0.22	0.93	0.24	1.00	0.74	1.00	1.38
32.0	-18.4	13	ML	35		120	125	3647.8	2624.5	925.8	2750.0	1377.2	0.90	1	1	1.00	15.2	5.00	1.20	23.3	0.26	0.91	0.24	1.00	0.72	1.00	1.65
34.5	-20.9	23	ML	35		120	125	3920.6	2741.2	886.1	3062.5	1533.7	0.88	1	1	1.00	26.0	5.00	1.20	36.3	2.00	0.89	0.23	1.00	0.63	1.00	3.00
37.0	-23.4	20	SP-SM	10		120	125	4195.1	2859.8	848.1	3375.0	1690.2	0.86	1	1	1.00	21.9	0.87	1.02	23.2	0.26	0.87	0.23	1.00	0.66	1.00	1.72
40.0	-26.4	20	SP-SM	8		120	125	4527.0	3004.5	805.0	3750.0	1878.0	0.84	1	1	1.00	21.9	0.30	1.01	22.5	0.25	0.85	0.22	1.00	0.66	1.00	1.70
42.0	-28.4	4	ML	90		120	125	4749.8	3221.0	777.8	4000.0	2003.2	0.81	1	1	1.00	4.0	5.00	1.20	9.8	0.11	0.83	0.22	1.00	0.80	1.00	0.77
44.5	-30.9	19	SP	6		120	125	5030.1	3501.3	745.6	4312.5	2159.7	0.78	1	1	1.00	19.2	0.03	1.00	19.3	0.21	0.81	0.21	1.00	0.68	0.99	1.47
47.0	-33.4	15	SP	6		120	125	5312.2	3783.4	715.2	4625.0	2316.2	0.75	1	1	1.00	14.8	0.03	1.00	14.9	0.16	0.79	0.21	1.00	0.72	0.98	1.13
50.0	-36.4	21	SP	6		120	125	5653.1	4124.3	681.1	5000.0	2504.0	0.72	1	1	1.00	19.7	0.03	1.00	19.8	0.21	0.77	0.20	1.00	0.68	0.95	1.52

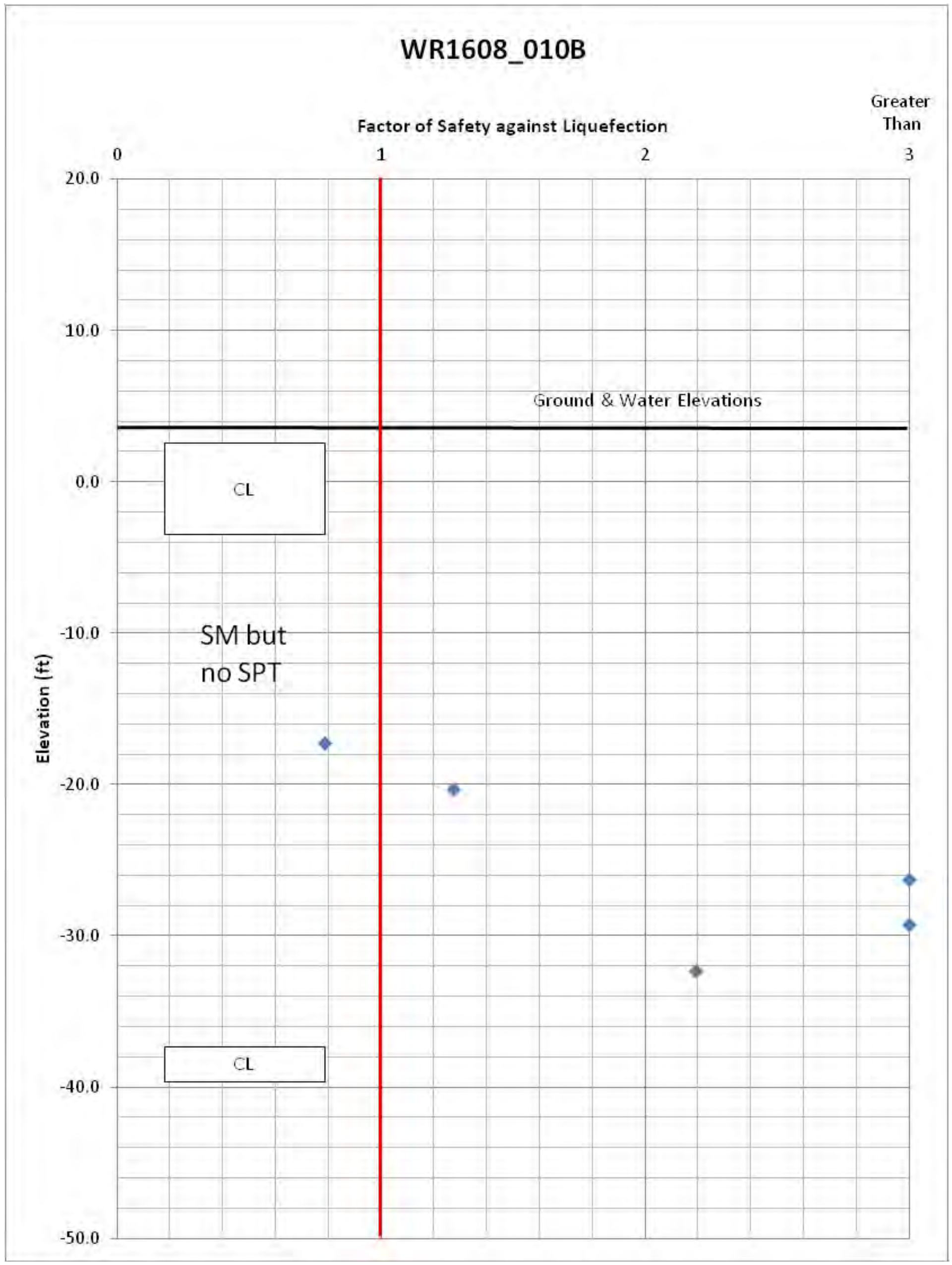


Fig. C-65. Lincoln Village, Station 159+48

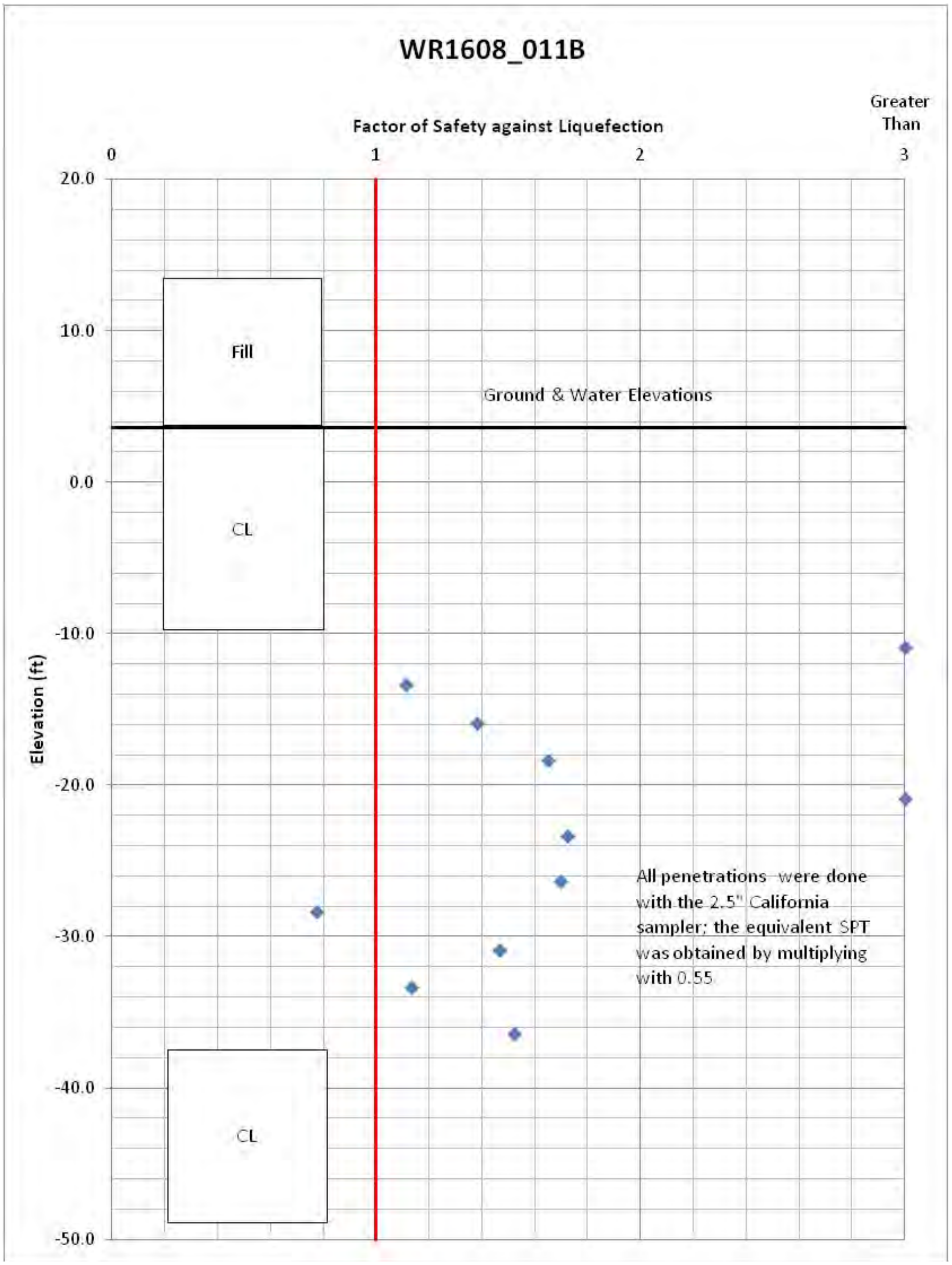


Fig. C-66. Lincoln Village, Station 164+99

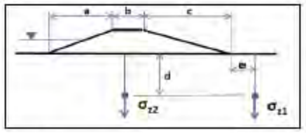
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Lincoln Village
Levee Station: 142+28
Boring Number: WR1608_005M

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	12.7 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	4.9 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	84	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	4.9 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	25.7 ft
Crest Width, b (ft)	12.0 ft
Landside/Downstream Slope, c (ft)	25.7 ft
Dist. of Boring from Levee Toe [1] (ft)	-31.7 ft
Embankment Height, H (ft)	7.8 ft

Boring	WR1608_005M
Boring on the crest	
SPT Ground Elevation Used in Analysis	
12.70 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
26.0	-13.3	11	SC	21		120	125	2967.9	2262.8	727.4	2275.0	1139.3	0.97	1	1	1.00	14.9	3.78	1.09	20.0	0.21	0.94	0.24	1.00	0.72	1.00	1.32
31.0	-18.3	8	ML	92		120	125	3520.7	2503.6	655.2	2900.0	1452.3	0.92	1	1	1.00	10.3	5.00	1.20	17.4	0.18	0.92	0.24	1.00	0.77	1.00	1.16
41.0	-28.3	11	SM	14		120	125	4651.2	3028.8	535.7	4150.0	2078.3	0.84	1	1	1.00	12.9	2.20	1.04	15.6	0.17	0.84	0.22	1.00	0.74	1.00	1.14
46.0	-33.3	49	GP	5		120	125	5228.3	3605.9	487.8	4775.0	2391.3	0.77	1	1	1.00	52.6	0.00	1.00	52.6	2.00	0.80	0.21	1.00	0.60	0.95	3.00

NOTE
[1] "e" is the distance from landside toe, positive downstream and negative going upstream.
[2] Soil description may be used to estimate fines content where lab testing is not available.
Based on Youd et. Al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Worshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and Geoenvironmental Engineering, October 2001.
Surcharge from embankment calculation is presented in Poulos & Davis (1978) which based on Boussinesq formulas for stresses generated by infinite length trapezoidal loading on elastic half-space.
[3] CSR is calculated without consideration of the influence of embankment to reflect free-field condition consistent with the PGA used.
[4] It is conservative to answer "No" if unsure about sampling method; answering "Yes" implies that sampler has room for liner (1.5-inch inside diameter) but the liner is not inserted.

Updated April 2013

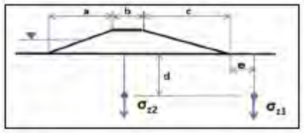
LIQUERFACTION TRIGGERING ANALYSIS

Project: Lower San Joaquin
Study Area: Lincoln Village
Levee Station: 201+51
Boring Number: WCNBFM_001B

Prepared by: Vlad Perlea
Checked by:

Date: 7/23/2013
Date:

Input Parameters					
Embankment Crest Elevation (ft)	13.0 ft	Rod Length Above GS. (ft)	7	Magnitude, M	6.4
Base Elevation (ft)	6.6 ft	Sampler without Liner? (Y/N)	n	PGA (g's)	0.2
Height below Crest of Embankment (ft)	0.0 ft	Borehole Dia. (inch)	4.5		
Groundwater Elevation during Drilling (ft)	-2.0 ft	Hammer Efficiency	84	Assumed Embankment UW (pcf)	
Groundwater Elevation for Analysis (ft)	6.6 ft			120.0 pcf	



Surcharge Information	
Waterside/Upstream Slope, a (ft)	12.9 ft
Crest Width, b (ft)	8.0 ft
Landside/Downstream Slope, c (ft)	14.2 ft
Dist. of Boring from Levee Toe [1] (ft)	-18.2 ft
Embankment Height, H (ft)	6.5 ft

Boring	WCNBFM_001B
Boring on the crest	
SPT Ground Elevation Used in Analysis	
13.00 ft	

Depth (ft)	Elevation (ft)	Field Blow Count, N	USCS Soil Type/Description [2]	Fines Content (%<#200)	Flag for Analysis "Clay" or "Unsaturated"	Wet Unit Weight (pcf)	Saturated Unit Weight (pcf)	Total Overburden Pressure during Drilling (psf)	Effective Overburden Pressure during Drilling (psf)	Surcharge Influence during Drilling (psf)	Total Overburden Pressure for Analysis (psf)	Effective Overburden Pressure for Analysis (psf)	Overburden Correction Factor, C_N [Liao&Whitman]	C_B	C_R	C_S	$N_{1,60}$ [Liao& Whitman]	Alpha	Beta	$(N_{1,60})_{cs}$ [Liao&Whitman]	$CRR_{7.5}$	r_d	CSR[3]	K_α	f parameter	K_σ	FS against Liquefaction
16.0	-3.0	9	SP-SM	34		120	125	1780.1	1717.7	629.1	1197.1	598.1	1.11	1	0.95	1.00	13.3	4.93	1.19	20.7	0.22	0.96	0.25	1.00	0.74	1.00	1.34
21.0	-8.0	14	SP-SM	17		120	125	2299.6	1925.2	523.6	1822.1	911.1	1.05	1	0.95	1.00	19.5	3.01	1.06	23.7	0.27	0.95	0.25	1.00	0.68	1.00	1.63
26.0	-13.0	13	SP-SM	17		120	125	2840.1	2153.7	439.1	2447.1	1224.1	0.99	1	1	1.00	18.0	3.01	1.06	22.1	0.24	0.94	0.24	1.00	0.69	1.00	1.50
31.0	-18.0	7	SP-SM	18		120	125	3399.9	2401.5	373.9	3072.1	1537.1	0.94	1	1	1.00	9.2	3.23	1.07	13.0	0.14	0.92	0.24	1.00	0.78	1.00	0.88
36.0	-23.0	8	SP-SM	10		120	125	3974.6	2976.2	323.6	3697.1	1850.1	0.84	1	1	1.00	9.4	0.87	1.02	10.5	0.12	0.88	0.23	1.00	0.78	1.00	0.77
41.0	-28.0	31	SW-SM	7		120	125	4560.2	3561.8	284.2	4322.1	2163.1	0.77	1	1	1.00	33.5	0.12	1.01	33.9	2.00	0.84	0.22	1.00	0.60	0.99	3.00

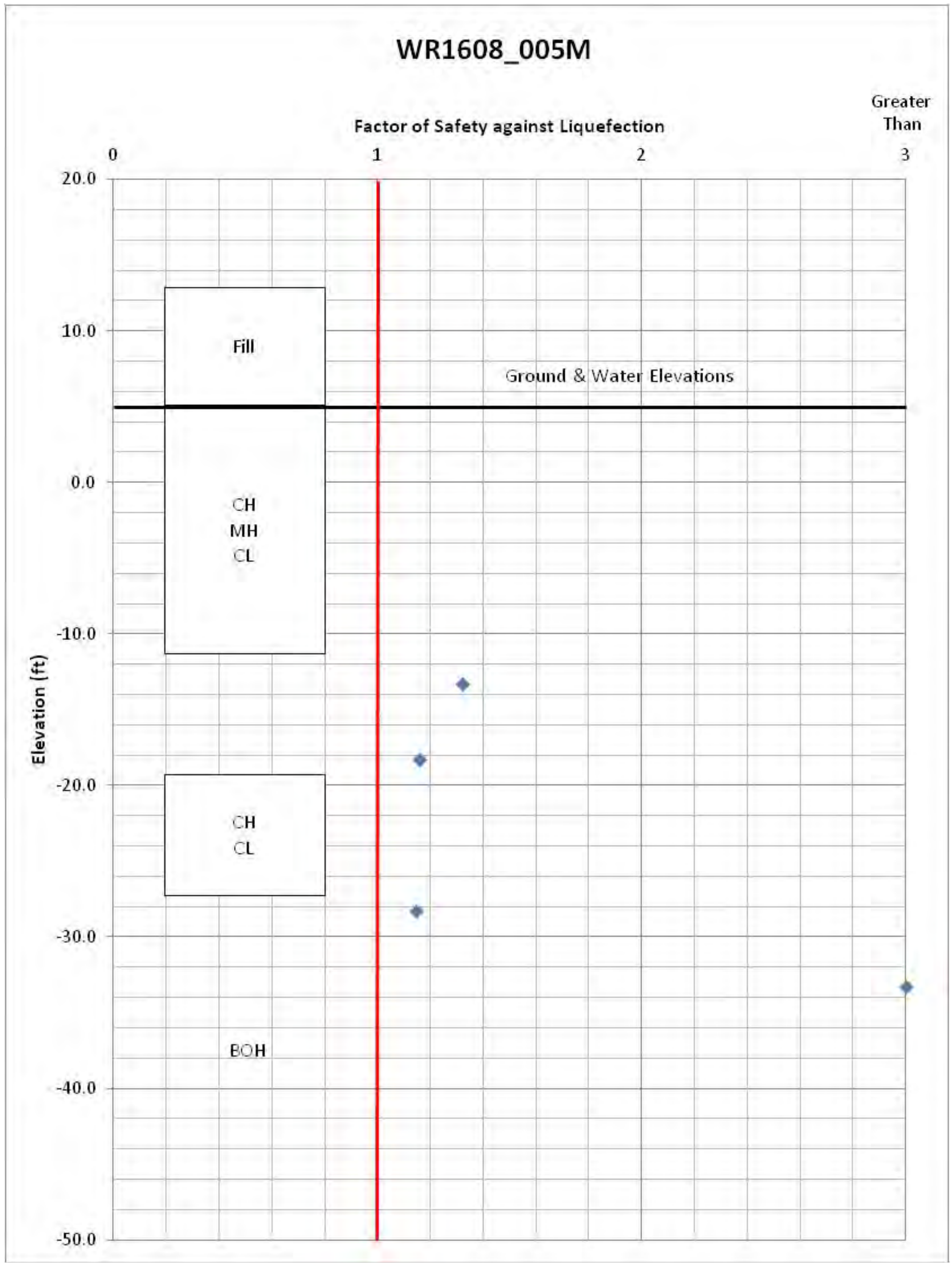


Fig. C-67. Lincoln Village, Station 142+28

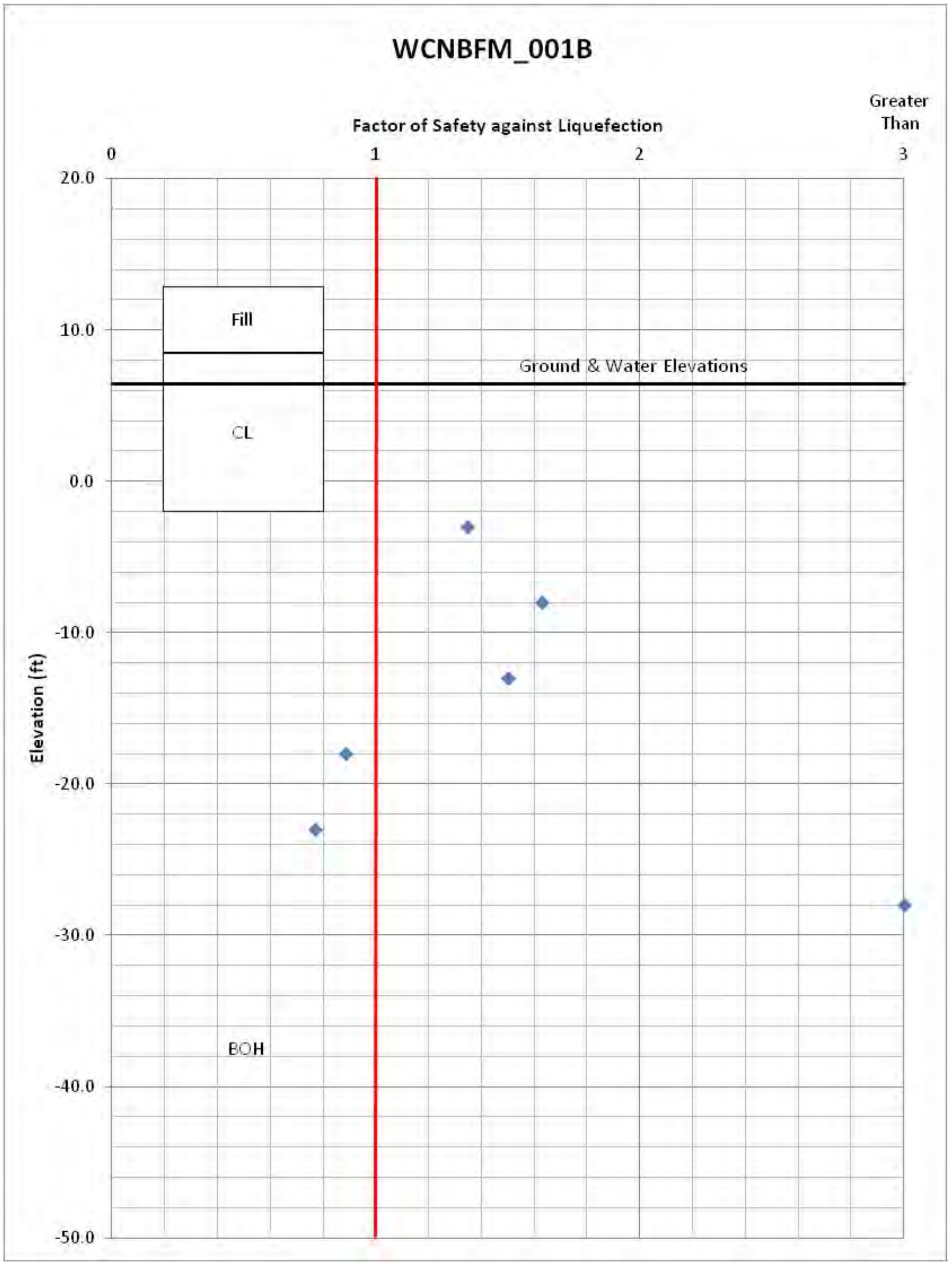


Fig. C-68. Lincoln Village, Station 201+51

Appendix D

Selected Boring Logs

DATE STARTED 2/6/07 · DATE COMPLETED 2/8/07	GROUND ELEVATION 29.4 ft · ELEVATION BASIS **Andregg Survey**	TOTAL DEPTH OF BORING **101.5 ft**
DRILLING CONTRACTOR **Westex**	DRILLER'S NAME **Chris Minor** · HELPER'S NAME **Boyd Dortsch**	TOTAL DEPTH OF FILL **18 ft**
DRILLING METHOD **0'-26.5' HSA, 26.5'-101.5' Rotary**	DRILL RIG MAKE AND MODEL **CME 550**	CONSULTANT COMPANY **Fugro West, Inc.**
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) **4 3/8 inches**	DRILLING ROD TYPE AND DIAMETER **HQ core 94mm, NWJ 67mm**	FIELD LOGGER **Spyridon Giannakos**
☒ VERTICAL ☐ INCLINED	CASING TYPE, DIAMETER, INSTALLATION DEPTH **10" HSA, 26.5'**	FIELD LOG REVIEWER **Duston Marlow**
SAMPLER TYPE(S) **SPT(1.375"), MC(2"), Punch Core(2.25"), Shelby Tube(3")**	HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP **140 lb CME Automatic Hammer/ 30-inch drop**	HAMMER EFFICIENCY **72%**
BOREHOLE BACKFILL OR COMPLETION **cement grout to ground surface**	GROUNDWATER READING: DURING DRILLING **24 ft**	AFTER DRILLING (DATE-TIME) **Not Measured**

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	0		CLAYEY GRAVEL (GC); dense; brown (10YR 5/3); dry; medium to coarse sand; fine to coarse gravel; (FILL).		1	83	12 14 17	31	37						S01C_000_000S, S01B_000_001S, S01A_001_001S
	1		SANDY SILT (ML); dense; brown (10YR 5/3); dry; low plasticity, low dry strength, slow dilatancy, low toughness fines; (FILL).												
	2		SILTY SAND (SM); dense; dark yellowish brown (10YR 4/4); dry; fine to medium sand; (FILL).												
	3														
25	4		SANDY SILT (ML); brown (10YR 4/3); dry; low plasticity, low dry strength, slow dilatancy, low toughness fines; with clay mottling (FILL).												
	5				2	83	5 8 10	18			12			45	S02B_006_006M, S02A_006_007M
	6		CLAYEY SAND (SC); brown (7.5YR 5/4); dry; 55% fine to medium sand; 45% fines; trace organics (FILL).												
	7		Poorly Graded SAND with Silt (SP-SM); medium dense; pale yellow (2.5Y 8/2); dry; 95% medium to coarse sand; 5% fines; (FILL).		3	83	4 6 7	13	16		2			5	S03B_007_007S, S03A_007_008S
	8		SANDY SILT (ML); brown (10YR 4/3); moist; low plasticity, low dry strength, slow dilatancy, low toughness fines; (FILL).												
20	9														
	10		Poorly Graded SAND (SP); loose; very pale brown (10YR 8/2); 98% fine to medium sand; 2% fines; (FILL).		4	78	WOH 2 3	5	6		3			2	S04B_010_010S, S04A_010_011S
	11														
	12														
	13		LEAN CLAY (CL); very dark grayish brown (10YR 3/2); dry; low to medium dry strength, no to slow dilatancy, low toughness fines; (FILL).		5	90					30	44	22	88	S05A_013_015T 3" Shelby 0 psi
15	14		SILT (ML); brown (10YR 4/3); moist; 88% low dry strength, slow dilatancy, low toughness fines; 12% sand; oxidized (FILL).												
	15		SANDY SILT (ML); loose; dark yellowish brown (10YR 4/4); dry; 50% fine sand; 50% fines; trace mica (FILL).		6	61	2 3 4	7	8		13			50	S06A_015_016S
	16														
	17		(BASE OF FILL).												
	18		SILTY SAND (SM); medium dense; brown (10YR 5/3); moist; 83% fine to medium sand; 17% fines; with white mottling (NATIVE).												
10	19														
	20														

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,122,291.98 East 6,324,796.46
Levee Station or Milepost: STA: 1553+82.13 Offset: 3.24 feet Left
GPS: Latitude 37.82064 Longitude -121.32006
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_069B

Sheet 1 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA: Water Content, %	LABORATORY DATA: Liquid Limit	LABORATORY DATA: Plasticity Index	LABORATORY DATA: Fines, % < #200	REMARKS
	20		7	100	3 5 7	12			9			17	S07B_021_021M, S07A_021_022M
	21	81% sand; 19% fines; very pale brown (10YR 8/3) below 21.5'.	8	83	3 5 7	12	14		11			19	S08A_022_023S
∇ 5	23	Poorly Graded SAND with Silt (SP-SM); medium dense; light yellowish brown (10YR 6/4); wet; 94% medium sand; 6% fines.											
	25	Olive (5Y 5/3) from 26.0' to 30.5'.	9	67	3 8 8	16	19		25			6	S09A_025_026S
0	26.5		10	67									S10A_027_028P Switch to Rotary
	30	Dark greenish gray (10G 4/1) below 30.5'.	11	44	5 6 7	13	16						S11B_030_030S, S11A_030_031S
-5	31.5			12									No Sample Taken
	35	SILT with Sand (ML); medium dense; olive gray (5Y 4/2); moist; low plasticity, low to medium dry strength, slow dilatancy, low toughness fines.	12	83	3 4 5	9	11	1.0P 1.8P					S12B_035_036S, S12A_036_036S
	36	LEAN CLAY with Sand (CL); very stiff; olive gray (5Y 5/2); moist; medium dry strength, slow dilatancy, low toughness fines; trace sand, oxidized, trace organics, white mottling.	13	67				2.5P	29	45	28		S13A_037_038P
-10	38	SANDY SILT (ML); medium dense; gray (5Y 5/1); moist; low plasticity, low dry strength, slow dilatancy, low toughness fines; oxidized, trace organics.											
	40	Dark grayish brown (2.5Y 4/2) mottled with red, medium plasticity below 40.0'.	14	60					29	24	1		S14A_040_042T 3" Shelby 100 psi
	42.5		15	72	7 6 8	14	17	2.0P 4.5P					S15A_043_044S
-15	44	SILT (ML); medium dense; brown (10YR 4/3); moist; low	16	100									S16A_045_045P
	45												

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,122,291.98 East 6,324,796.46
Levee Station or Milepost: STA: 1553+82.13 Offset: 3.24 feet Left
GPS: Latitude 37.82064 Longitude -121.32006
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_069B

Sheet 2 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N$_{60}$(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	45 46	plasticity, low to medium dry strength, slow dilatancy, low toughness fines; trace mica, trace organics, oxidized. SILTY SAND (SM); medium dense; olive brown (2.5Y 4/3); moist; fine to medium sand; trace mica.	☒	17	89	2 6 11	17	20						S17B_045_046S, S17A_046_047S
-20	47 48 49 50	CLAYEY SAND (SC); gray (5Y 5/1); 75% fine to medium sand; 25% fines; trace mica.	▮	18	57					20			25	S18A_048_048P
	51	SILTY SAND (SM); dense; olive brown (2.5Y 4/4); moist; 74% fine sand; 26% fines; trace mica.	☒	19	61	12 15 15	30	36						S19B_050_051S, S19A_051_051S
-25	52 53 54	Oxidized from 52.5' to 53.0'.	▮	20	81					21			26	S20A_052_053P
	55 56	Poorly Graded SAND (SP); very dense; olive (5Y 4/3); moist; fine to medium sand; trace mica.	☒	21	56	17 25 33	58	70						S21B_055_055S, S21A_055_056S
-30	57 58 59	Well-Graded SAND (SW); olive gray (5Y 4/2); moist; fine to coarse sand; trace mica. SILTY SAND (SM); very dense; olive gray (5Y 4/2); moist; 73% fine sand; 27% fines; trace mica, gray mottling.	▮	22	60					21			27	S22A_059_060P
	60 61		☒	23	61	19 25 33	58	70						S23A_060_061S
-35	62 63 64	Poorly Graded SAND with Silt (SP-SM); olive gray (5Y 4/2); 89% fine to coarse sand; 11% fines; trace mica.	▮	24	100					25			11	S24A_062_063P
	65 66 67	LEAN CLAY with Sand (CL); stiff; greenish gray (10G 5/1); moist; high dry strength, no to slow dilatancy, medium toughness fines; oxidized.	▮	NR	0									No Recovery
	68		☒	25	100	6 7 9	16	19	2.0P	31	40	24		S25A_068_069S
-40	69 70		▮	26	67									S26A_069_070P

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,122,291.98 East 6,324,796.46
Levee Station or Milepost: STA: 1553+82.13 Offset: 3.24 feet Left
GPS: Latitude 37.82064 Longitude -121.32006
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_069B
Sheet 3 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA: Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	70 71		LEAN CLAY (CL); very stiff; greenish gray (10G 5/1); dry; medium plasticity, high dry strength, no to slow dilatancy; medium toughness fines; oxidized.	☒	27	100	6 8 8	16	19	3.8P					S27A_070_072S
-45	72 73 74		3' layer of silty sand (SM). SILTY SAND (SM); olive gray (5Y 4/2); moist; 87% fine to coarse sand; 13% fines; polygenic coarse sand.	▮	28	100					17			13	S28A_074_075P
	75 76		Poorly Graded SAND with Silt (SP-SM); dense; light yellowish brown (2.5Y 6/3); moist; fine to medium sand. LEAN CLAY (CL); olive (5Y 4/3); moist; medium plasticity, medium to high dry strength, no to slow dilatancy, medium toughness fines; oxidized, trace organics.	☒	29	61	23 23 18	41	49						S29B_075_076S, S29A_076_076S
-50	77 78 79		SANDY SILT (ML); olive brown (2.5Y 4/3); moist; low plasticity, medium to high dry strength, no to slow dilatancy, medium toughness fines; trace organics, trace mica.	▮	30	100									S30A_077_078P
	80 81 82		CLAYEY SAND (SC); olive brown (2.5Y 4/3); moist; fine sand; trace mica. LEAN CLAY with Sand (CL); light olive brown (2.5Y 5/3); moist; medium to high dry strength, slow dilatancy, low toughness fines; trace mica, trace organics.	▮	31	90					26	42	28		S31A_080_083S 3" Shelby 0 psi
	83			☒	NR	0	7 8 11	19	23						No Recovery
-55	84		SILTY CLAYEY SAND (SC-SM); dense; light olive brown (2.5Y 5/3); moist; 62% sand; 38% low to medium dry strength, slow dilatancy, low toughness fines; trace mica, oxidized.	▮	32	100									S32A_084_085P
	85 86			☒	33	61	8 11 15	26	31	1.0P	26	29	7	38	S33A_085_086S
-60	87 88 89		SILTY SAND (SM); medium dense; olive gray (5Y 4/2); moist; 53% fine sand; 47% fines; trace mica, oxidized.	▮	34	57									
	90 91		Trace organics below 90.5'.	☒	35	61	9 10 14	24	29		22			47	S35B_090_091S, S35A_091_091S
-65	92 93 94 95		SILT (ML); olive brown (2.5Y 4/3); moist; medium plasticity, medium to high dry strength, slow dilatancy, low toughness fines; trace mica, oxidized.	▮	NR	0									No Recovery

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,122,291.98 East 6,324,796.46
Levee Station or Milepost: STA: 1553+82.13 Offset: 3.24 feet Left
GPS: Latitude 37.82064 Longitude -121.32006
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_069B

Sheet 4 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	95														
	96		LEAN CLAY with Sand (CL); medium stiff; gray (5Y 5/1); moist; medium to high dry strength, no to slow dilatancy; medium to high toughness fines; trace mica, oxidized.		36	100	WOH 4 7	11	13	0.8P					S36B_095_095S, S36A_095_097S
	97 98 99		Silty sand lens at 97.5'. Very stiff to hard below 98.0'.		37	86				1.8P 3.0T 4.5P	38	43	23		S37A_097_098P
-70															
	100 101		CLAYEY SAND (SC); medium dense; dark grayish brown (2.5Y 4/2); moist; low plasticity, medium to high dry strength, slow dilatancy, medium toughness fines; fine to medium sand; trace oxidation. SILTY SAND (SM); medium dense; dark grayish brown (2.5Y 4/2); moist; 52% fine to coarse sand; 48% fines; trace oxidation.		38	67	9 10 12	22	26		16			48	S38B_100_101S, S38A_101_101S
	102 103		Borehole terminated at 101.5 feet. Backfilled with cement grout..												
	104														
-75	105														
	106														
	107														
	108														
	109														
-80	110														
	111														
	112														
	113														
	114														
-85	115														
	116														
	117														
	118														
	119														
-90	120														

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,122,291.98 East 6,324,796.46
Levee Station or Milepost: STA: 1553+82.13 Offset: 3.24 feet Left
GPS: Latitude 37.82064 Longitude -121.32006
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_069B
Sheet 5 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

DATE STARTED 2/9/07 — DATE COMPLETED 2/12/07	GROUND ELEVATION 29.9 ft — ELEVATION BASIS Andregg Survey	TOTAL DEPTH OF BORING 101.5 ft
DRILLING CONTRACTOR Westex	DRILLER'S NAME Chris Minor — HELPER'S NAME Boyd Dortsch	TOTAL DEPTH OF FILL 20 ft
DRILLING METHOD 0'-26.5' HSA, 26.5'-101.5' Rotary	DRILL RIG MAKE AND MODEL CME 555	CONSULTANT COMPANY Fugro West, Inc.
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) 4 3/8 inches	DRILLING ROD TYPE AND DIAMETER HQ core 94mm, NWJ 67mm	FIELD LOGGER J. Anderson/S. Giannakos
☒ VERTICAL ☐ INCLINED	CASING TYPE, DIAMETER, INSTALLATION DEPTH 10" HSA, 26.5'	FIELD LOG REVIEWER Duston Marlow
SAMPLER TYPE(S) SPT(1.375"), MC(2"), Punch Core(2.25"), Shelby Tube(3")	HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP 140 lb CME Automatic Hammer/ 30-inch drop	HAMMER EFFICIENCY 72%
BOREHOLE BACKFILL OR COMPLETION cement grout to ground surface	GROUNDWATER READING: DURING DRILLING Not Measured	AFTER DRILLING (DATE-TIME) 25.5 ft on 2/12/07 11:00AM

Elevation, feet	Depth, feet	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60} (ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	0	CLAYEY GRAVEL (GC); dense; light olive brown (2.5Y 5/3); dry; medium to coarse sand; fine to coarse gravel; base rock (FILL).	1	61	19 15 15	30	36						S01B_000_001S, S01A_001_001S
	1	LEAN CLAY with Sand (CL); very dark grayish brown (10YR 3/2); moist; medium plasticity, medium to high dry strength, no to slow dilatancy, medium toughness fines; (FILL).											
	4	Poorly Graded SAND (SP); medium dense; brown (10YR 4/3); dry; fine to medium sand; trace mica (FILL).											
25	5		2	61	6 7 5	12	14						S02B_005_006S, S02A_006_006S
	6	SANDY SILT (ML); loose to medium dense; very dark grayish brown (2.5Y 3/2); moist; 65% medium dry strength, slow dilatancy, medium toughness fines; 35% sand; trace mica (FILL). Dark yellowish brown (10YR 4/4), fine to medium sand from 7.5' to 8.0'. Dark grayish brown (10YR 4/2), slow to rapid dilatancy below 8.0'.											
	7		3	93					19	27	2	65	S03A_008_010T 3" Shelby 0 psi
20	10		4	44	3 3 3	6	7						S04A_010_011S
	12	LEAN CLAY (CL); dark grayish brown (10YR 4/2); dry; medium dry strength, slow dilatancy, low toughness fines; trace mica (FILL).											
	13	LEAN CLAY with Sand (CL); very dark grayish brown (2.5Y 3/2); moist; medium plasticity, medium to high dry strength, no to slow dilatancy, low toughness fines; trace mica, oxidized (FILL).	5	90					32	43	20		S05A_013_015T 3" Shelby 0 psi
15	15		6	56	3 3 3	6	7						S06A_015_016S
10	20	(BASE OF FILL).											

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,120,457.89 East 6,325,060.90
Levee Station or Milepost: STA: 1595+33.28 Offset: 4.31 feet Right
GPS: Latitude 37.81561 Longitude -121.31911
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_074B

Sheet 1 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	20		Poorly Graded SAND with Silt (SP-SM); loose; dark grayish brown (10YR 4/2); 88% fine to medium sand; 12% fines; trace mica, oxidized. Dark brown (10YR 3/3) below 21.0'.		7	94	3 4 6	10			11				S07C_020_021M, S07B_021_021M, S07A_021_022M
	21.5				8	72	4 3 4	7	8		9			12	S08A_022_023S
	23		SANDY SILT (ML); dark gray (2.5Y 4/1); moist; medium plasticity, medium to high dry strength, no to slow dilatancy, medium toughness fines; with clay, trace mica, oxidized.												
5	25				9	83	1 2 3	5	6		35			11	S09B_025_026S, S09A_026_026S
▼	25.5		Poorly Graded SAND with Silt (SP-SM); loose; dark grayish brown (2.5Y 4/2); moist; 89% fine to medium sand; 11% fines; oxidized, trace mica.												
	26.5		Dark gray below 28.5'.			14									No Sample Taken. Switch to Rotary
0	30		Medium dense below 30.0'.		10	39	4 5 5	10	12						S10A_030_031S
	31.5				11	29									S11A_031_032P
	32		CLAYEY SAND (SC); medium dense; dark greenish gray (10Y 4/1); wet; 86% fine to medium sand; 14% fines; trace mica.												
-5	35				12	50	5 6 7	13	16		31			14	S12A_035_036S
	36.5				13	24									S13A_036_037P
-10	40		Greenish gray (5GY 5/1) below 41.0'.		NR	0									No Recovery Hydraulically Advanced SPT Sampler
	41.5				14	33	4 5 7	12	14						S14A_042_042S
	43		87% sand; 13% fines.		15	38					29			13	S15A_043_044P
-15	45														

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,120,457.89 East 6,325,060.90
Levee Station or Milepost: STA: 1595+33.28 Offset: 4.31 feet Right
GPS: Latitude 37.81561 Longitude -121.31911
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_074B
Sheet 2 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA				REMARKS
											Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	
	45				16	22	10 8 8	16							S16A_045_046M
	47		2" of medium to coarse sand at 46.5'. Well-Graded SAND with Clay (SW-SC); medium dense to dense; dark greenish gray (5GY 4/1); wet; 89% sand; 11% fines; trace mica.		17	44	4 5 6	11	13						S17B_047_047S, S17A_047_047S no sand size description
	48				18	50					28			11	S18A_048_049P
-20	50		Well-Graded SAND (SW); dense; greenish gray (10Y 5/1); wet.		19	56	15 20 21	41	49						S19B_050_051S, S19A_051_051S
	51		CLAYEY SAND (SC); dense; dark greenish gray (10GY 4/1); wet; 82% fine to coarse sand; 18% fines.		20	50					14			18	S20A_051_052P
-25	55		87% sand; 13% fines; light greenish gray (10Y 8/1), fine to medium sand from 55.0' to 59.0'.		21	67	14 31 29	60			19			13	S21B_055_056M, S21A_056_056M
	57				22	28	5 9 21	30	36						S22A_057_057S
	58		Greenish gray (10Y 6/1), fine to coarse sand below 59.0'.		23	100									S23A_058_059P
-30	60		87% sand; 13% fines.		24	50	14 19 20	39	47		20			13	S24A_060_061S
	62		SILT with Sand (ML); light olive gray (5Y 6/2); moist; low plasticity, low dry strength, slow dilatancy, low toughness fines; trace mica.		NR	0									No Recovery
-35	65		LEAN CLAY with Sand (CL); stiff to very stiff; olive gray (5Y 5/2); moist; high dry strength, no dilatancy, low toughness fines; oxidized.		25	72	5 6 7	13	16	1.0P 1.8P	28	39	25		S25B_065_065S, S25A_065_066S
	67		Greenish gray (10Y 5/1), medium toughness below 67.5'. 1" silty fine to medium sand lens at 67.8'.		26	88				3.0P 2.5T					S26A_068_069P
-40	70														

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,120,457.89 East 6,325,060.90
Levee Station or Milepost: STA: 1595+33.28 Offset: 4.31 feet Right
GPS: Latitude 37.81561 Longitude -121.31911
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_074B

Sheet 3 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	70 71 72				NR	0									No Recovery 3" Shelby 0 psi
	73				27	100	6 8 10	18	22	2.3P					S27A_073_074S
	74		Grayish brown (2.5Y 5/2), low to medium toughness, trace organics and mica below 74.0'.		28	100				1.8P	27	49	34		S28A_074_075P
-45	75 76 77				29	83									S29A_075_077T 3" Shelby
	78		LEAN CLAY (CL); olive gray (5Y 5/2); moist; high dry strength, no dilatancy, low toughness fines; oxidized.		30	100	0 1 7	8	10	0.1P	26	41	28		S30A_078_079S
	79				31	100				0.5P 3.5T					S31A_079_080P
-50	80				32	100	0 4 5	9	11	0.8P					S32B_080_081S
	81 82 83 84		SANDY LEAN CLAY (CL); stiff; olive gray (5Y 5/2); moist; 52% medium dry strength, no to slow dilatancy, low toughness fines; 48% sand; oxidized. SANDY SILT (ML); greenish gray (10Y 6/1); moist; low plasticity, low dry strength, slow dilatancy, low toughness fines; oxidized, trace mica. Dark greenish gray (5GY 4/1) below 84.5'.		33	100				0.1P	26	32	19	52	S33A_082_083P
-55	85 86		LEAN CLAY (CL); soft to medium stiff; dark greenish gray (5GY 4/1); moist; low plasticity, medium dry strength, no to slow dilatancy, low toughness fines; trace mica. SANDY SILT (ML); dark greenish gray (5GY 4/1); moist; low plasticity, low to medium dry strength, slow dilatancy, low toughness fines; trace mica.		34	83	0 4 14	18	22	0.6P 0.3P					S34C_085_086S, S34B_086_086S, S34A_086_086S
	87 88 89		Well-Graded SAND with Silt (SW-SM); medium dense; dark greenish gray (5GY 4/1); fine to coarse sand. FAT CLAY (CH); soft; greenish gray (10Y 5/1); wet; medium to high plasticity, no dry strength, no dilatancy, low toughness fines.			52				0.3P					No Sample Taken
-60	90 91 92		SANDY LEAN CLAY (CL); olive gray (5Y 5/2); moist; low dry strength, slow dilatancy, low toughness fines; with sand, oxidized.		35	90					30	60	43		S35A_090_093T 3" Shelby
	93		LEAN CLAY (CL); medium stiff to stiff; light olive gray (5Y 6/2); moist; low plasticity, high dry strength, no dilatancy, medium toughness fines; oxidized, trace mica.		36	83	6 6 7	13	16	1.5P					S36B_093_093S, S36A_096_094S
-65	94 95					100									No Sample Taken

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,120,457.89 East 6,325,060.90
Levee Station or Milepost: STA: 1595+33.28 Offset: 4.31 feet Right
GPS: Latitude 37.81561 Longitude -121.31911
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_074B

Sheet 4 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	95 96	▨		☒	37	100	0 4 8	12	14	0.8P					S37A_095_097S
	97 98 99	░	SANDY SILT (ML); very dense; grayish brown (2.5Y 5/2); moist; 67% low dry strength, slow dilatancy, low toughness fines; 33% sand; oxidized, trace mica.	■	38	71				1.0P 0.8P	30	33	9	67	S38A_098_099P
-70	100 101	░		☒	39	94	14 24 30	54	65	2.5P					S39A_100_102S
	102 103 104		Borehole terminated at 101.5 feet. Backfilled with cement grout..												
-75	105 106 107 108 109														
-80	110 111 112 113 114														
-85	115 116 117 118 119														
-90	120														

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,120,457.89 East 6,325,060.90
Levee Station or Milepost: STA: 1595+33.28 Offset: 4.31 feet Right
GPS: Latitude 37.81561 Longitude -121.31911
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_074B

Sheet 5 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

DATE STARTED **12/15/06**	DATE COMPLETED **12/21/06**	GROUND ELEVATION **22.9 ft**
ELEVATION BASIS **Andregg Survey**	TOTAL DEPTH OF BORING **126.5 ft**	
DRILLING CONTRACTOR **Westex**	DRILLER'S NAME **Chris Minor**	HELPER'S NAME **Boyd Dortsch**
TOTAL DEPTH OF FILL **15 ft**		
DRILLING METHOD **0'-25' HSA, 25'-126.5' Rotary**	DRILL RIG MAKE AND MODEL **CME 550**	CONSULTANT COMPANY **Fugro West, Inc.**
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) **4 3/8 inches**	DRILLING ROD TYPE AND DIAMETER **HQ core 94mm, NWJ 67mm**	FIELD LOGGER **Spyridon Giannakos**
☒ VERTICAL ☐ INCLINED	CASING TYPE, DIAMETER, INSTALLATION DEPTH **HSA, 10", 25'**	FIELD LOG REVIEWER **Duston Marlow**
SAMPLER TYPE(S) **SPT(1.375'), Punch Core(2.25"), Shelby Tube(3')**	HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP **Automatic CME 140 lb, 30-inch drop**	HAMMER EFFICIENCY **72%**
BOREHOLE BACKFILL OR COMPLETION **cement grout to ground surface**	GROUNDWATER READING: DURING DRILLING **19 ft**	AFTER DRILLING (DATE-TIME) **31 ft on 12/19/06 8:00AM**

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60} (ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	0		CLAYEY GRAVEL (GC); loose; brown (7.5YR 4/4); dry to moist; medium to coarse sand; fine to coarse gravel; (FILL).	☒	1	100	3 3 3	6	7						S01A_000_002S
	1		LEAN CLAY (CL); brown (7.5YR 4/4); moist; medium plasticity, low dry strength, slow dilatancy, medium toughness fines; some sand (FILL).												
	2														
20	3														
	4														
	5			☒											S02B_005_006S, S02A_006_006S
	6		SILTY CLAYEY SAND (SC-SM); loose; dark reddish brown (5YR 3/4); fine to medium sand; (FILL).		2	56	6 4 3	7	8		12	22	5		
	7														
15	8														
	9														
	10		CLAYEY SAND (SC); loose; yellowish brown (10YR 5/4); moist; 84% fine to medium sand; 16% fines; (FILL).	▮											S03A_010_013T 3" Shelby 300 psi
	11			▮	3	72					5				
	12			▮											
10	13			☒											S04A_013_015S
	14		(BASE OF FILL).		4	100	3 2 4	6	7		7			16	
	15		SILTY SAND (SM); dark grayish brown (10YR 4/2); moist; 86% fine sand; 14% fines; (NATIVE).	▮											S05_A_015_018T 3" Shelby 300 psi
	16			▮	5	80					26			14	
	17			▮											
5	18		SANDY LEAN CLAY (CL); dark reddish brown (5YR 3/2); moist; low plasticity, low dry strength, no to slow dilatancy, low toughness fines; trace mica.	☒	6	100	2 2 4	6	7						S06A_018_019S
∇	19		Wet at 19.0'. CLAYEY SAND (SC); loose; dark reddish brown (5YR	▮	7	77					26	32	12		S07A_019_021T 3" Shelby
	20														

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,152,579.89 East 6,323,939.91
Levee Station or Milepost: STA: 1151+05.61 Offset: 11.19 feet Left
GPS: Latitude 37.90375 Longitude -121.32397
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_019B

Sheet 1 of 6

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	20	3/2); moist; low dry strength, no dilatancy, low toughness fines.											300 psi
	21	6" silty sand slough at 21.5'.	7	77					26	32	12		
	22	Poorly Graded SAND with Clay (SP-SC); loose to medium dense; dark gray (10YR 4/1); wet; 94% fine sand; 6% fines; micaceous.	8	100	2 4 4	8	10		28			6	S08B_022_022S, S08A_022_023S
0	23												
	24												
	25	LEAN CLAY with Sand (CL); stiff; reddish gray (5YR 5/2); moist; low dry strength, low toughness fines.	9	89	2 4 5	9	11	4.3P	31	35	13		S09_025_027S Switch to Rotary
	26												
	27												S10A_026_029P
-5	28		10	81									
	29												
	30	SANDY SILT (ML); medium dense; dark reddish brown (5YR 3/2); wet; some iron oxide mottling, micaceous.	11	100	5 7 9	16	19		28	26	4		S11A_030_032S
	31												
	32												S12A_032_033P
-10	33		12	45									
	34												
	35												S13A_035_036S
	36	2" medium sand lens at 35.5'.	13	72	5 11 7	18	22						
	37												S14A_036_039P
-15	38	LEAN CLAY with Sand (CL); stiff to very stiff; dark reddish gray (5YR 4/2); moist to wet; medium dry strength, no dilatancy, medium toughness fines; micaceous.	14	90				2.3P	25	45	31		
	39												
	40	Stiff from 40.0' to 43.0'.											S15A_040_042S
	41		15	100	3 5 5	10	12	1.8P					
	42	1" to 2" subangular gravel lens at 41.5'.											S16A_042_043P
-20	43		16	100				3.3P					
	44												
	45												

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,152,579.89 East 6,323,939.91
Levee Station or Milepost: STA: 1151+05.61 Offset: 11.19 feet Left
GPS: Latitude 37.90375 Longitude -121.32397
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_019B

Sheet 2 of 6

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	45		White mottling at 45.0'.		17	100	6 9 10	19	23						S17A_045_047S
	46														
	47														S18A_050_050P
-25	48				18	14									
	49														
	50				19	100	7 13 16	29	35						S19B_050_051S, S19A_051_052S
	51		<u>SILTY SAND</u> (SM); dense; dark reddish brown (5YR 3/3); moist; 70% fine sand; 30% fines; some brown and black mottling.												
	52										24			26	S20A_052_053P
-30	53		Light olive gray (5Y 5/2), fine to medium sand at 53.0'.		20	76									
	54														
	55		Medium to coarse sand at 55.0'.												S21A_056_057S
	56		<u>LEAN CLAY</u> (CL); very stiff; brown (7.5YR 4/2); moist; medium to high plasticity, medium dry strength, no dilatancy, medium toughness fines; micaceous.		21	100	6 9 8	17	20	2.8P					
	57														S22A_057_058P
-35	58				22	100				2.5P					
	59														
	60		<u>SANDY LEAN CLAY</u> (CL); medium stiff; (5GY 3/1) (4/4); moist; low dry strength, slow dilatancy, low toughness fines; fine sand.		23	100	0 0 2	2	2	0.8P	33	34	20		S23A_060_062S
	61														
	62														S24A_062_063P
-40	63		<u>CLAYEY SAND</u> (SC); dense; dark grayish brown (10YR 4/2); moist to wet; fine to medium sand; micaceous.		24	81									
	64														
	65				25	89	6 15 17	32	38						S25B_065_066S, S25A_066_067S
	66		Dark greenish gray (5G 4/1); 66% sand; 34% fines; dark greenish gray (5G 4/1) below 66.0'.												
	67										27			34	S26A_067_068P
-45	68				26	76									
	69														
	70														

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,152,579.89 East 6,323,939.91
Levee Station or Milepost: STA: 1151+05.61 Offset: 11.19 feet Left
GPS: Latitude 37.90375 Longitude -121.32397
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_019B

Sheet 3 of 6

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

											LABORATORY DATA				
Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	70														S27B_070_071S, S27A_071_072S
	71		LEAN CLAY with Sand (CL); very stiff, dark greenish gray (5G 4/1); dry to moist; no dilatancy, medium toughness fines; some iron oxide mottling.		27	100	14 13 16	29	35	2.5P					
	72														S28A_073_074P
-50	73		Decrease in stiffness, trace fine sand from 73.0'.		28	100									
	74									1.0P					
	75		Hard at 75.0'.												S29A_075_077S
	76				29	100	6 10 11	21	25	4.3P	23	37	24		
	77														S30A_077_078P
-55	78		Very stiff at 78.0'.		30	100									
	79														
	80		Reddish brown (5YR 4/4) mottled with iron oxide staining at 80.0'.												S31A_080_082S
	81				31	100	12 16 18	34	41	3.0P					
	82		Dark greenish gray (5BG 4/1) at 82.0'.							3.5P					S32A_082_083P
-60	83				32	100									
	84														
	85		SILTY SAND (SM); dense; dark greenish gray (5G 4/1); wet; 67% fine sand; 33% fines; micaceous.								33			33	S33A_085_087S
	86				33	100	8 12 16	28	34						
	87														S34A_086_087P
-65	88				34	31									
	89														
	90		SANDY SILT (ML); loose; dark olive gray (5Y 3/2); moist to wet; high plasticity, low dry strength, no dilatancy, low toughness fines; micaceous.												S35A_090_092S
	91				35	100	0 0 8	8	10	0.3P					
	92														S36A_094_095P
-70	93				36	71									
	94		2" medium to coarse sand lens at 93.5'. SANDY LEAN CLAY (CL); very stiff; very dark greenish gray (10GY 3/1); dry; 57% medium dry strength, no								16			57	
	95														

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,152,579.89 East 6,323,939.91
Levee Station or Milepost: STA: 1151+05.61 Offset: 11.19 feet Left
GPS: Latitude 37.90375 Longitude -121.32397
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_019B

Sheet 4 of 6

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	95–97		dilatancy, medium toughness fines; 43% fine sand; micaceous. FAT CLAY with Sand (CH); very stiff; dark greenish gray (5G 4/1); dry to moist; medium dry strength, no dilatancy, medium toughness fines; white mottling.		37	100	8 12 15	27	32	3.7P					S37A_095_097S
-75	97–100		Hard from 97.0' to 100.0'.		38	100				4.5+P					S38A_097_098P
	100–102		Trace fine sand below 101.0'.		39	100	10 15 19	34	41	3.3P	29	53	38		S39A_100_102S
-80	102–105		6" silt lens at 102.5'. Hard below 103.5'.		40	100				4.5+P					S40A_102_103P
	105–106.5				41	100	8 10 14	24	29						S41B_106_106S, S41A_106_107S
-85	106–110		SILTY SAND (SM); very dark greenish gray (10Y 3/1); moist to wet; 80% fine sand; 20% fines. Dark gray (10YR 4/1), fine to medium sand, micaceous below 107.0'.		42	90					20			20	S42A_107_108P
	110–111.5		CLAYEY SAND (SC); very dense; dark gray (2.5Y 4/1); wet; 82% sand; 18% fines.		43	100	22 32 29	61	73		21			18	S43A_110_112S
-90	111.5–115		LEAN CLAY (CL); olive gray (5Y 5/2); moist; medium to high dry strength, no dilatancy, high toughness fines; iron oxide mottling.		44	57				4.0P					S44A_113_114P
	115–116.5		SILT (ML); medium dense; light gray (5Y 7/2); moist; low plasticity fines; fine sand; trace sand. LEAN CLAY with Sand (CL); stiff; bluish gray (5B 5/1); dry; medium dry strength, no dilatancy, high toughness fines; some iron oxide staining, interlayered with lenses of soft silt.		45	67	8 10 12	22	26						S45A_115_116S
-95	116.5–120				46	100				1.5P					S46A_118_119P

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,152,579.89 East 6,323,939.91
Levee Station or Milepost: STA: 1151+05.81 Offset: 11.19 feet Left
GPS: Latitude 37.90375 Longitude -121.32397
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_019B

Sheet 5 of 6

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	120	▨	Medium stiff from 120.0' to 123.5'.												S47A_120_122S
	121	▨		☒	47	100	1 3 4	7	8	0.8P	34	39	23		
	122	▨		■											S48A_123_124P
-100	123	▨		■	48	100				2.5P					
	124	▨	Very stiff below 123.5'. White mottling at 124.0'.	■											
	125	▨													No Recovery
	126	▨		☒	NR	0	4 8 12	20	24						
	127		Borehole terminated at 126.5 feet. Backfilled with cement grout..												
-105	128														
	129														
	130														
	131														
	132														
-110	133														
	134														
	135														
	136														
	137														
-115	138														
	139														
	140														
	141														
	142														
-120	143														
	144														
	145														

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,152,579.89 East 6,323,939.91
Levee Station or Milepost: STA: 1151+05.61 Offset: 11.19 feet Left
GPS: Latitude 37.90375 Longitude -121.32397
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_019B
Sheet 6 of 6

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

DATE STARTED **12/13/06**	DATE COMPLETED **12/15/06**	GROUND ELEVATION **22.8 ft**	ELEVATION BASIS **Andregg Survey**	TOTAL DEPTH OF BORING **101.5 ft**
DRILLING CONTRACTOR **Westex**		DRILLER'S NAME **Chris Minor**	HELPER'S NAME **Boyd Dortsch**	TOTAL DEPTH OF FILL **15 ft**
DRILLING METHOD **0'-25' HSA, 25'-101.5' Rotary**		DRILL RIG MAKE AND MODEL **CME 550**		CONSULTANT COMPANY **Fugro West, Inc.**
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) **4 3/8 inches**		DRILLING ROD TYPE AND DIAMETER **HQ core 94mm, NWJ 67mm**		FIELD LOGGER **Spyridon Giannakos**
☒ VERTICAL ☐ INCLINED		CASING TYPE, DIAMETER, INSTALLATION DEPTH **HSA, 10", 25'**		FIELD LOG REVIEWER **Duston Marlow**
SAMPLER TYPE(S) **SPT(1.375'), Punch Core(2.25"), Shelby Tube(3')**		HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP **Automatic CME 140 lb, 30-inch drop**		HAMMER EFFICIENCY **72%**
BOREHOLE BACKFILL OR COMPLETION **cement grout to ground surface**		GROUNDWATER READING:	DURING DRILLING **20 ft**	AFTER DRILLING (DATE-TIME) **Not Measured**

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in	Blows per Foot	N_{60} (ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	0		CLAYEY GRAVEL (GC); brown (7.5YR 4/4); medium to coarse sand; fine to coarse gravel; well graded gravel (FILL). SANDY LEAN CLAY (CL); brown (7.5YR 4/3); dry; low dry strength, slow dilatancy, low to medium toughness fines; trace gravel and organics (FILL).	☒	1	44	9 5 8	13	16						S01B_000_001S, S01A_001_001S
	1														
	2														
20	3														
	4														
	5		63% fines; 37% sand; with orange-brown mottling, micaceous below 5.0'.	☒	2	39	3 4 9	13	16		16	32	17	63	S02A_005_006S
	6														
	7		LEAN CLAY with Sand (CL); reddish brown (5YR 4/4); moist; 75% low dry strength, slow dilatancy, low toughness fines; 25% fine sand; some iron oxide staining (FILL).	▮	3	50					22	32	14	75	S03A_007_009T 3" Shelby 0 psi
15	8														
	9														
	10		With approximately 0.5 mm lenses of silty sand, some clay, medium brown (5YR 3/4) to dusty brown (5YR 2/2) at 10.0'.	☒	4	67	3 5 4	9	11						S04A_010_011S
	11														
	12														
10	13														
	14		(BASE OF FILL).												
	15		FAT CLAY with Sand (CH); medium stiff; dark yellowish brown (10YR 4/4); moist; low dry strength, slow dilatancy, low toughness fines; micaceous (NATIVE).	▮	5	87					31				S05A_015_017T 3" Shelby 300 psi
	16														
	17														
5	18			☒	6	100	0 3 3	6	7	1.0P	43	53	30		S06A_018_019S
	19		Dusty brown (5YR 2/2), some iron oxide staining below 19.0'.												
▽	20														

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,148,895.80 East 6,322,760.53
Levee Station or Milepost: STA: 1191+43.18 Offset: 8.34 feet Left
GPS: Latitude 37.89359 Longitude -121.32796
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_024B

Sheet 1 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Number	Recovery, %	Blows per 6 in	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	20		7	100									S07A_020_023T 3" Shelby 300 psi
	21	SANDY LEAN CLAY (CL); soft; dark olive gray (5Y 3/2); moist; 69% no dilatancy, low toughness fines; 31% fine sand.											
0	23	Lenses of iron oxide staining, fine grained sand at 22.5'.	8	67	2 3 3	6	7	0.3P	25	29	16	69	S08A_023_024S
	25		9	100	2 2 4	6	7						S09B_025_026S, S09A_025_027S Switch to Rotary
	26	SILTY SAND (SM); loose; dark olive gray (5Y 3/2); wet; fine sand; trace mica.											
	27	SILT (ML); olive gray (5Y 5/2); moist; low plasticity, low dry strength, low toughness fines; some iron oxide staning, trace mica.	10	100				4.5P					S10A_028_029P
-5	28	LEAN CLAY (CL); stiff; olive gray (5Y 4/2); moist; medium plasticity, medium dry strength, slow dilatancy, medium toughness fines.						2.0P					
	29	SANDY LEAN CLAY (CL); very stiff; dark grayish brown (10YR 4/2); medium dry strength, slow dilatancy, medium toughness fines; trace fine sand and mica.											
	30		11	100	6 9 5	14	17	2.5P	31	38	19		S11A_030_032S
	31	SILT (ML); reddish brown (5YR 4/4); wet; fine to medium sand; with clay lenses.											S12A_033_034P
-10	33	Some iron oxide staining at 32.5'. Sand and silt lenses at 33.0'.	12	71				2.5P	28			90	
	34	LEAN CLAY (CL); reddish brown (5YR 4/4); moist; 90% medium plasticity, medium dry strength, medium toughness fines; 10% sand.											
	35		13	100	6 7 9	16	19						S13A_035_037S
	36	CLAYEY SAND (SC); medium dense; dark yellowish brown (10YR 4/4); 72% fine to medium sand; 28% fines; poorly graded, trace clay.	14	64					25			28	S14A_037_038P
-15	38												
	40	Loose at 40.5'.	15	100	3 1 2	3	4	2.5P					S15A_040_042S
	42												S16A_043_044P
-20	43		16	100									
	44	SANDY LEAN CLAY (CL); reddish gray (5YR 5/2); low plasticity, medium dry strength, medium toughness fines; fine to medium sand.											
	45												

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,148,895.80 East 6,322,760.53
Levee Station or Milepost: STA. 1191+43.18 Offset: 8.34 feet Left
GPS: Latitude 37.89359 Longitude -121.32796
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_024B
Sheet 2 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	REMARKS
	45 46		CLAYEY SAND (SC); dense; yellowish brown (10YR 5/4); 73% fine to medium sand; 27% fines.		17	44	6 12 19	31	37		20			27	S17A_045_047S
-25	47 48 49				18	81									S18A_048_049P
	50 51		FAT CLAY with Sand (CH); stiff to very stiff; yellow (10YR 8/6); low dry strength, no dilatancy, medium toughness fines; pockets of black organics and iron oxide stained fine sand.		19	100	1 4 6	10	12	2.5P 3.5P	31	54	41		S19A_050_052S
-30	52 53 54		Stiff from 53.0' to 57.0'.		20	100				1.8P 1.3P 4.2T					S20A_053_054P
	55 56		Dark yellowish brown, micaceous at 55.0'.		21	100	6 9 12	21	25	1.3P					S21A_056_057S
-35	57 58 59		Interlayered fine to medium sand from 57.0' to 58.5' (1" to 4" in thickness).		22	100				2.5P 4.0T					S22A_058_059P
	60 61		SILTY SAND (SM); dense; dark reddish brown (5YR 3/2); wet; rounded to subrounded gravel; 83% fine to coarse sand; 17% fines; micaceous.		23	100	8 19 22	41	49		18			17	S23A_060_062S
-40	62 63 64		Increase in coarse sand at 64.0'.		24	86									S24A_063_064P Driller's Note (DN): Hard drilling below 62.0'
	65 66		Grayish brown (10YR 5/2), very dense below 65.0'. Dark yellowish brown below 66.0'		25	44	17 30 32	62	74						S25A_065_066S
-45	67 68 69 70		3" poorly-graded sand lens, medium to coarse sand, dark yellowish orange (10YR 2/2) at 68.5'.		26	100									S26A_066_067P

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,148,895.80 East 6,322,760.53
Levee Station or Milepost: STA: 1191+43.18 Offset: 8.34 feet Left
GPS: Latitude 37.89359 Longitude -121.32796
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_024B
Sheet 3 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA				REMARKS
											Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	
	70 71		86% sand; 14% fines; dark greenish gray (10Y 4/1), fine to medium sand below 70.0'.		27	83	9 24 22	46	55		28			14	S27A_073_074S
-50	72 73 74				28	95									S28A_073_074P
	75 76		Trace coarse sand at 75.0'.		29	89	18 25 24	49	59						S29A_075_077S
-55	77 78 79				30	71									S30A_078_079P
	80 81		Dense from 80.0' to 85.0'.		31	89	19 23 15	38	46						S31A_080_082S
-60	82 83 84				32	48									S32A_082_083P
	85 86				33	100	9 21 23	44	53						S33A_085_087S
-65	87 88 89		6" well-graded sand, fine to coarse sand, fine to coarse gravel at 87.5'. LEAN CLAY (CL); very stiff; dry to moist; high plasticity, high dry strength, no dilatancy, high toughness fines; micaceous. CLAYEY SAND (SC); medium dense to dense; very dark gray (10YR 3/1); 61% fine to medium sand; 39% fines.		34	100									S34A_088_089P
	90 91				35	100	9 11 14	25	30		16			39	S35A_090_092S
-70	92 93 94 95				36	81									S36A_093_094P

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,148,895.80 East 6,322,760.53
Levee Station or Milepost: STA: 1191+43.18 Offset: 8.34 feet Left
GPS: Latitude 37.89359 Longitude -121.32796
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_024B

Sheet 4 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	FIELD CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.	Blows per Foot	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA: Water Content, %	LABORATORY DATA: Liquid Limit	LABORATORY DATA: Plasticity Index	LABORATORY DATA: Fines, % < #200	REMARKS
	95		SILT (ML); very dense; very dark gray (10YR 3/1); dry to moist; low plasticity fines.		37	100	12 21 27	48	58						S37A_095_097S
	96		LEAN CLAY (CL); dark gray (10YR 4/1); dry to moist; medium plasticity, medium dry strength, no to slow dilatancy, medium toughness fines.												S38B_097_098P, S38A_098_099P
	97														
-75	98		Poorly Graded SAND with Silt (SP-SM); dark grayish brown (10YR 4/2); wet; fine to medium sand.		38	100									
	99														
	100		LEAN CLAY (CL); dark gray (10YR 4/1); low plasticity, low dry strength, no dilatancy, low toughness fines; micaceous, white lineations.		39	100	12 26 21	47	56						S39A_100_101S
	101														
	102		Borehole terminated at 101.5 feet. Backfilled with cement grout..												
-80	103														
	104														
	105														
	106														
	107														
-85	108														
	109														
	110														
	111														
	112														
-90	113														
	114														
	115														
	116														
	117														
-95	118														
	119														
	120														

Final Report Version 9/30/2008

Borehole Location: Crest of Levee
Coordinates: North 2,148,895.80 East 6,322,760.53
Levee Station or Milepost: STA: 1191+43.18 Offset: 8.34 feet Left
GPS: Latitude 37.89359 Longitude -121.32796
Channel / River Name / Feature: San Joaquin River
County: San Joaquin

LOG OF BORING
WR0017_024B

Sheet 5 of 5

Engineering Support Services
Urban Levee Geotechnical Evaluations Program

DATE STARTED 10/19/10 **DATE COMPLETED** 10/19/10	**GROUND ELEVATION** 22.36 ft **ELEVATION DATUM** NAVD 88	**TOTAL DEPTH OF BORING** 73.5 ft
DRILLING CONTRACTOR Pitcher	**DRILLER'S NAME** James Musich **HELPER'S NAME** William Stewart	**TOTAL DEPTH OF FILL** 14.5 ft
DRILLING METHOD HSA and Mud Rotary	**DRILL RIG MAKE AND MODEL** CME-55, PD 56	**CONSULTANT COMPANY** URS Corporation
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) 10" Auger, 4.875" Drag bit, 4.25 HQ bit	**DRILLING ROD TYPE AND DIAMETER** 3-1/2" HQ, 2-5/8" N, 10" HSA	**FIELD LOGGER** M. Palmer
☒ **VERTICAL** ☐ **INCLINED**	**CASING TYPE, DIAMETER, INSTALLATION DEPTH** 5.5 OD x 5" ID Steel to 15'	**FIELD LOG REVIEWER** M. Palmer
SAMPLER TYPE(S) SPT, 2.5" ID Punch Core, 3" Osterberg	**HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP** CME Autohammer, 140 lbs/30-inch	**HAMMER EFFICIENCY** 80%
BOREHOLE BACKFILL OR COMPLETION Cement-bentonite grout	**GROUNDWATER READING: DURING DRILLING** Not noted	**AFTER DRILLING (DATE-TIME)**

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60} (ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
	0		Asphalt Concrete Road.												0-17.5' HSA with center plug
			AB Road Base.												
	1		[LEVEE FILL] SANDY LEAN CLAY (CL); olive brown (2.5Y 4/3); moist; 70% low to medium plasticity, high dry strength, no dilatancy fines; 30% fine sand; micaceous.												
20	2–4				S01A	17	1 2 3 [5]	7							S01A_003_004S
	4–5														4-5' HSA with center plug
	5				S02A	87									S02A_005_007T 3" Osterberg Piston sample 150 psi
15	6–7		[LEVEE FILL] SILT with Sand (ML); black (10YR 2/1); moist; 75% high dry strength, no to slow dilatancy fines; 25% fine sand.							24	34	8		UW	
	8		[LEVEE FILL] SANDY LEAN CLAY (CL); dark olive brown (2.5Y 3/3); 60% low plasticity, high dry strength, slow dilatancy fines; 40% fine to coarse sand; no reaction with HCl; orange and black mottling; micaceous.		S03B S03A	44	1 3 2 [5]	7							S03B_007_008S S03A_008_009S
	9														9-10' HSA with center plug
	10–11				S04A	83				16	NP	NP	63	UW	S04A_010_012T 3" Osterberg Piston sample 200 psi
10	12–13		[LEVEE FILL] SANDY SILT (ML); loose; olive brown (2.5Y 4/3); moist; 63% low dry strength, rapid dilatancy fines; 37% fine to coarse sand; weak reaction with HCl; micaceous, some laminations.		S05A	50	2 2 2 [4]	5					63	HD	S05A_013_014S
	14														14-15' with center plug
FILL	14.5		SANDY SILT (ML); loose; olive brown (2.5Y 4/3); moist; 51% low dry strength, rapid dilatancy fines; 49% fine to coarse sand; strong reaction with HCl; micaceous.												
	15–17				S06A	100				14 18	 18	 1	 51	UW UW HD	S06A_015_018T 3" Osterberg Piston sample 100 psi After SPT, pull augers and set 5.5" x 5.0" steel casing to 15', then cleanout and advance with 4-7/8 drag bit to 20'.
5	18		At 17.5 feet 54% fines; 46% sand.		S07A	67	2 2 3 [5]	7					54	HD	S07A_018_019S
	19–20														19' - 20' Mud Rotary drag bit

Final Report Version 2/22/2011

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,157,195.76 **Easting:** 6,329,451.55
Latitude: 37.91655 **Longitude:** -121.30500
Levee Station or Milepost: 1175+01 **Levee Mile:**
Levee Segment
Survey Method: GIS/LiDAR **Coord. System:** CA State Plane Zone II
Channel / River Name / Feature: French Camp Slough

LOG OF BORING
WR0404_041B

Sheet 1 of 4

Engineering Support Services Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	CLASSIFICATION OF MATERIALS (Description)	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N60(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
0	20 21 22	Well-Graded SAND with Silt (SW-SM); loose; olive brown (2.5Y 4/3); wet; 92% fine to medium sand; 8% no plasticity fines; no reaction with HCl; micaceous.	S08A	0									3" Osterberg Piston sample 300 psi Sample fell out of tube into mud tank. Sand.
	23		S09A	50	2 2 2 [4]	5					8	HD	S09A_023_024S 22.5-73.5"101 geobarrel
	24 25 26		S10A	33									25' - 27' NR
-5	27 28	LEAN CLAY (CL); stiff; grayish brown (2.5Y 5/2); moist; 90% high dry strength, no dilatancy fines; 10% fine sand; variably strong reaction with HCl.	S11A	72	2 5 6 [11]	15	1.5P	25	39	20			S11A_027_028S
	29 30 31	Sand in cuttings indicate SILTY SAND (SM).	S12A	14									29' - 30' NR
-10	32 33	LEAN CLAY (CL); stiff; light olive brown (2.5Y 5/3); moist; 90% high dry strength, no dilatancy fines; 10% fine to medium sand; weak reaction with HCl; scattered orange and black mottling.	S13A	100	0 2 3 [5]	7	1.0P	26	39	18			S13A_032_033S
	34		S14A	100			1.0P						
	35 36		S15A	100			1.0P 1.0P						
-15	37 38	LEAN CLAY with Sand (CL); very stiff; light olive brown (2.5Y 5/3) grades to olive brown (2.5Y 4/3); moist; 80% high dry strength, no dilatancy fines; 20% fine to medium sand; no reaction with HCl; orange iron oxide mottling throughout; blocky texture.	S16A	94	6 8 10 [18]	24		23	40	19			S16A_037_038S
	39 40 41	FAT CLAY (CH); very stiff; olive brown (2.5Y 4/3); moist; 90% high plasticity, very high dry strength, no dilatancy fines; 10% fine to medium sand; black mottling throughout, micaceous.	S17A	95			2.25P 3.0P 2.75P						
-20	42 43	At 41.9 feet 1-inch thick black medium sand lens.	S18A	50	4 9 9 [18]	24	2.5P	29					S18A_042_043S
	44 45	At 44 feet grayish green (5G 4/2).	S19A	100									

Final Report Version 2/22/2011

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,157,195.76 **Easting:** 6,329,451.55
Latitude: 37.91655 **Longitude:** -121.30500
Levee Station or Milepost: 1175+01 **Levee Mile:**
Levee Segment
Survey Method: GIS/LiDAR **Coord. System:** CA State Plane Zone II
Channel / River Name / Feature: French Camp Slough

LOG OF BORING
WR0404_041B
Sheet 2 of 4

Engineering Support Services Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
	45 46		LEAN CLAY with Sand (CL); medium stiff; dark greenish gray (5G 4/1); moist; 80% high dry strength, no to slow dilatancy fines; 20% fine sand; weak reaction with HCl; trace brown organic fibers.		S19A	100									
-25	47 48		At 48' very stiff.		S20A	83	1 6 9 [15]	20	0.5P 3.5P	28					S20A_047_048S
	49 50 51				S21A	21			3.75P						49.75-52' NR Sample blocked off in shoe.
-30	52 53		At 52.5 feet up to 20% calcium carbonate precipitation throughout; strong reaction with HCl.		S22A	100	3 8 9 [17]	23		24	41	22			S22A_052_053S
	54 55 56		SANDY SILT (ML); dense; dark greenish gray (5G 4/1); moist; 55% low plasticity, high dry strength, slow to rapid dilatancy fines; 45% fine sand; no reaction with HCl; micaceous.		S23A	86									
-35	57 58				S24A	78	5 11 16 [27]	36							S24A_057_058S
	59 60 61		SILTY SAND (SM); very dense; dark greenish gray (5G 4/1); moist; 76% fine sand; 24% no plasticity fines; no reaction with HCl.		S25A	29									59.5-62' NR
-40	62 63				S26A	67	13 25 32 [57]	76					24		S26A_062_063S
	64 65 66				S27A	43									
-45	67 68		FAT CLAY (CH); very stiff; dark greenish gray (5G 4/1); moist; 90% medium to high plasticity, very high dry strength, no dilatancy fines; 10% fine sand; strong reaction with HCl; white calcium carbonate mottling throughout.		S28A	56	7 8 12 [20]	27							S28A_067_068S
	69 70				S29A	95			2.25P						

Final Report Version 2/22/2011

Borehole Location: Levee Crest
County: San Joaquin
Coordinates: Northing: 2,157,195.76
Easting: 6,329,451.55
Latitude: 37.91655
Longitude: -121.30500
Levee Station or Milepost: 1175+01
Levee Mile:
Levee Segment
Survey Method: GIS/LiDAR
Coord. System: CA State Plane Zone II
Channel / River Name / Feature: French Camp Slough

LOG OF BORING
WR0404_041B
Sheet 3 of 4

Engineering Support Services Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
	70	▨	SANDY LEAN CLAY (CL); very stiff; dark greenish gray (5G 4/1); moist; 60% medium plasticity, high dry strength, no dilatancy fines; 40% fine to coarse sand; strong reaction with HCl; scattered thin seams of Clayey Sand (SC).						2.25P						
	71				S29A	95			2.25P						
-50	72				S30A	67	8 19 27 [46]	61							S30A_072_073S
	73														
	74		Bottom of boring 73.5'. Backfilled by tremmie to bottom of boring with approximately 50 gallons grout mix consisting of 35 gallons of water, 6-47lbs sacks of portland cement and 15 lb bentonite to within 2 feet of surface, 1/2 sack Holeplug to within 1-1/2 feet of surface then levee cuttings and finished with AB.												
	75														
	76														
-55	77														
	78														
	79														
	80														
	81														
-60	82														
	83														
	84														
	85														
	86														
-65	87														
	88														
	89														
	90														
	91														
-70	92														
	93														
	94														
	95														

Final Report Version 2/22/2011

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,157,195.76 **Easting:** 6,329,451.55
Latitude: 37.91655 **Longitude:** -121.30500
Levee Station or Milepost: 1175+01 **Levee Mile:**
Levee Segment
Survey Method: GIS/LiDAR **Coord. System:** CA State Plane Zone II
Channel / River Name / Feature: French Camp Slough

LOG OF BORING
WR0404_041B
Sheet 4 of 4

Engineering Support Services Urban Levee Geotechnical Evaluations Program

DWR LEVEE UMU SOIL LOG REV1: RD404.GPJ DWR OFFICIAL LIBRARY 12222010.GLB 2/21/11

DATE STARTED 10/6/11 **DATE COMPLETED** 10/6/11	**GROUND ELEVATION** 19.5 ft **ELEVATION DATUM** NAVD 88	**TOTAL DEPTH OF BORING** 51.5 ft
DRILLING CONTRACTOR Gregg Drilling & Testing, Inc.	**DRILLER'S NAME** Angel Salazar **HELPER'S NAME** Martin Soto	**TOTAL DEPTH OF FILL** 11 ft
DRILLING METHOD Hand Auger/HSA/Mud Rotary	**DRILL RIG MAKE AND MODEL** Mobile B-80 (D-21)	**CONSULTANT COMPANY** URS Corporation
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) 6" Carbide tooth auger, 4-7/8" drag bit	**DRILLING ROD TYPE AND DIAMETER** 2-1/2" NWJ	**FIELD LOGGER** M. Palmer
☒ **VERTICAL** ☐ **INCLINED**	**CASING TYPE, DIAMETER, INSTALLATION DEPTH** 6-5/8" O.D. Steel to 10'	**FIELD LOG REVIEWER** R. Nixon
SAMPLER TYPE(S) Bag, Dames&Moore (2.5"x20"), Osterberg (3"x36"), SPT(1.375")	**HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP** Marl, automatic, 140 lbs / 30-inch drop	**HAMMER EFFICIENCY** 85%
BOREHOLE BACKFILL OR COMPLETION Cement-bentonite grout	**GROUNDWATER READING: DURING DRILLING** Not encountered due to drilling method	**AFTER DRILLING (DATE-TIME)**

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
	0		[LEVEE FILL] SANDY LEAN CLAY (CL); yellowish brown (10YR 5/6); moist; 55% high dry strength, slow dilatancy fines; 45% fine sand; no reaction with HCl.												Hand Auger to 6' HSA to 5'
	2				S01A					14	31	10	55		S01A_002_004B
15	4.5		[LEVEE FILL] LEAN CLAY with Sand (CL); very stiff; dark grayish brown (2.5Y 4/2); moist; 85% low plasticity, high dry strength, no to slow dilatancy fines; 15% fine to medium sand; no reaction with HCl; reddish-orange iron oxide staining throughout; micaceous.		S02A	72			3.0P	19	39	22		UW	S02A_005_008T Osterberg 400 psi
	8														Advance with 6" HSA.
	9				S03A	91			3.9P	19	39	15	70	UW	S03A_009_010T Dames & Moore 250 psi
10	9.5		[LEVEE FILL] LEAN CLAY with Sand (CL); very stiff; olive (5Y 5/3); moist; 70% high dry strength, no to slow dilatancy fines; 30% fine to medium sand.												Set casing to 10'. Clean out and advanced to with 4-7/8" drag bit, switched to Mud Rotary.
←FILL	11		FAT CLAY (CH); dark grayish brown (2.5Y 4/2); 90% high plasticity, very high dry strength, no dilatancy fines; 10% fine sand; weak reaction with HCl.		S04A	63				24 24	55	40		UW UW CCRS	S04A_011_014T Osterberg 500 psi
5	14		FAT CLAY with organics (CH); very dark gray (5Y 3/1); moist; 88% high dry strength, no dilatancy fines; 12% organic matter; carbonized organic matter (fine gravel size); slight organic odor; slight spongy feel.		S05A	89									S05A_015_016T Dames & Moore 350 psi
	17				S06A	67			1.0P	78 43 45	58 73	33 49		UW OC SG UW DS UW CCRS	S06A_017_020T Osterberg 350 psi
0	19		At 19 feet medium stiff to stiff.												
	20														

Draft 3 After All Lab Data Added 7/25/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,180,127.61 **Easting:** 6,319,568.23
Latitude: 37.97928 **Longitude:** -121.33997
Levee Station or Milepost: 6565+02 **Levee Mile:** 22.4
Levee Segment Reach L
Survey Method: GPS **Coord. System:** State Plane
Channel / River Name / Feature: Calaveras River

LOG OF BORING
WR1614_018B
Sheet 1 of 3

Levee Evaluations

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA					REMARKS
										Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	
	20 21 22 23		<u>SILT with organics</u> (ML); soft; very dark gray (5Y 3/1); moist; 95% slow dilatancy fines; 5% fine grained brown and black organics; slight organic odor; micaceous.		S07A	84			0.5P	35	43	16		UW	S07A_021_022T Dames & Moore 180 psi
-5	24 25 26		<u>ELASTIC SILT with organics</u> (MH); soft; very dark gray (5Y 3/1); moist; 90% fines; 10% black fibrous organics; slight organic odor; micaceous. <u>ORGANIC ELASTIC SILT</u> (OH); very stiff; very dark gray (10YR 3/1); moist; 70% fines; organic odor; fibrous; 30% organic material.		S08A	100			2.4P	98 157 170 190 188	145 300 278	91 178 174		UW CD UW OC UW UW DS UW CCRS	S08A_023_026T Osterberg 320 psi
	27 28		<u>ELASTIC SILT with organics</u> (MH); very dark gray (5Y 3/1); moist; 90% low plasticity, medium to high dry strength, rapid dilatancy fines; 10% fine grained organics; slight organic odor; trace shell fragments at bottom of sample.		S09A	100				163	140	65		UW	S09A_027_029T Dames & Moore 380 psi
-10	29 30 31 32		At 29 feet 89% low plasticity, medium dry strength, rapid dilatancy fines; 6% organic material; 5% fine to medium sand; strong reaction with HCl; shell fragments, spongy texture.		S10A	100			0.4P	132 160 145	125 127	41 56		UW OC SG UW CCRS UW DS	S10A_030_032T Osterberg 300 psi
-15	33 34 35		At 32.5 feet medium stiff; dark greenish gray (5GY 4/1); 95% low plasticity, medium to high dry strength, rapid dilatancy fines; 5% fine sand; no reaction with HCl; trace mica; scattered brown, woody fragments.		S11A	89			0.6P						S11A_033_035T Dames & Moore 500 psi
	36 37		<u>LEAN to FAT CLAY with organics</u> (CL/CH); dark greenish gray (5GY 4/1); moist; 5% organics. At 36.5 trace shells variably scattered shells have strong reaction with HCl.		S12A	100				65 94 27	94 29	60 14		UW OC UW CCRS UW CCRS	S12A_036_038T Osterberg 520 psi Lost 3" sample out of tube.
-20	38 39 40		<u>Poorly Graded SAND with Silt</u> (SP-SM); black (N 2.5/); wet; 90% fine to medium sand; 10% no plasticity fines; no reaction with HCl; trace brown woody fibrous fragments.		S13A	100									S13A_039_041T Dames & Moore 100 psi
	41 42		At 40.5 feet very loose.		S14A	100	2 1 1 [2]	3							S14A_041_042S
-25	43 44 45		<u>ELASTIC SILT with Sand</u> (MH); dark greenish gray (5GY 4/1); moist; 95% low plasticity, medium to high dry strength, rapid dilatancy fines; 5% fine to medium sand; no reaction with HCl; trace brown woody fibrous fragments.		S15A	100									S15A_043_046T Osterberg 500 psi

Draft 3 After All Lab Data Added 7/25/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,180,127.61 **Easting:** 6,319,568.23
Latitude: 37.97928 **Longitude:** -121.33997
Levee Station or Milepost: 6565+02 **Levee Mile:** 22.4
Levee Segment Reach L
Survey Method: GPS **Coord. System:** State Plane
Channel / River Name / Feature: Calaveras River

LOG OF BORING
WR1614_018B
Sheet 2 of 3

Levee Evaluations

DWR LEVEE LINU SOIL LOG REV1 SJAFCA-20091208.GPJ DWR OFFICIAL LIBRARY 10092012.GLB 2/26/13

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
	45			█	S15A	100			0.5P	62	68	27	80	UW	
	46		At 45.5 feet soft.												
	47	▨	FAT CLAY (CH); dark greenish gray (5GY 4/1); moist; 87% fines; 13% sand.	█	S16A	100			0.25P	60	56	26	87	UW	S16A_047_048T Dames & Moore 220 psi
	48	▨													
	49	▨	At 49 feet slight organic odor.	█											S17A_049_052T Osterberg 400 psi
-30	50	▨		█	S17A	100									
	51		Poorly Graded SAND (SP); very dark gray (N 3/); wet; 96% no plasticity fines; 4% fine to coarse sand; trace fine gravel, organics.	█						21			4	UW	
	52														
	53														
	54														
-35	55														
	56														
	57														
	58														
	59														
-40	60														
	61														
	62														
	63														
	64														
-45	65														
	66														
	67														
	68														
	69														
-50	70														

Total Depth Drilled 51.5 feet

Borehole backfilled with 80 gallons of cement bentonite grout consisting of 15 bags (47 lbs) of Type II-V Portland cement, 35 pounds of bentonite and approximately 60 gallons of water.

Draft 3 After All Lab Data Added 7/25/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,180,127.61 **Easting:** 6,319,568.23
Latitude: 37.97928 **Longitude:** -121.33997
Levee Station or Milepost: 6565+02 **Levee Mile:** 22.4
Levee Segment Reach L
Survey Method: GPS **Coord. System:** State Plane
Channel / River Name / Feature: Calaveras River

LOG OF BORING
WR1614_018B
Sheet 3 of 3

Levee Evaluations

DATE STARTED 11/3/11	**DATE COMPLETED** 11/3/11	**GROUND ELEVATION** 15.4 ft	**ELEVATION DATUM** NAVD 88	**TOTAL DEPTH OF BORING** 31.0 ft
DRILLING CONTRACTOR Gregg Drilling & Testing, Inc.		**DRILLER'S NAME** Luis Torres	**HELPER'S NAME** Rob Ramirez	**TOTAL DEPTH OF FILL** 0 ft
DRILLING METHOD Hand Auger/Mud Rotary		**DRILL RIG MAKE AND MODEL** Mobile B-53 (D-26)		**CONSULTANT COMPANY** URS Corporation
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) 4-5/8" drag bit		**DRILLING ROD TYPE AND DIAMETER** 2-1/2" NWJ, 4" HWJ		**FIELD LOGGER** P. Crispell
☒ **VERTICAL** ☐ **INCLINED**		**CASING TYPE, DIAMETER, INSTALLATION DEPTH** 8" LCS to 12'		**FIELD LOG REVIEWER** M. Turner
SAMPLER TYPE(S) PCore(2.5"), SPT(1.375")		**HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP** Marl, automatic, 140 lbs / 30-inch drop		**HAMMER EFFICIENCY** 77%
BOREHOLE BACKFILL OR COMPLETION Cement-bentonite grout		**GROUNDWATER READING: DURING DRILLING** Not encountered due to drilling method		**AFTER DRILLING (DATE-TIME)**

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
15	0		SILTY SAND (SM); brown (7.5YR 5/4); moist; 60% fine sand; 40% low plasticity, low dry strength, slow dilatancy, low toughness fines.												Hand auger to 5'
10	5		At 5 feet very dense.		S01A	33	14 21 22 [43]	55		24	46	8	40		S01A_005_007S
	6.5		At 6.5 feet hard pan.		S02A	93									
5	10		Gradational change. SILT with Sand (ML); loose; brown (7.5YR 4/3); wet; 81% low dry strength, slow dilatancy, low toughness fines; 19% fine sand; indurated.												
	11				S03A	72	2 3 5 [8]	10		43	46	11	81		S03A_011_013S
	12.5		LEAN CLAY with Sand (CL); brown (7.5YR 4/3); moist; 85% fines; 15% fine sand.		S04A	64			0.9P	42	41	20			S04A_014_015P
0	15.5		ELASTIC SILT (MH); medium stiff; dark brown (10YR 3/3); moist; 100% high dry strength, no dilatancy, low toughness fines.												
	16				S05A	78	0 3 5 [8]	10		45	50	13			S05A_016_018S
	17.5		FAT CLAY (CH); dark brown (10YR 3/3); moist; 100% high plasticity, very high dry strength, no dilatancy, high toughness fines.		S06A	57									
	20														

Draft 3 After All Lab Data Added 12/31/2012

Borehole Location: Landside Levee Toe **County:** San Joaquin
Coordinates: Northing: 2,182,117.89 **Easting:** 6,328,852.24
Latitude: 37.98497 **Longitude:** -121.30782
Levee Station or Milepost: 6669+40 **Levee Mile:** 20.4
Levee Segment Reach O
Survey Method: GPS **Coord. System:** State Plane
Channel / River Name / Feature: Calaveras River

LOG OF BORING
WR1614_019B
Sheet 1 of 2

Levee Evaluations

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
-5	20				S06A	57									
	21		At 21 feet very dark grayish brown (10YR 3/2).		S07B S07A	89	6 6 8 [14]	18							S07B_021_022S S07A_022_023S
	22 23		SANDY SILT (ML); medium dense; dark grayish brown (10YR 4/2); moist; 60% low plasticity fines; 40% fine sand. At 23 feet 75% fines; 25% sand.												S08B_023_024P
-10	24 25		LEAN CLAY with Sand (CL); very dark grayish brown (10YR 3/2); moist; 78% medium dry strength, no dilatancy, medium toughness fines; 22% fine sand. At 25.5 feet medium stiff.		S08A	76			0.7P	30	34	14	78		S08A_024_025P
	26 27		SANDY SILT (ML); very dark grayish brown (10YR 3/2); moist; 53% fines; 47% sand.		S09A	100	0 0 0 [0]	0		38	35	6	53		S09A_026_028S
-15	28 29 30		At 29 feet 54% fines; 46% sand.		S10A	57			0.6P	29	37	12	54		S10A_029_030P
	31 32 33 34		Total Depth Drilled 31 feet Borehole backfilled with 50 gallons of cement bentonite grout consisting of 8 bags (47 lbs) of Type II-V Portland cement, 20 pounds of bentonite and approximately 40 gallons of water.												
-20	35 36 37 38 39														
-25	40 41 42 43 44 45														

Draft 3 After All Lab Data Added 12/31/2012

Borehole Location: Landside Levee Toe **County:** San Joaquin
Coordinates: Northing: 2,182,117.89 **Easting:** 6,328,852.24
Latitude: 37.98497 **Longitude:** -121.30782
Levee Station or Milepost: 6669+40 **Levee Mile:** 20.4
Levee Segment Reach O
Survey Method: GPS **Coord. System:** State Plane
Channel / River Name / Feature: Calaveras River

LOG OF BORING
WR1614_019B
Sheet 2 of 2

Levee Evaluations

DWR LEVEE UMU SOIL LOG REV1   SJAFCA 20091208.GPJ   DWR OFFICIAL LIBRARY 10092012.GLB   2/14/13

DATE STARTED 8/20/10	**DATE COMPLETED** 8/20/10	**GROUND ELEVATION** 14.00 ft	**ELEVATION DATUM** NAVD 88	**TOTAL DEPTH OF BORING** 40.0 ft
DRILLING CONTRACTOR Neil O. Anderson		**DRILLER'S NAME** Mike Young	**HELPER'S NAME** Sean McNeil	**TOTAL DEPTH OF FILL** 9.5 ft
DRILLING METHOD HSA/Rotary Wash		**DRILL RIG MAKE AND MODEL** CME 75		**CONSULTANT COMPANY** Kleinfelder
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) 10" HSA, 4-7/8" Drag Bit		**DRILLING ROD TYPE AND DIAMETER** NWJ 2-5/8"		**FIELD LOGGER** G. Lenehan
☒ **VERTICAL** ☐ **INCLINED**		**CASING TYPE, DIAMETER, INSTALLATION DEPTH** HSA 10" OD, 15 ft		**FIELD LOG REVIEWER** G. Lenehan
SAMPLER TYPE(S) SPT (1.375"), PC (2.5")		**HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP** CME Auto Hammer 140 lbs/30-inch drop		**HAMMER EFFICIENCY** 79%
BOREHOLE BACKFILL OR COMPLETION Grout		**GROUNDWATER READING: DURING DRILLING** Not Measured Due to Drilling Method		**AFTER DRILLING (DATE-TIME)**

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
	0 1 2		SANDY SILT (ML); very stiff; yellowish brown (10YR 5/4); dry; 55% low dry strength, slow dilatancy, low toughness fines; 45% fine to medium sand; strong reaction with HCl; [Levee Fill]. At 1.0 foot, 1 inch lens of silty gravel (GM).		S01A	75	9 10 11 [21]	28	2.25P		39	13			S01A_000_002S S01A_000_002S Hollow-Stem Auger to 22 feet
10	3 4		GRAVELLY LEAN CLAY with Sand (CL); brown (10YR 4/3); moist; 60% medium plasticity, medium dry strength, no dilatancy, medium toughness fines; 20% fine, subrounded gravel; 20% fine to medium sand; strong reaction with HCl; [Levee Fill].		S02A	44									S02A_002_005P S02A_002_005P
	5 6		SANDY LEAN CLAY (CL); very stiff; very dark grayish brown (10YR 3/2); 70% high dry strength, no dilatancy, medium toughness fines; 30% fine sand; [Levee Fill].		S03A	75	1 2 2 [4]	5	2.0P	16	37	13			S03A_005_007S S03A_005_007S
5 ←FILL	7 8 9		SILTY SAND (SM); dark yellowish brown (10YR 4/4); moist; 76% fine to medium sand; 24% fines; [Levee Fill].		S04A	39			0.75P				24	PA	S04A_007_010P S04A_007_010P
	10 11 12		LEAN CLAY (CL); medium stiff; very dark grayish brown (10YR 3/2); moist; 90% medium plasticity, medium dry strength, no dilatancy, medium toughness fines; 10% fine sand; with organics. At 12.0 feet, 5 inch lens of sandy lean clay (CL).		S05A	67								SG	S05A_010_012T S05A_010_012T SG = 2.80
0	13 14		SANDY SILT (ML); very soft; very dark grayish brown (10YR 3/2); moist; 70% low dry strength, slow dilatancy, low toughness fines; 30% fine sand.		S06A	100			0P	37	40	14		OC	S06A_012_015P S06A_012_015P OC = 4.6%
	15		SANDY SILT (ML); dark yellowish brown (10YR 4/6); moist; 70% low plasticity, low dry strength, slow dilatancy, low toughness fines; 30% fine sand.												
	16		ORGANIC SILT with Sand (OL); soft; black (N 2.5/); moist; 85% medium plasticity, medium dry strength, slow dilatancy, medium toughness fines; 15% fine sand.		S07A	100	0 0 0 [0]	0	0.25P	41 62				OC	S07A_015_017S S07A_015_017S OC = 14.6%
	17 18		LEAN CLAY with Sand (CL); medium stiff; black (10YR 2/1); moist; 80% medium plasticity, medium dry strength, no dilatancy, medium toughness fines; 20% fine sand.		S08A	100			0.5P					OC	S08A_017_020P S08A_017_020P OC = 12.2%
-5	19		SILT (ML); black (N 2.5/); moist; 90% no plasticity, low dry strength, slow to rapid dilatancy, low toughness fines; 10% fine sand; with organics.						0.5P						
	20		ORGANIC SILT with Sand (OH); medium stiff; black (N 2.5/); moist; 80% medium dry strength, slow dilatancy,						0.5P	71	60	21			

Draft 3 After All Lab Data Added 5/16/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,178,670.55 **Easting:** 6,310,266.10
Latitude: 37.97505 **Longitude:** -121.37220
Levee Station or Milepost: 117+51 **Levee Mile:**
Levee Segment Brookside
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: San Joaquin River

LOG OF BORING
WR2074_003M
Sheet 1 of 2

Engineering Support Services Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in.- [Blows per ft]	N_{60} (ASTM)	PP or TV, tsf	LABORATORY DATA					REMARKS
										Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	
	20 21		medium toughness fines; 20% fine sand.		S09A	100								SG	S09A_020_022T S09A_020_022T SG = 2.62
-10	22 23 24		At 22.0 feet, 6 inch lens of sandy lean clay (CL). SANDY ORGANIC SILT (OH); medium stiff; black (7.5YR 2.5/1); moist; 70% medium dry strength, slow dilatancy, medium toughness fines; 30% fine sand.		S10A	100			0.5P 0.5P	145	65	7		OC	S10A_022_025P Switch to Rotary Wash S10A_022_025P OC = 23.7%
	25 26		LEAN CLAY with Sand (CL); very soft; very dark greenish gray (10BG 3/1); wet; 80% medium plasticity, medium dry strength, no dilatancy, medium toughness fines; 20% fine sand; with organics.		S11A	100	0 0 0 [0]	0	0P	89					S11A_025_027S S11A_025_027S
-15	27 28 29		LEAN CLAY (CL); stiff; very dark greenish gray (10BG 3/1); wet; 93% high dry strength, no dilatancy, medium toughness fines; 7% fine sand.		S12A	44			1.25P		44	20	93		S12A_027_030P S12A_027_030P
	30 31		SILTY SAND (SM); loose; very dark greenish gray (10BG 3/1); wet; 75% fine to medium sand; 25% fines.		S13A	100	1 3 3 [6]	8		32					S13A_030_032S S13A_030_032S
-20	32 33 34		SILT with Sand (ML); very dark greenish gray (10BG 3/1); wet; 74% low plasticity, low dry strength, slow dilatancy, low toughness fines; 26% fine sand. At 33.5 feet, 6 inch lens of poorly graded sand with silt (SP-SM).		S14A	89							74		S14A_032_035P S14A_032_035P
	35 36		Poorly Graded SAND with Silt (SP-SM); medium dense; bluish black (5B 2.5/1); wet; 90% fine to medium sand; 10% fines.		S15A	100	2 4 4 [8]	11							S15A_035_037S Vibrating wire piezometer installed at 35 feet S15A_035_037S
-25	37 38 39		CLAYEY SAND (SC); greenish black (5GY 2.5/1); wet; 69% fine to medium sand; 31% fines. SILTY SAND (SM); very dark greenish gray (10BG 3/1); wet; 75% fine to medium sand; 25% fines. At 38.5 feet, 4 inch lens of poorly graded sand with silt (SP-SM).		S16A	86					28	9	31		S16A_037_040P S16A_037_040P
-30	40 41 42 43 44 45		Total depth drilled 40 feet. Boring backfilled with neat cement grout: 2 bags (94 lb.) Portland Cement 20 lbs. of bentonite 50 gallons of water												

Draft 3 After All Lab Data Added 5/16/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,178,670.55 **Easting:** 6,310,266.10
Latitude: 37.97505 **Longitude:** -121.37220
Levee Station or Milepost: 117+51 **Levee Mile:**
Levee Segment Brookside
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: San Joaquin River

LOG OF BORING
WR2074_003M
Sheet 2 of 2

Engineering Support Services Urban Levee Geotechnical Evaluations Program

DATE STARTED 10/14/11 **DATE COMPLETED** 10/14/11	**GROUND ELEVATION** 14.60 ft **ELEVATION DATUM** NAVD 88	**TOTAL DEPTH OF BORING** 51.5 ft
DRILLING CONTRACTOR Pitcher Drilling Inc.	**DRILLER'S NAME** James Musich **HELPER'S NAME** Malakai Fakalolo	**TOTAL DEPTH OF FILL** 7.5 ft
DRILLING METHOD HSA/Rotary Wash	**DRILL RIG MAKE AND MODEL** CME 55	**CONSULTANT COMPANY** Kleinfelder
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) 8" HSA, 3-7/8" Drag Bit	**DRILLING ROD TYPE AND DIAMETER** NWJ 2-5/8"	**FIELD LOGGER** M. Luna
☒ **VERTICAL** ☐ **INCLINED**	**CASING TYPE, DIAMETER, INSTALLATION DEPTH** HSA, 10" OD, 15 ft	**FIELD LOG REVIEWER** M. Briseno
SAMPLER TYPE(S) StdCal(2.5"), SPT (1.375"), DM (2.5")	**HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP** CME Auto Hammer 140 lbs/30-inch drop	**HAMMER EFFICIENCY** 77%
BOREHOLE BACKFILL OR COMPLETION Grout	**GROUNDWATER READING: DURING DRILLING** Not Measured Due to Drilling Method	**AFTER DRILLING (DATE-TIME)**

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
	0	▨	LEAN CLAY (CL); reddish gray (10R 5/1); moist; 100% medium plasticity, medium dry strength, no dilatancy, medium toughness fines; [Levee fill].												Hand Auger to 5 feet
10	2				S01A										S01A_002_004B
	5	▨	FAT CLAY (CH); very stiff; dark brown (10YR 3/3); moist; 85% high dry strength, no dilatancy, high toughness fines; 15% fine to coarse, subangular sand; [Levee fill].		S02A	61			2.5P	38 36	71	52		UW UW	S02A_005_007T Switch to Hollow-Stem Auger D&M pushed with 250 psi
←FILL	7.5		SILT (ML); dark brown (10YR 3/3); moist; 100% low plasticity, low dry strength, slow to rapid dilatancy, low toughness fines.		S03A	100			1.5P 0.75P	24				UW	S03A_007_009T D&M pushed with 300 psi
5	9		Below 9.0 feet, very dark grayish brown (2.5Y 3/2); 4 inch lens of poorly graded sand (SP); trace of organics (<5%).												
	10				S04A	97			0.75P 0.25P	35 49	49	25		UW UW	S04A_010_012T D&M pushed with 250 psi Switched to mud rotary
	10.5	▨	LEAN CLAY (CL); soft; very dark gray (10YR 3/1); moist; 100% medium dry strength, no dilatancy, medium toughness fines. Below 11.25 feet, dark grayish brown (10YR 4/2);.												
0	12.5		Below 12.5 feet, medium stiff.		S05A	97			0.50P 0.60P	37 39 39	43 42	20 20		UW UW UW	S05A_012_014T D&M pushed with 200 psi
	15	▨	FAT CLAY (CH); medium stiff; greenish black (5BG 2.5/1); moist; 95% high dry strength, no dilatancy, high toughness fines; 5% fine sand.		S06A	100			0.75P 0.75P	49 40 46	51 54	25 31		UW UW UW	S06A_015_017T Switch to Rotary Wash D&M pushed with 275 psi
	17	▨	FAT CLAY (CH); stiff; black (10YR 2/1); moist; 100% high dry strength, no dilatancy, high toughness fines; with organics. At 18.4 feet, lense of ELASTIC SILT(MH). Below 19.0 feet, medium stiff.		S07A	100			1.25P 0.75P	68 66 69 65 72	85 97 88 92	58 59 49 55		UW UW UW UW UW	S07A_017_019T D&M pushed with 300 psi
-5	20				S08A	100			0.5P						S08A_020_022T

Draft 3 After All Lab Data Added 5/16/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,178,721.48 **Easting:** 6,310,263.88
Latitude: 37.97518 **Longitude:** -121.37221
Levee Station or Milepost: 118+02 **Levee Mile:**
Levee Segment Brookside
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: Tenmile Slough

LOG OF BORING
WR2074_009B

Sheet 1 of 3

Engineering Support Services Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA					REMARKS
										Water Content, %	Liquid Limit	Plasticity Index	Fines, % <#200	Other Lab Tests	
	20 21 22				S08A	100			0.75P	108 75	102	63		UW UW	D&M pushed with 200 psi
	23 24		ORGANIC ELASTIC SILT (OH); stiff; black (10YR 2/1); moist; 100% medium dry strength, slow dilatancy, medium toughness fines.		S09A	100			0.5P 1.0P	108 81	96	52		UW OC	S09A_022_024T D&M pushed with 300 psi OC = 8.7%
-10	25		PEAT (PT); black (10YR 2/1); wet.		S10A	100			1.0P 0.75P	217 102 93	97 84	56 45		UW UW UW	S10A_025_027T D&M pushed with 250 psi
	26 27		ORGANIC ELASTIC SILT (OH); medium stiff; black (10YR 2/1); moist; 100% medium dry strength, slow dilatancy, medium toughness fines.												
	28 29		LEAN CLAY (CL); medium stiff; dark greenish gray (10GY 4/1); 100% medium dry strength, no dilatancy fines. Below 29.0 feet, stiff.		S11A	100			0.75P 1.75P	38 34 30	48 49	31 31		UW UW UW	S11A_027_029T D&M pushed with 300 psi
-15	30		Below 30.0 feet, medium stiff.		S12A	100			0.75P 3.5P	27	27	8		UW	S12A_030_032T D&M pushed with 300 psi
	31 32		SANDY SILT (ML); dark greenish gray (10GY 4/1); 70% low plasticity, low dry strength, slow to rapid dilatancy, low to medium toughness fines; 30% fine sand.												
	33 34		SILTY SAND (SM); dark greenish gray (10GY 4/1); 81% fine sand; 19% fines.		S13B S13A	72	3 4 5 [9]			29 30			19 19	UW HD PA UW HD PA	S13B_033_034C S13A_034-034C
-20	35 36		35.0 to 36.5 feet, medium dense.		S14A	56	2 4 6 [10]	13							S14A_035_037S
	37 38 39		Poorly Graded SAND (SP); greenish gray (10GY 5/1); wet; 96% fine to medium sand; 4% fines.		S15B S15A	89	6 8 7 [15]			23			4	UW PA	S15B_038_039C S15A_039_039C
-25	40 41		40.0 to 41.5 feet, medium dense.		S16A	67	1 4 7 [11]	14							S16A_040_042S
	42 43 44		Poorly Graded SAND with Silt (SP-SM); greenish gray (10GY 5/1); moist; 90% sand; 10% fines. At 43.5 feet, 6 inch lens of LEAN CLAY (CL).		S17B S17A		2 5 6 [11]			34			10	UW HD PA	S17B_043_044C S17A_044_044C
-30	45														

Draft 3 After All Lab Data Added 5/16/2012

Borehole Location: Levee Crest	County: San Joaquin
Coordinates: Northing: 2,178,721.48	Easting: 6,310,263.88
Latitude: 37.97518	Longitude: -121.37221
Levee Station or Milepost: 118+02	Levee Mile:
Levee Segment Brookside	
Survey Method: Ground Survey	Coord. System: CA State Plane Zone III
Channel / River Name / Feature: Tenmile Slough	

LOG OF BORING

WR2074_009B

Sheet 2 of 3

Engineering Support Services Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60} (ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
	45		Below 45.0 feet, loose.		S18A	61	3 4 2 [6]	8							S18A_045_047S
	46														
	47		<u>SANDY LEAN CLAY</u> (CL); very stiff; dark greenish gray (10GY 4/1); 70% medium dry strength, no dilatancy, medium toughness fines; 30% fine sand.												
	48				S19A	97			3.0P 4.5P	21	40	21		UW	S19A_047_049T D&M pushed with 300 psi
	49		Below 49.0 feet, hard.												
-35	50		<u>LEAN CLAY</u> (CL); hard; dark greenish gray (10GY 4/1); 100% medium plasticity, medium dry strength, no dilatancy, medium toughness fines.												
	51				S20A	64			4.5P						S20A_050_052T D&M pushed with 300 psi
	52		Total depth drilled 51.5 feet. Boring backfilled with neat cement grout: 8 bags (47 lb.) Portland Cement 32 gallons of water												
	53														
	54														
-40	55														
	56														
	57														
	58														
	59														
-45	60														
	61														
	62														
	63														
	64														
-50	65														
	66														
	67														
	68														
	69														
-55	70														

Draft 3 After All Lab Data Added 5/16/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,178,721.48 **Easting:** 6,310,263.88
Latitude: 37.97518 **Longitude:** -121.37221
Levee Station or Milepost: 118+02 **Levee Mile:**
Levee Segment Brookside
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: Tenmile Slough

LOG OF BORING
WR2074_009B
Sheet 3 of 3

Engineering Support Services Urban Levee Geotechnical Evaluations Program

DATE STARTED 8/16/10	**DATE COMPLETED** 8/17/10	**GROUND ELEVATION** 15.18 ft	**ELEVATION DATUM** NAVD 88	**TOTAL DEPTH OF BORING** 50.0 ft
DRILLING CONTRACTOR Neil O. Anderson		**DRILLER'S NAME** Mike Young	**HELPER'S NAME** Sean McNeil	**TOTAL DEPTH OF FILL** 13.5 ft
DRILLING METHOD HSA/Rotary Wash		**DRILL RIG MAKE AND MODEL** CME 75		**CONSULTANT COMPANY** Kleinfelder
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) 10" HSA, 4-7/8" Drag Bit		**DRILLING ROD TYPE AND DIAMETER** NWJ 2-5/8"		**FIELD LOGGER** G. Lenehan
☒ **VERTICAL** ☐ **INCLINED**		**CASING TYPE, DIAMETER, INSTALLATION DEPTH** HSA 10" OD		**FIELD LOG REVIEWER** G. Lenehan
SAMPLER TYPE(S) SPT (1.375"), PC (2.5")		**HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP** CME Auto Hammer 140 lbs/30-inch drop		**HAMMER EFFICIENCY** 79%
BOREHOLE BACKFILL OR COMPLETION Grout		**GROUNDWATER READING: DURING DRILLING** Not Measured Due to Drilling Method		**AFTER DRILLING (DATE-TIME)**

Elevation, feet	Depth, feet	CLASSIFICATION OF MATERIALS (Description)	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA: Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
15	0	SAND AND GRAVEL at surface [Levee Fill]. LEAN CLAY with Sand (CL); very stiff; very dark gray (7.5YR 3/1); moist; 75% medium plasticity, high dry strength, no dilatancy, medium toughness fines; 25% fine sand; weak reaction with HCl; [Levee Fill].	S01A	100	6 7 7 [14]	18	2.75P						S01A_000_002S Hollow-Stem Auger to 15 feet
	2	Below 2.0 feet, black (5YR 2.5/1.	S02A	100									S02A_002_005P
	4	FAT CLAY (CH); hard; very dark grayish brown (10YR 3/2); moist; 87% high dry strength, no dilatancy, high toughness fines; 13% fine sand; [Levee Fill].					4.5P		52	25	87		
10	5	Below 5.0 feet, very stiff, black (2.5Y 2.5/1).	S03A	100	3 5 8 [13]	17	2.25P	18					S03A_005_007S
	7		S04A	100									S04A_007_010P
	8	LEAN CLAY (CL); hard; very dark greenish gray (5GY 3/1); moist; 90% medium dry strength, no dilatancy, medium toughness fines; 10% fine sand; [Levee Fill].							30	16			
	9.5	At 9.5 feet, 6 inch lens of clayey sand (SC).											
5	10	Below 10.5 feet, very dark greenish gray (10Y 3/1).	S05A	100	3 5 6 [11]	14	4.5P	16					S05A_010_012S
	12	Below 12.0 feet, very stiff.	S06A	100			3.5P 3.5P						S06A_012_015P
	13.5	←FILL LEAN CLAY with Sand (CL); very stiff; dark greenish gray (10Y 4/1); moist; 75% high dry strength, no dilatancy, medium toughness fines; 25% fine sand.											
0	15	Below 16.0 feet, black (N 1.5/).	S07A	100	3 6 9 [15]	20	3.75P	21	32	13			S07A_015_017S Switch to Rotary Wash
	17	LEAN CLAY with Sand soft; dark greenish gray (10Y 4/1); moist; 75% high dry strength, no dilatancy, medium toughness fines; 25% fine sand; with organics. At 18.0 feet, wet.	S08A	53			0.5P					OC	S08A_017_020P OC = 7.2%
	19.5	Below 19.5 feet, very stiff.					3.5P	30	37	16			
	20												

Draft 3 After All Lab Data Added 5/16/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,179,929.44 **Easting:** 6,311,172.33
Latitude: 37.97853 **Longitude:** -121.36909
Levee Station or Milepost: 133+82 **Levee Mile:**
Levee Segment Brookside
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: Tenmile Slough

LOG OF BORING

WR2074_007B

Sheet 1 of 3

Engineering Support Services Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	CLASSIFICATION OF MATERIALS (Description)	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60} (ASTM)	PP or TV, tsf	LABORATORY DATA: Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
-5	20		S09A	67								SG	S09A_020_022T SG = 2.55
	22	ELASTIC SILT (MH); medium stiff; greenish black (10Y 2.5/1); wet; 100% low dry strength, no dilatancy, low toughness fines; with organics. Below 24.0 feet, moist.	S10A	69			0.5P	82	77	31		OC	S10A_022_025P OC = 13.4%
-10	25	CLAYEY SAND (SC); medium dense; dark bluish gray (5B 4/1); moist; 70% fine to medium sand; 30% fines.	S11A	100	2 3 5 [8]	11	1.5P	28				HD	S11A_025_027S
	27	SILT with Sand (ML); dark greenish gray (5BG 4/1); moist; 78% low dry strength, slow dilatancy, low toughness fines; 22% fine sand.	S12A	72			2.0P		32	7	78		S12A_027_030P
-15	30	SILTY SAND (SM); medium dense; very dark greenish gray (10BG 3/1); moist; 51% fine sand; 49% fines.	S13A	100	7 10 13 [23]	30		25					S13A_030_032S
	32	Below 32.0 feet, very dark greenish gray (5BG 3/1).	S14A	83							49	PA	S14A_032_035P
-20	35	CLAYEY SAND (SC); medium dense; very dark greenish gray (5BG 3/1); moist; 80% fine sand; 20% fines.	S15A	88	2 3 5 [8]	11		25	29	12			S15A_035_037S
	37	SANDY SILT (ML); very dark greenish gray (5BG 3/1); moist; 53% low plasticity, low to medium dry strength, no to slow dilatancy, low toughness fines; 47% fine sand.	S16A	89							53		S16A_037_040P
-25	40	40.0 to 41.5 feet, medium dense.	S17A	92	5 8 9 [17]	22		29					S17A_040_042S
	42	Below 42.0 feet, 67% fines; 33% sand.	S18A	89							67		S18A_042_045P
	45												

Draft 3 After All Lab Data Added 5/16/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,179,929.44 **Easting:** 6,311,172.33
Latitude: 37.97853 **Longitude:** -121.36909
Levee Station or Milepost: 133+82 **Levee Mile:**
Levee Segment Brookside
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: Tenmile Slough

LOG OF BORING
WR2074_007B
Sheet 2 of 3

Engineering Support Services Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60} (ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
-30	45		Below 45.0 feet, very loose; very dark greenish gray (5BG 3/1).		S19A	100	1 1 2 [3]	4		33					S19A_045_047S
	46														
	47		SILTY SAND (SM); greenish black (5GY 2.5/1); wet; 79% fine to medium sand; 21% fines.		S20A	89							21	PA	S20A_047_050P
	48														
	49														
-35	50														

Total depth drilled 50 feet.
Boring backfilled with neat cement grout:
7 bags (47 lb.) Portland Cement
50 gallons of water

Elevation, feet	Depth, feet
	51
	52
	53
	54
-40	55
	56
	57
	58
	59
-45	60
	61
	62
	63
	64
-50	65
	66
	67
	68
	69
	70

Draft 3 After All Lab Data Added 5/16/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,179,929.44 **Easting:** 6,311,172.33
Latitude: 37.97853 **Longitude:** -121.36909
Levee Station or Milepost: 133+82 **Levee Mile:**
Levee Segment Brookside
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: Tenmile Slough

LOG OF BORING

WR2074_007B

Sheet 3 of 3

Engineering Support Services Urban Levee Geotechnical Evaluations Program

DWR LEVEE UNIJ SOIL LOG REV1 BROOKSIDE BORINGS.GPJ DWR OFFICIAL LIBRARY 02/23/2012.GLB 5/20/12

DATE STARTED 2/6/09 **DATE COMPLETED** 2/6/09	**GROUND ELEVATION** 3.27 ft **ELEVATION DATUM** NAVD 88	**TOTAL DEPTH OF BORING** 32.0 ft
DRILLING CONTRACTOR Neil O Anderson	**DRILLER'S NAME** James Young **HELPER'S NAME** Sean McNeil	**TOTAL DEPTH OF FILL** 8 ft
DRILLING METHOD HSA/Rotary Wash	**DRILL RIG MAKE AND MODEL** CME 75	**CONSULTANT COMPANY** Kleinfelder
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) 10" HSA, 5" Punch Core	**DRILLING ROD TYPE AND DIAMETER** NWJ 2-5/8 OD; PC 3" ID, 3-1/2 OD	**FIELD LOGGER** M. Shubert
☒ **VERTICAL** ☐ **INCLINED**	**CASING TYPE, DIAMETER, INSTALLATION DEPTH** HSA: 10" O.D./6" I.D. Case to 5'	**FIELD LOG REVIEWER** A. Kilinger
SAMPLER TYPE(S) SPT 2" O.D./1-3/8" I.D., 2-1/2" Punch Core	**HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP** CME Auto Hammer, 140 lbs / 30-inch drop	**HAMMER EFFICIENCY** 84%
BOREHOLE BACKFILL OR COMPLETION Grout	**GROUNDWATER READING:** **DURING DRILLING** 7 ft	**AFTER DRILLING (DATE-TIME)**

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Blows per 6 in. [Blows per ft]	N_{60} (ASTM)	PP or TV, tsf	LABORATORY DATA					Piezometer Installation Schematic	REMARKS AND PIEZOMETER INSTALLATION NOTES
									Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests		
	0		CLAYEY SAND (SC); dark brown (7.5YR 3/2); moist; 40% fines; 60% fine sand; [Fill].												S01A_000_005P
	1														
	2		Poorly Graded GRAVEL with Clay and Sand (GP-GC); brown (10YR 4/3); moist; 10% fines; 60% fine, subrounded gravel; 30% fine to coarse, angular to subangular sand; strong reaction with HCl; [Fill].		S01A										
0	3		FAT CLAY with Sand (CH); very stiff; black (5YR 2.5/1); moist; 84% high dry strength, no dilatancy, high toughness fines; 16% fine sand; [Fill].												
	4							3.5P	17	57	39	84	HD		
	5		LEAN CLAY with Sand (CL); yellowish red (5YR 5/6); moist; 80% medium dry strength, no dilatancy, medium toughness fines; 20% fine sand; [Fill].												S02A_005_007S Switched to mud rotary OC = 2.6%
	6				S02A	5 11 15 [26]	36		19	38	21		OC		
FILL	7 ∇														S03A_007_010P
-5	8		CLAYEY SAND (SC); yellowish red (5YR 5/6); moist; 34% fines; 66% fine to medium sand.		S03A							34	PA		
	9														
	10							3.0P							S04A_010_012S
	11		LEAN CLAY (CL); very stiff; grayish green (5G 5/2); moist; 85% medium dry strength, slow dilatancy, medium toughness fines; 15% fine sand.		S04A	6 9 10 [19]	27	4.5P	23	37	20				
	12														S05A_012_015P
-10	13				S05A					38	19				
	14		SILTY SAND (SM); medium dense; reddish brown (5YR 5/4); wet; 73-84% fine sand; 16-27% fines.												
	15														S06A_015_017S
	16				S06A	7 8 10 [18]	25					16	PA		
	17														S07A_017_020P SG = 2.74
-15	18				S07A							24	PA SG		
	19														
	20														

Final Report Version 10/29/2009

DWR LEVEE UNU SOIL + PIEZ LOG REV1 LINCOLN VILLAGE BORINGS.GPJ DWR OFFICIAL LIBRARY 02/22/2012.GLB 6/26/12

Borehole Location: Levee Toe (Landside) **County:** San Joaquin
Coordinates: Northing: 2,192,612.86 **Easting:** 6,312,861.86
Latitude: 38.01330 **Longitude:** -121.36358
Levee Station or Milepost: 43+00 **Levee Mile:**
Levee Segment Lincoln Village
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: Fivemile Slough

LOG OF BORING
WR1608_002M

Sheet 1 of 2

Engineering Support Services Urban Levee Geotechnical Evaluations Program

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	Piezometer Installation Schematic	REMARKS AND PIEZOMETER INSTALLATION NOTES
	20–22		At 20 feet, loose.		S08A	2 3 4 [7]	10					27			S08A_020_022S
-20	22–25				S09A										S09A_022_025P
	25–27		Poorly Graded SAND with Silt (SP-SM); medium dense; reddish brown (5YR 4/3); wet; 8% fines; 92% fine sand.		S10A	3 4 4 [8]	11					8			S10A_025_027S Vibrating wire piezometer installed at 24 feet
-25	27–30				S11A										S11A_027_030P
	30–32		LEAN CLAY with Sand (CL); dark bluish gray (10B 4/1); moist; 80% medium plasticity, medium dry strength, slow dilatancy, medium toughness fines; 20% fine sand.		S12A	3 4 5 [9]	13								S12A_030_032S

Total Depth Drilled 32 Feet.
Boring backfilled with neat cement grout.

Elevation, feet: -30, -35, -40
Depth, feet: 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45

Final Report Version 10/29/2009

Borehole Location: Levee Toe (Landside) **County:** San Joaquin **Coordinates: Northing:** 2,192,612.86 **Easting:** 6,312,861.86 **Latitude:** 38.01330 **Longitude:** -121.36358 **Levee Station or Milepost:** 43+00 **Levee Mile:** **Levee Segment** Lincoln Village **Survey Method:** Ground Survey **Coord. System:** CA State Plane Zone III **Channel / River Name / Feature:** Fivemile Slough	LOG OF BORING WR1608_002M Sheet 2 of 2 Engineering Support Services Urban Levee Geotechnical Evaluations Program

DWR LEVEE UNU SOIL + PIEZ LOG REV1 LINCOLN VILLAGE BORINGS.GPJ DWR OFFICIAL LIBRARY 02232012.GLB 6/26/12

DATE STARTED 11/1/11 **DATE COMPLETED** 11/1/11	**GROUND ELEVATION** 13.00 ft **ELEVATION DATUM** NAVD 88	**TOTAL DEPTH OF BORING** 40.0 ft
DRILLING CONTRACTOR Pitcher Drilling Inc.	**DRILLER'S NAME** James Musich **HELPER'S NAME** Malakai Fakalolo	**TOTAL DEPTH OF FILL** 12 ft
DRILLING METHOD HSA/Rotary Wash	**DRILL RIG MAKE AND MODEL** CME 55	**CONSULTANT COMPANY** Kleinfelder
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) 8" HSA, 3-7/8" Drag Bit	**DRILLING ROD TYPE AND DIAMETER** NWJ 2-5/8"	**FIELD LOGGER** G. Lenehan
☒ **VERTICAL** ☐ **INCLINED**	**CASING TYPE, DIAMETER, INSTALLATION DEPTH** HSA, 10" OD, 15 ft.	**FIELD LOG REVIEWER** M. Briseno
SAMPLER TYPE(S) StdCal(2.5"), SPT (1.375"), DM (2.5")	**HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP** CME Auto Hammer, 140 lbs / 30-inch drop	**HAMMER EFFICIENCY** 77%
BOREHOLE BACKFILL OR COMPLETION Grout	**GROUNDWATER READING:** **DURING DRILLING** Not Measured Due to Drilling Method	**AFTER DRILLING (DATE-TIME)**

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
	0		Asphalt concrete - 3 inches. Aggregate base - 4 inches. LEAN CLAY (CL); hard; black (2.5Y 2.5/1); moist; 90% high dry strength, no dilatancy, medium toughness fines; [Levee Fill].												
	1														Hand Auger to 4 feet
	2				S01A										S01A_002_003B
10	3														
	4				S02A	61			>4.5P	22	40	20		UW	S02A_004_006T Switch to hollow-stem auger at 4 ft. D&M pushed with 450 psi
	5														
	6														
	7														
5	8				S03A	72			>4.5P						S03A_008_010T D&M pushed with 450 psi
	9														
	10														
	11		Below 11.0 feet, stiff.		S04A	89			1.25P 1.5P						S04A_011_013T D&M pushed with 350 psi
↑FILL	12		LEAN CLAY (CL); stiff; black (2.5Y 2.5/1); moist; 100% high dry strength, no dilatancy, medium toughness fines.												
0	13														
	14				S05A	61			3.25P	22	40	26		UW OC	S05A_014_016T D&M pushed with 300 psi OC = 4.2% Began rotary wash at 15 ft.
	15		Below 15.0 feet, very stiff; olive brown (2.5Y 4/4).												
	16														
	17		LEAN CLAY (CL); very stiff; dark greenish gray (10GY 4/1); moist; 90% medium plasticity, medium dry strength, no dilatancy, medium toughness fines; 10% fine sand.		S06A	67			2.0P						S06A_017_019T D&M pushed with 300 psi
-5	18														
	19														
	20														

Final Report Version 5/25/2012

Borehole Location: Levee Crest **County:** San Joaquin **Coordinates: Northing:** 2,187,777.13 **Easting:** 6,311,545.68 **Latitude:** 38.00012 **Longitude:** -121.36798 **Levee Station or Milepost:** 109+90 **Levee Mile:** **Levee Segment** Lincoln Village **Survey Method:** Ground Survey **Coord. System:** CA State Plane Zone III **Channel / River Name / Feature:** Fourteenmile Slough	**LOG OF BORING** **WR1608_008B** **Sheet 1 of 2** **Engineering Support Services Urban Levee Geotechnical Evaluations Program**

DWR LEVEE UNU SOIL LOG REV1 LINCOLN VILLAGE BORINGS.GPJ DWR OFFICIAL LIBRARY 022012.GLB 6/26/12

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60} (ASTM)	PP or TV, tsf	LABORATORY DATA					REMARKS
										Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	
	20				S07A	50			2.75P						S07A_020_022T D&M pushed with 250 psi
	22		SANDY LEAN CLAY (CL); medium stiff; dark greenish gray (10GY 4/1); moist; 53% medium dry strength, no dilatancy, medium toughness fines; 47% fine sand.												
-10	23				S08A	100			0.5P	24	31	15	53	UW	S08A_023_025T D&M pushed with 350 psi
	24		SANDY SILT (ML); very dark greenish gray (5GY 3/1); moist; 60% low plasticity, low dry strength, rapid dilatancy, low toughness fines; 40% fine sand.												
	26		Poorly Graded SAND with Silt (SP-SM); dark greenish gray (10GY 4/1); wet; 90% fine to medium sand; 10% fines.		S09B S09A	78	6 9 10 [19]			25			10	UW HD PA PE	S09B_026_027C K = 5.7E-04 cm/sec S09A_027_028C
-15	28														
	29		SILT (ML); dense; very dark greenish gray (10GY 3/1); moist; 100% low plasticity, low dry strength, slow dilatancy, low toughness fines.		S10A	72	3 11 14 [25]	32							S010A_028_030S
	31		SILTY SAND (SM); dark greenish gray (10GY 4/1); wet; 67% fine to medium sand; 33% fines.		S11B S11A	83	5 9 9 [18]			18			33	UW HD PA PE	S11B_031_032C K = 2.8E-06 cm/sec S11A_032_032C
-20	33		32.5 to 34.0 feet, medium dense.		S12A	78	4 9 11 [20]	26							S12A_032_034S
	35		Poorly Graded SAND with Silt (SP-SM); dark greenish gray (10GY 4/1); wet; 89% fine to medium sand; 11% fines.		S13B S13A	78	9 13 17 [30]			20			11	UW	S13B_035_036C S13A_036_036C
	37		36.5 to 38.0 feet, very dense.		S14A	56	13 24 21 [45]	58							S14A_036_038S
-25	38														
	39		SILTY SAND (SM); dark greenish gray (10GY 4/1); wet; 84% fine to medium sand; 16% fines.		S15B S15A	78	17 27 29 [56]			16			18	UW	S15B_039_040C S15A_040_040C
	40		Total Depth Drilled 40 Feet. Boring backfilled with neat cement grout: 8 bags (94 lb.) Portland Cement 30 gallons of water												
-30	43														
	45														

Final Report Version 5/25/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,187,777.13 **Easting:** 6,311,545.68
Latitude: 38.00012 **Longitude:** -121.36798
Levee Station or Milepost: 109+90 **Levee Mile:**
Levee Segment Lincoln Village
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: Fourteenmile Slough

LOG OF BORING
WR1608_008B
Sheet 2 of 2

Engineering Support Services Urban Levee Geotechnical Evaluations Program

DWR LEVEE UNU SOIL LOG REV1 LINCOLNVILLAGEBORINGS.GPJ DWR OFFICIAL LIBRARY 02082012.GLB 6/26/12

DATE STARTED 11/15/11 · **DATE COMPLETED** 11/15/11	**GROUND ELEVATION** 2.40 ft · **ELEVATION DATUM** NAVD 88	**TOTAL DEPTH OF BORING** 41.5 ft
DRILLING CONTRACTOR Pitcher Drilling Inc.	**DRILLER'S NAME** James Musich · **HELPER'S NAME** Malakai Fakalolo	**TOTAL DEPTH OF FILL** 0 ft
DRILLING METHOD Rotary Wash	**DRILL RIG MAKE AND MODEL** CME 55	**CONSULTANT COMPANY** Kleinfelder
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) 3-7/8" Drag Bit	**DRILLING ROD TYPE AND DIAMETER** NWJ 2-5/8"	**FIELD LOGGER** G. Lenehan
☒ **VERTICAL** ☐ **INCLINED**	**CASING TYPE, DIAMETER, INSTALLATION DEPTH** HSA, 10" OD, 15 ft.	**FIELD LOG REVIEWER** M. Briseno
SAMPLER TYPE(S) StdCal(2.5"), SPT (1.375"), DM (2.5")	**HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP** CME Auto Hammer, 140 lbs / 30-inch drop	**HAMMER EFFICIENCY** 77%
BOREHOLE BACKFILL OR COMPLETION Grout	**GROUNDWATER READING: DURING DRILLING** Not Measured Due to Drilling Method	**AFTER DRILLING (DATE-TIME)**

Elevation, feet	Depth, feet	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA: Water Content, %	Liquid Limit	Plasticity Index	Fines, % <#200	Other Lab Tests	REMARKS
	0	Asphalt concrete - 3 inches. Aggregate base - 6 inches.												
	1	SANDY LEAN CLAY (CL); black (10YR 2/1); moist; 70% medium plasticity, medium dry strength, no dilatancy, medium toughness fines; 30% fine sand.		S01A										S01A_002_003B Hand Auger to 5 feet
0	2													
	4	LEAN CLAY (CL); stiff; dark greenish gray (5GY 4/1); moist; 60% high dry strength, no dilatancy, medium toughness fines; 40% fine sand.												
	5			S02A	100			1.5P	28	40	25		UW	S02A_006_007T Switch to hollow-stem auger at 5 ft. D&M pushed with 300 psi
	6	SILTY SAND (SM); dark greenish gray (5GY 4/1); moist; 54% fine sand; 46% fines.												
-5	7													
	8			S03A	94				29			48	UW	S03A_008_010T D&M pushed with 200 psi
	11	At 11.0 feet, 6 inch lens of Poorly Graded SAND With SILT (SP-SM).		S04A	89				31			11 19	UW HD PA PE	S04A_011_013T D&M pushed with 100 psi K = 1.0E-03 cm/sec
-10	11.5	SILTY SAND (SM); dark greenish gray (5GY 4/1); wet; 81% fine sand; 19% fines.												
	14			S05A	89									S05A_014_016T D&M pushed with 300 psi
-15	17	Poorly Graded SAND with Silt (SP-SM); dark greenish gray (5GY 4/1); wet; 90% fine to medium sand; 10% fines.		S06B S06A	83	4 9 10 [19]						10		S06B_017_018C S06A_018_019C
	20													

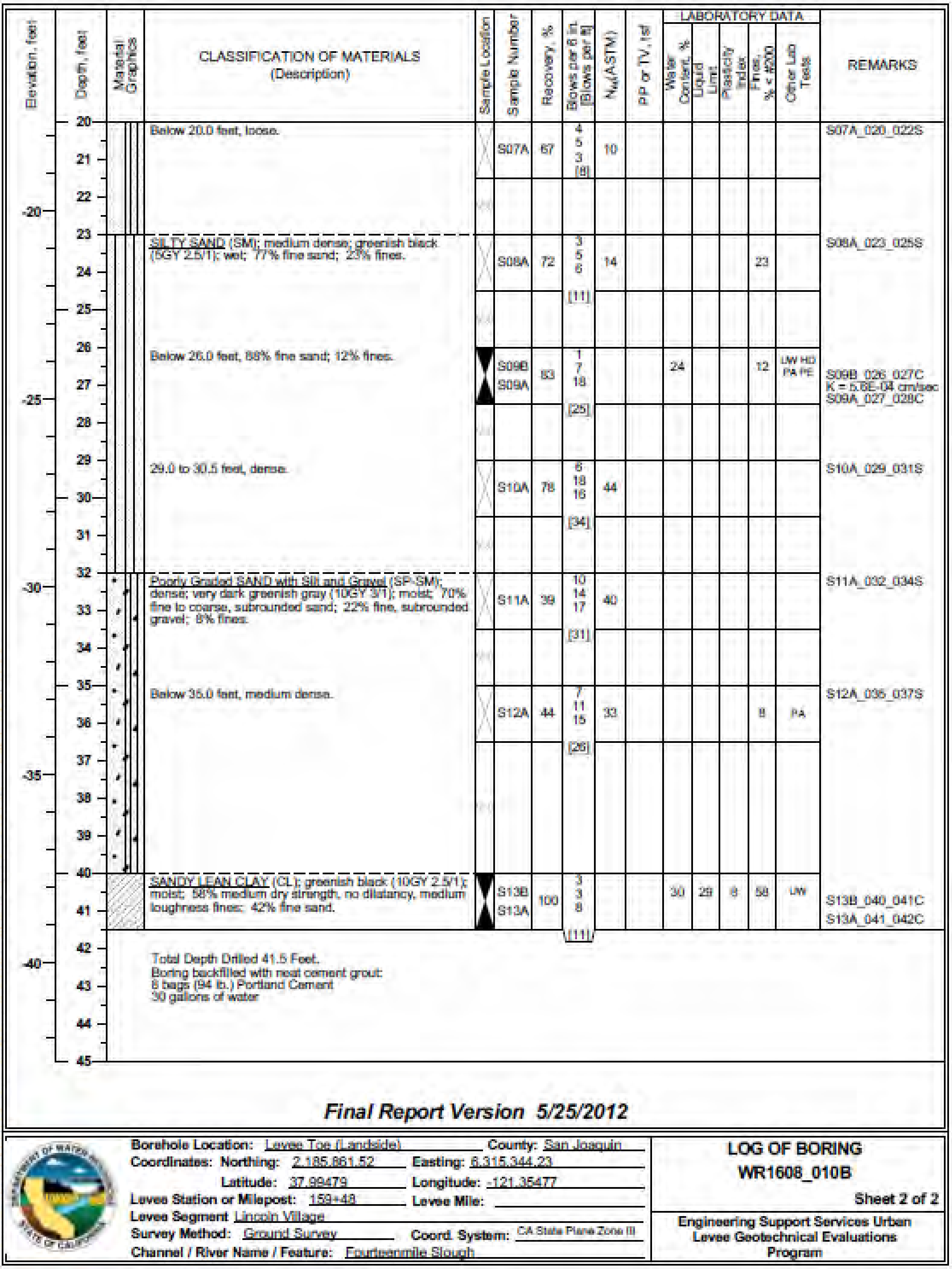
Final Report Version 5/25/2012

Borehole Location: Levee Toe (Landside) County: San Joaquin
Coordinates: Northing: 2,185,861.52 Easting: 6,315,344.23
Latitude: 37.99479 Longitude: -121.35477
Levee Station or Milepost: 159+48 Levee Mile:
Levee Segment Lincoln Village
Survey Method: Ground Survey Coord. System: CA State Plane Zone III
Channel / River Name / Feature: Fourteenmile Slough

LOG OF BORING
WR1608_010B

Sheet 1 of 2

Engineering Support Services Urban Levee Geotechnical Evaluations Program



Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
-20	20		Below 20.0 feet, loose.		S07A	67	4 5 3 [8]	10							S07A_020_022S
	23		SILTY SAND (SM); medium dense; greenish black (5GY 2.5/1); wet; 77% fine sand; 23% fines.		S08A	72	3 5 6 [11]	14					23		S08A_023_025S
-25	26		Below 26.0 feet, 88% fine sand; 12% fines.		S09B S09A	83	1 7 18 [25]			24			12	UW HD PA PE	S09B_026_027C K = 5.8E-04 cm/sec S09A_027_028C
	29		29.0 to 30.5 feet, dense.		S10A	78	6 18 16 [34]	44							S10A_029_031S
-30	32		Poorly Graded SAND with Silt and Gravel (SP-SM); dense; very dark greenish gray (10GY 3/1); moist; 70% fine to coarse, subrounded sand; 22% fine, subrounded gravel; 8% fines.		S11A	39	10 14 17 [31]	40							S11A_032_034S
-35	35		Below 35.0 feet, medium dense.		S12A	44	7 11 15 [26]	33					8	PA	S12A_035_037S
	40		SANDY LEAN CLAY (CL); greenish black (10GY 2.5/1); moist; 58% medium dry strength, no dilatancy, medium toughness fines; 42% fine sand.		S13B S13A	100	3 3 8 [11]			30	29	8	58	UW	S13B_040_041C S13A_041_042C

Total Depth Drilled 41.5 Feet.
Boring backfilled with neat cement grout:
8 bags (94 lb.) Portland Cement
30 gallons of water

-40 42 43 44 45

Final Report Version 5/25/2012

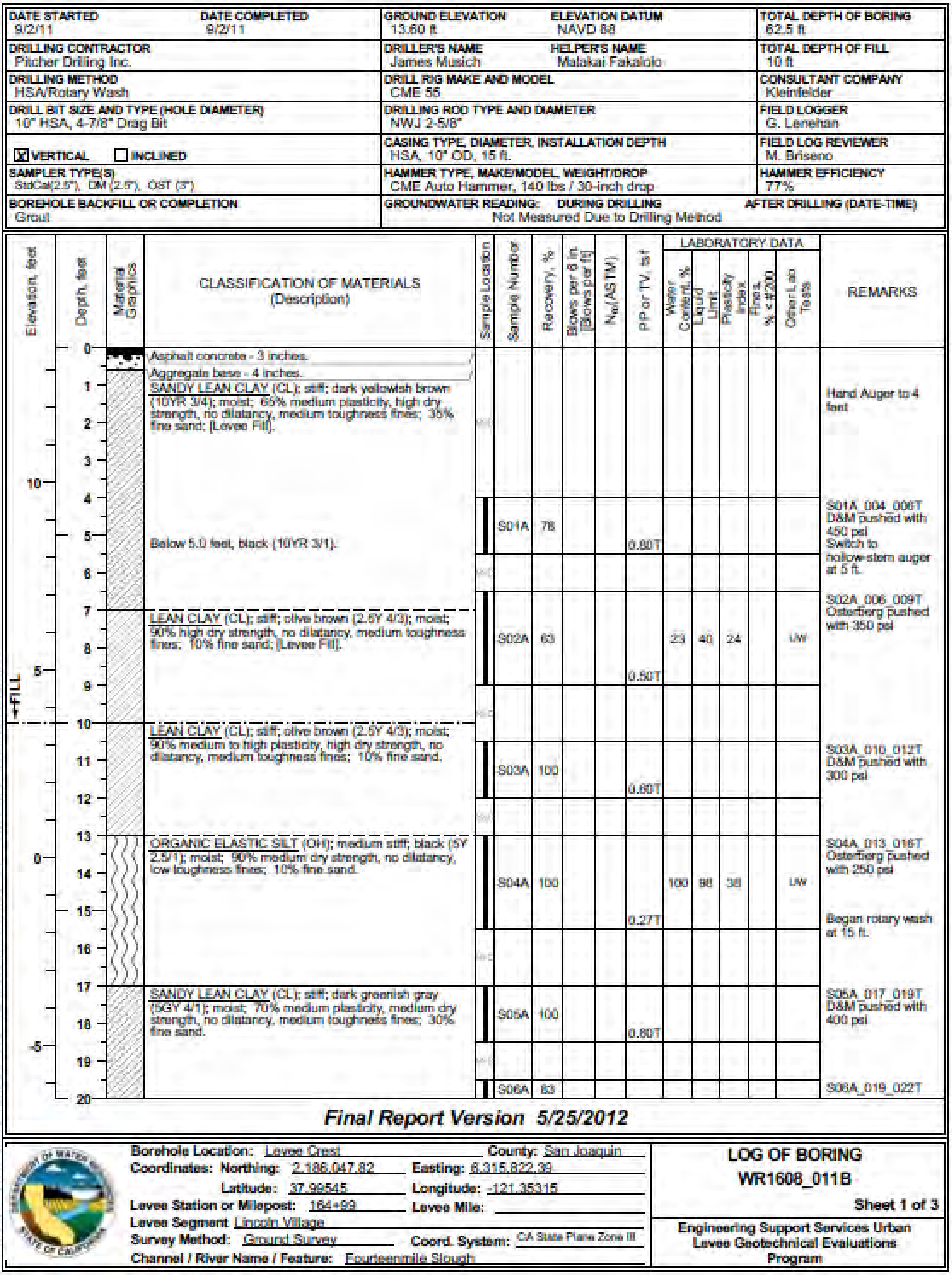
Borehole Location: Levee Toe (Landside) County: San Joaquin
Coordinates: Northing: 2,185,861.52 Easting: 6,315,344.23
Latitude: 37.99479 Longitude: -121.35477
Levee Station or Milepost: 159+48 Levee Mile:
Levee Segment Lincoln Village
Survey Method: Ground Survey Coord. System: CA State Plane Zone III
Channel / River Name / Feature: Fourteenmile Slough

LOG OF BORING
WR1608_010B

Sheet 2 of 2

Engineering Support Services Urban Levee Geotechnical Evaluations Program

DWR LEVEE UAU SOIL LOGS REV1 LINCOLN VILLAGE BORINGS.GPJ DWR OFFICIAL LIBRARY 020120 12.GLB 6/29/12



DATE STARTED 9/2/11 **DATE COMPLETED** 9/2/11	**GROUND ELEVATION** 13.60 ft **ELEVATION DATUM** NAVD 88	**TOTAL DEPTH OF BORING** 62.5 ft
DRILLING CONTRACTOR Pitcher Drilling Inc.	**DRILLER'S NAME** James Musich **HELPER'S NAME** Malakai Fakalolo	**TOTAL DEPTH OF FILL** 10 ft
DRILLING METHOD HSA/Rotary Wash	**DRILL RIG MAKE AND MODEL** CME 55	**CONSULTANT COMPANY** Kleinfelder
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) 10" HSA, 4-7/8" Drag Bit	**DRILLING ROD TYPE AND DIAMETER** NWJ 2-5/8"	**FIELD LOGGER** G. Lenehan
☒ **VERTICAL** ☐ **INCLINED**	**CASING TYPE, DIAMETER, INSTALLATION DEPTH** HSA, 10" OD, 15 ft.	**FIELD LOG REVIEWER** M. Briseno
SAMPLER TYPE(S) StdCal(2.5"), DM (2.5"), OST (3")	**HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP** CME Auto Hammer, 140 lbs / 30-inch drop	**HAMMER EFFICIENCY** 77%
BOREHOLE BACKFILL OR COMPLETION Grout	**GROUNDWATER READING: DURING DRILLING** Not Measured Due to Drilling Method	**AFTER DRILLING (DATE-TIME)**

								LABORATORY DATA					
Elevation, feet	Depth, feet	CLASSIFICATION OF MATERIALS (Description)	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	Water Content, %	Liquid Limit	Plasticity Index	Fines, % <#200	Other Lab Tests	REMARKS
	0	Asphalt concrete - 3 inches.											
		Aggregate base - 4 inches.											
10	1	SANDY LEAN CLAY (CL); stiff; dark yellowish brown (10YR 3/4); moist; 65% medium plasticity, high dry strength, no dilatancy, medium toughness fines; 35% fine sand; [Levee Fill].											Hand Auger to 4 feet
	4	Below 5.0 feet, black (10YR 3/1).	S01A	78			0.80T						S01A_004_006T D&M pushed with 450 psi Switch to hollow-stem auger at 5 ft.
5	7	LEAN CLAY (CL); stiff; olive brown (2.5Y 4/3); moist; 90% high dry strength, no dilatancy, medium toughness fines; 10% fine sand; [Levee Fill].	S02A	63			0.50T	23	40	24		UW	S02A_006_009T Osterberg pushed with 350 psi
FILL	10	LEAN CLAY (CL); stiff; olive brown (2.5Y 4/3); moist; 90% medium to high plasticity, high dry strength, no dilatancy, medium toughness fines; 10% fine sand.	S03A	100			0.60T						S03A_010_012T D&M pushed with 300 psi
0	13	ORGANIC ELASTIC SILT (OH); medium stiff; black (5Y 2.5/1); moist; 90% medium dry strength, no dilatancy, low toughness fines; 10% fine sand.	S04A	100			0.27T	100	98	38		UW	S04A_013_016T Osterberg pushed with 250 psi Began rotary wash at 15 ft.
-5	17	SANDY LEAN CLAY (CL); stiff; dark greenish gray (5GY 4/1); moist; 70% medium plasticity, medium dry strength, no dilatancy, medium toughness fines; 30% fine sand.	S05A	100			0.80T						S05A_017_019T D&M pushed with 400 psi
	19		S06A	83									S06A_019_022T
	20												

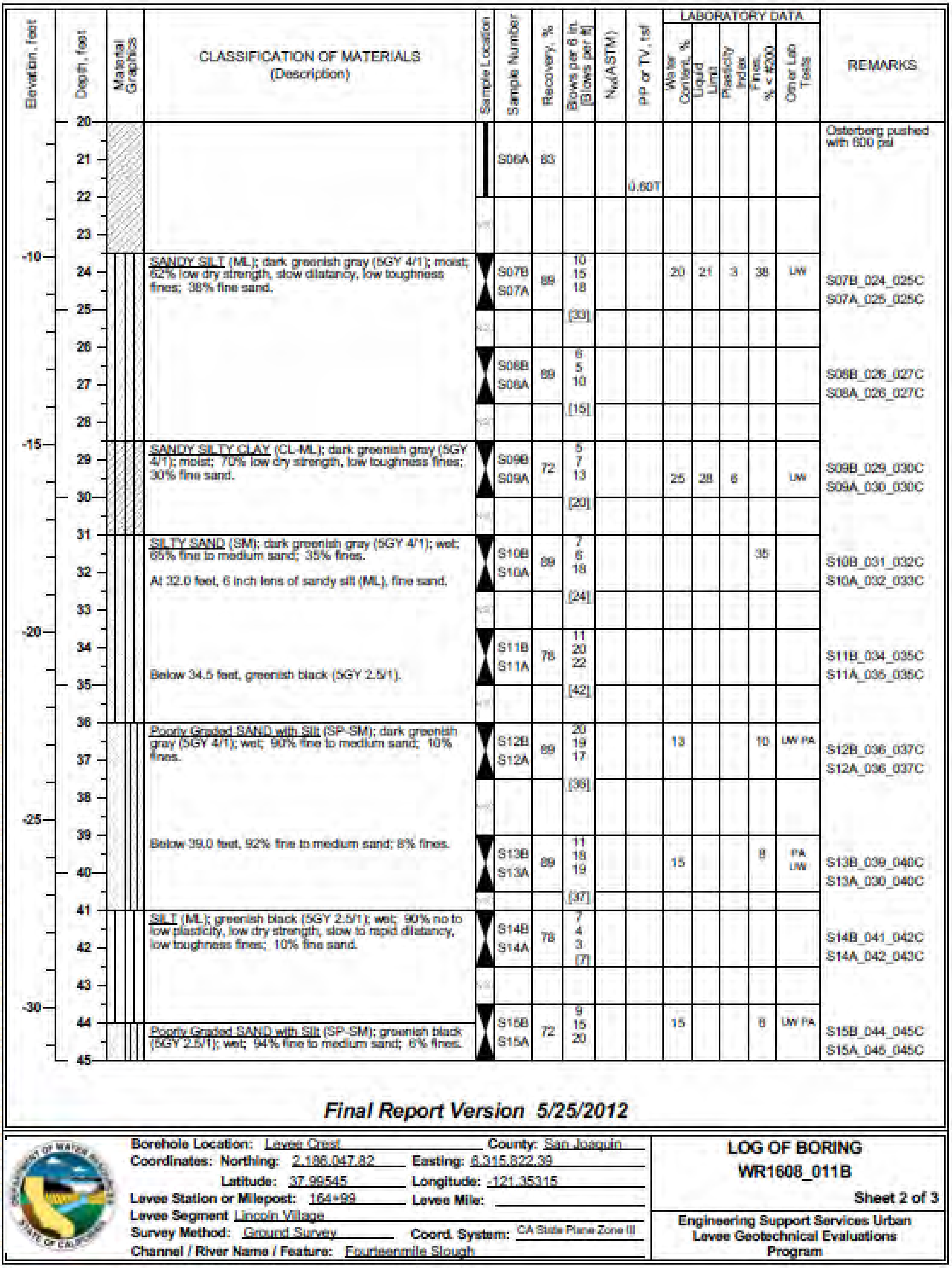
Final Report Version 5/25/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,186,047.82 **Easting:** 6,315,822.39
Latitude: 37.99545 **Longitude:** -121.35315
Levee Station or Milepost: 164+99 **Levee Mile:**
Levee Segment Lincoln Village
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: Fourteenmile Slough

LOG OF BORING
WR1608_011B
Sheet 1 of 3

Engineering Support Services Urban Levee Geotechnical Evaluations Program

DWR LEVEE UNU SOIL LOG REV1 LINCOLN VILLAGE BORINGS.GPJ DWR OFFICIAL LIBRARY [illegible].GLB 6/6/12



Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
	20				S06A	83			0.60T						Osterberg pushed with 600 psi
-10	24		SANDY SILT (ML); dark greenish gray (5GY 4/1); moist; 62% low dry strength, slow dilatancy, low toughness fines; 38% fine sand.		S07B S07A	89	10 15 18 [33]			20	21	3	38	UW	S07B_024_025C S07A_025_025C
	26				S08B S08A	89	6 5 10 [15]								S08B_026_027C S08A_026_027C
-15	29		SANDY SILTY CLAY (CL-ML); dark greenish gray (5GY 4/1); moist; 70% low dry strength, low toughness fines; 30% fine sand.		S09B S09A	72	5 7 13 [20]			25	28	6		UW	S09B_029_030C S09A_030_030C
	31		SILTY SAND (SM); dark greenish gray (5GY 4/1); wet; 65% fine to medium sand; 35% fines. At 32.0 feet, 6 inch lens of sandy silt (ML), fine sand.		S10B S10A	89	7 6 18 [24]						35		S10B_031_032C S10A_032_033C
-20	34		Below 34.5 feet, greenish black (5GY 2.5/1).		S11B S11A	78	11 20 22 [42]								S11B_034_035C S11A_035_035C
	36		Poorly Graded SAND with Silt (SP-SM); dark greenish gray (5GY 4/1); wet; 90% fine to medium sand; 10% fines.		S12B S12A	89	20 19 17 [36]			13			10	UW PA	S12B_036_037C S12A_036_037C
-25	39		Below 39.0 feet, 92% fine to medium sand; 8% fines.		S13B S13A	89	11 18 19 [37]			15			8	PA UW	S13B_039_040C S13A_030_040C
	41		SILT (ML); greenish black (5GY 2.5/1); wet; 90% no to low plasticity, low dry strength, slow to rapid dilatancy, low toughness fines; 10% fine sand.		S14B S14A	78	7 4 3 [7]								S14B_041_042C S14A_042_043C
-30	44		Poorly Graded SAND with Silt (SP-SM); greenish black (5GY 2.5/1); wet; 94% fine to medium sand; 6% fines.		S15B S15A	72	9 15 20			15			8	UW PA	S15B_044_045C S15A_045_045C
	45														

Final Report Version 5/25/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,186,047.82 **Easting:** 6,315,822.39
Latitude: 37.99545 **Longitude:** -121.35315
Levee Station or Milepost: 164+99 **Levee Mile:**
Levee Segment Lincoln Village
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: Fourteenmile Slough

LOG OF BORING
WR1608_011B
Sheet 2 of 3

Engineering Support Services Urban Levee Geotechnical Evaluations Program

DWR LEVEE UNU SOIL LOG REV C LINCOLN VILLAGE BORINGS.GPJ DWR OFFICIAL LIBRARY 02232012.GLB 6/8/12

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA					REMARKS
										Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	
	45						[35]								
	46 47		Below 46.0 feet, fine to coarse, subangular sand.		S16B S16A	78	13 13 15								S16B_046_047C S16A_047_048C
-35	48		At 48.5 feet, 4 inch lens of silty sand (SM).				[28]								
	49 50				S17A	67	12 20 19								S17A_049_050C
							[39]								
	51 52		LEAN CLAY with Sand (CL); dark greenish gray (5GY 4/1); wet; 83% medium dry strength, no dilatancy, medium toughness fines; 17% fine sand.		S18B S18A	89	14 17 20			26	48	27	83	UW	S18B_051_052C S18A_052_053C
	53						[37]								
-40	54 55		Below 54.0 feet, 75% fines, 25% sand.		S19B S19A	78	14 18 20								S19B_054_055C S19A_055_056C
-45	56 57 58 59						[38]								
	60 61		Below 60.0 feet, 80% fines, 20% sand.		S20B S20A	89	17 30 41								S20B_060_061C S20A_061_062C
	62						[71]								
-50 -55	63 64 65 66 67 68 69 70		Total Depth Drilled 62.5 Feet. Boring backfilled with neat cement grout: 8 bags (94 lb.) Portland Cement 32 gallons of water												

Final Report Version 5/25/2012

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,186,047.82 **Easting:** 6,315,822.39
Latitude: 37.99545 **Longitude:** -121.35315
Levee Station or Milepost: 164+99 **Levee Mile:**
Levee Segment Lincoln Village
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: Fourteenmile Slough

LOG OF BORING
WR1608_011B

Sheet 3 of 3

Engineering Support Services Urban Levee Geotechnical Evaluations Program

DATE STARTED 5/14/09 **DATE COMPLETED** 5/14/09	**GROUND ELEVATION** 12.99 ft **ELEVATION DATUM** NAVD 88	**TOTAL DEPTH OF BORING** 50.0 ft
DRILLING CONTRACTOR Neil O Anderson	**DRILLER'S NAME** James Young **HELPER'S NAME** Sean McNeil	**TOTAL DEPTH OF FILL** 15 ft
DRILLING METHOD HSA/Rotary Wash	**DRILL RIG MAKE AND MODEL** CME 75	**CONSULTANT COMPANY** Kleinfelder
DRILL BIT SIZE AND TYPE (HOLE DIAMETER) 10" HSA, 5" Punch Core	**DRILLING ROD TYPE AND DIAMETER** NWJ 2-5/8 OD/PC 3" ID, 3-1/2 OD	**FIELD LOGGER** M. Shubert
☒ **VERTICAL** ☐ **INCLINED**	**CASING TYPE, DIAMETER, INSTALLATION DEPTH** HSA: 10" O.D./6" I.D. Case to 15'	**FIELD LOG REVIEWER** A. Killinger
SAMPLER TYPE(S) SPT 2" O.D./1-3/8" I.D., 2-1/2" PC	**HAMMER TYPE, MAKE/MODEL, WEIGHT/DROP** CME Auto Hammer, 140 lbs / 30-inch drop	**HAMMER EFFICIENCY** 84%
BOREHOLE BACKFILL OR COMPLETION Grout	**GROUNDWATER READING:** **DURING DRILLING** 17.5 ft	**AFTER DRILLING (DATE-TIME)**

Elevation, feet	Depth, feet	CLASSIFICATION OF MATERIALS (Description)	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
	0	LEAN CLAY with Sand (CL); dusky red (2.5YR 3/2); moist; 76% low to medium plasticity, medium dry strength, slow dilatancy, medium toughness fines; 24% fine sand; [Levee Fill].	S01A	43				12			78	OC SG	S01A_000_005P OC = 13.2% SG = 2.75
10	3	ORGANIC LEAN CLAY (OL); very stiff; black (10YR 2/1); moist; 100% medium dry strength, no dilatancy, medium toughness fines; strong reaction with HCl; [Levee Fill].					3.0P						
	5		S02A	100	2 4 4 [8]	11		23	43	27			S02A_005_007S
5	7	ORGANIC LEAN CLAY (OL); hard; black (10YR 2/1); moist; 100% medium dry strength, no dilatancy, medium toughness fines; [Levee Fill].	S03A	50			4.5P	17				OC	S03A_007_010P OC = 15.4%
	10		S04A	83			4.5P	20			100	HD SG	S04A_010_012T SG = 2.78
	12												S05A_012_015P
0	12.5	LEAN CLAY (CL); very stiff; weak red (2.5YR 4/2); moist; 100% medium dry strength, no dilatancy, medium toughness fines; [Levee Fill].	S05A	36			2.5P 2.5P		34	16			
←FILL	15	SILTY SAND (SM); medium dense; weak red (2.5YR 4/2); moist; 66-83% fine sand; 17-34% fines.	S06A	100	3 4 5 [9]	13		26			34		S06A_015_017S Switched to mud rotary
-5	17		S07A	39									S07A_017_020P
	20												

Final Report Version 10/29/2009

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,187,482.16 **Easting:** 6,318,764.23
Latitude: 37.99946 **Longitude:** -121.34299
Levee Station or Milepost: 201+51 **Levee Mile:**
Levee Segment Lincoln Village
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: Fourteenmile Slough

LOG OF BORING
WCNBFM_001B

Sheet 1 of 3

Engineering Support Services Urban Levee Geotechnical Evaluations Program

DWR LEVEE UNU SOIL LOG REV1 LINCOLN VILLAGE BORINGS.GPJ DWR OFFICIAL LIBRARY 02202012.GLB 6/6/12

Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
	20 21				S08A	100	5 7 7 [14]	20							S08A_020_022S
-10	22 23 24				S09A	39									S09A_022_025P
	25 26				S10A	100	5 6 7 [13]	18		25			17	PA SG	S10A_025_027S SG = 2.75
-15	27 28 29				S11A	44									S11A_027_030P
	30 31		At 30 feet, loose, 6% fine, subrounded gravel, 76% fine to coarse, subrounded sand, 18% fines.		S12A	100	4 3 4 [7]	10					18	PA	S12A_030_032S
-20	32 33 34		Poorly Graded SAND with Silt (SP-SM); medium dense; weak red (2.5YR 4/2); wet; 90-94% fine sand; 6-10% fines.		S13A	58			0.5P	17			6		S13A_032_035P
	35 36				S14A	100	3 4 4 [8]	11					10	PA	S14A_035_037S
-25	37 38 39				S15A	81									S15A_037_040P
	40 41		Well-Graded SAND with Silt and Gravel (SW-SM); dense; weak red (2.5YR 4/2); moist; 7% fines; 58% fine to coarse, subrounded sand; 35% fine, subrounded to rounded gravel.		S16A	100	11 12 19 [31]	43					7	PA SG	S16A_040_042S SG = 2.68
-30	42 43 44 45				S17A	33									S17A_042_045P

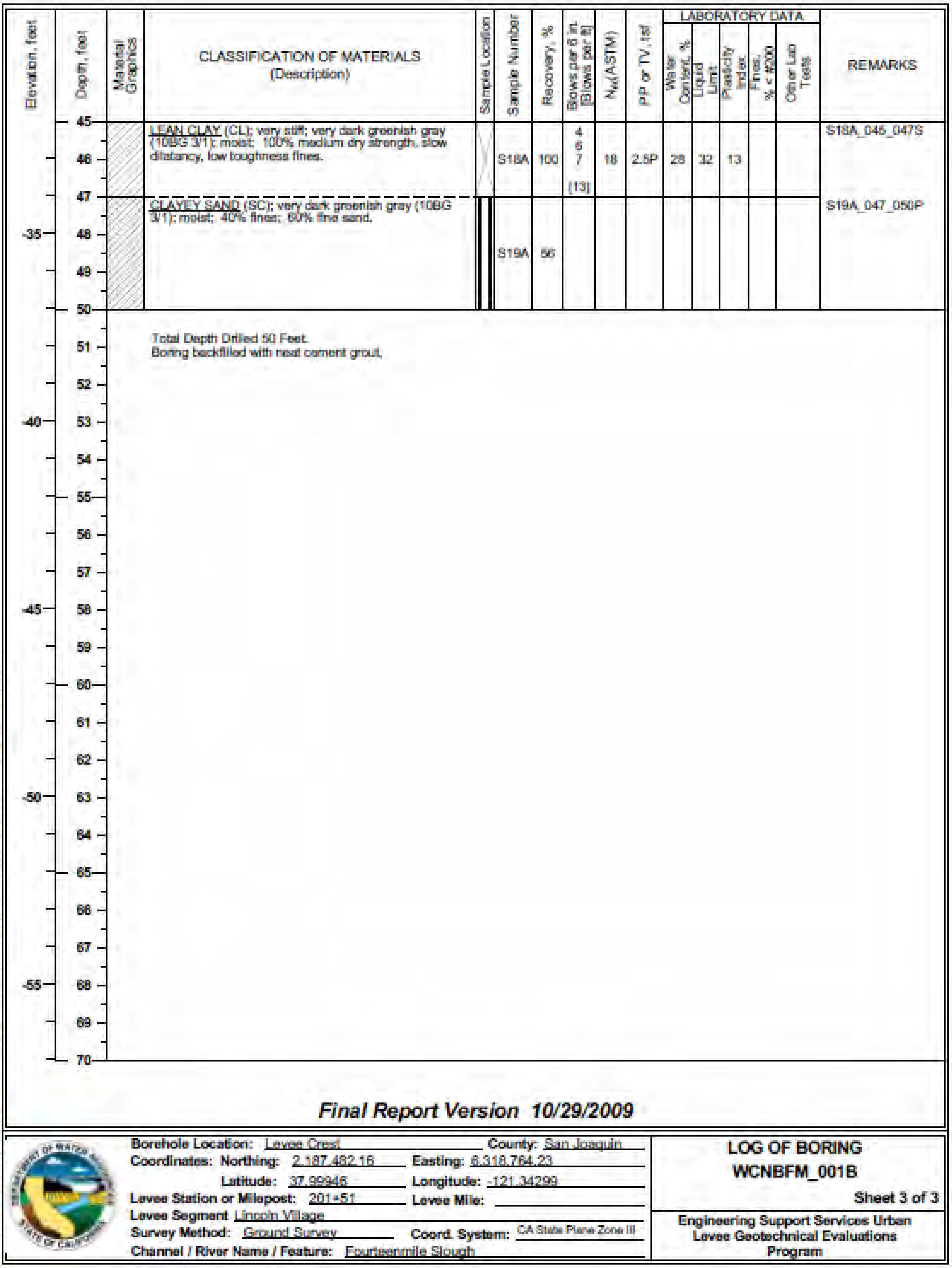
Final Report Version 10/29/2009

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,187,482.16 **Easting:** 6,318,764.23
Latitude: 37.99946 **Longitude:** -121.34299
Levee Station or Milepost: 201+51 **Levee Mile:**
Levee Segment Lincoln Village
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: Fourteenmile Slough

LOG OF BORING
WCNBFM_001B
Sheet 2 of 3

Engineering Support Services Urban Levee Geotechnical Evaluations Program

DWR LEVEE UNU SOIL LOG REV1 LINCOLN VILLAGE BORINGS.GPJ DWR OFFICIAL LIBRARY 022309 12.GLB 6/8/12



Elevation, feet	Depth, feet	Material Graphics	CLASSIFICATION OF MATERIALS (Description)	Sample Location	Sample Number	Recovery, %	Blows per 6 in. [Blows per ft]	N_{60}(ASTM)	PP or TV, tsf	LABORATORY DATA Water Content, %	Liquid Limit	Plasticity Index	Fines, % < #200	Other Lab Tests	REMARKS
	45–47	▨	LEAN CLAY (CL); very stiff; very dark greenish gray (10BG 3/1); moist; 100% medium dry strength, slow dilatancy, low toughness fines.		S18A	100	4 6 7 [13]	18	2.5P	28	32	13			S18A_045_047S
-35	47–50	▨	CLAYEY SAND (SC); very dark greenish gray (10BG 3/1); moist; 40% fines; 60% fine sand.		S19A	56									S19A_047_050P

Total Depth Drilled 50 Feet.
Boring backfilled with neat cement grout,

Final Report Version 10/29/2009

Borehole Location: Levee Crest **County:** San Joaquin
Coordinates: Northing: 2,187,482.16 **Easting:** 6,318,764.23
Latitude: 37.99946 **Longitude:** -121.34299
Levee Station or Milepost: 201+51 **Levee Mile:**
Levee Segment Lincoln Village
Survey Method: Ground Survey **Coord. System:** CA State Plane Zone III
Channel / River Name / Feature: Fourteenmile Slough

LOG OF BORING
WCNBFM_001B
Sheet 3 of 3

Engineering Support Services Urban Levee Geotechnical Evaluations Program

Appendix E

Sensitivity Analysis with Respect to Coincident Water Elevation

General.

The influence on the liquefaction assessment results of the assumed coincident water elevation (CWE) was determined significant:

- Primarily, due to the relative location of potentially liquefiable layers with respect to CWE. If these layers are above CWE they should be considered non-saturated and, therefore, non-liquefiable.
- Secondly, CWE has a major impact on the ratio between the total vertical stress and the effective vertical stress at the depth analyzed for liquefaction. The cyclic stress ratio (CSR) varies in direct proportionality with this ratio:

  $$CSR = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right) r_d$$

  as well as the factor of safety against liquefaction:

  $$FS_{liq} = (CRR_{7.5}/CSR) \cdot MSF \cdot K_\sigma \cdot K_\alpha$$

  Because the stress ratio can roughly vary between 1.0 and 2.0, FS_{liq} may vary between a maximum value when CWE is exactly at the depth of evaluation and half of that when CWE is at the ground surface. In other words, FS_{liq} may be calculated as 1.6 for a low CWE, but can drop below 1.0 if a higher CWE is justified.

The draft ETL “Guidelines for Seismic Evaluation of Levees” includes the following recommendation with respect to CWE selection:

> “The highest of the following three levels should be used to determine the coincident water level for combining with a 100-year return period or a less frequent seismic event (e.g., 200-year or 500-year):
>
> - The median annual water level. This should be the higher of the river level or the groundwater level.
> - The typical seasonal water level. For levees where the impact of failure would be low, the typical seasonal water level should be the average water level during the wettest month of the year, and is preferably a 10-year average (e.g., February for California’s Central Valley levees). For levees where the impact of failure might be severe, 84th percentile of seasonal water level should be considered as the typical seasonal water level.
> - The mean high tide elevation, for levees affected by tides. In these cases, consideration should be given to the predicted sea level rise expected in the decades ahead.
>
> If the coincident water level is at or below the landside levee toe, then the material within the levee embankment does not need to be evaluated for liquefaction susceptibility. Potentially liquefiable materials in the levee embankment or foundation should be

evaluated for liquefaction, if these materials are saturated under the analyzed coincident or analysis water level."

With this study, when information was available, the coincident water level was assumed to be the maximum level in a year without flood event. If this was not available, a conservative assumption of a water level at the ground surface was considered (i.e. unsaturated material in levee and saturated material – therefore potentially liquefiable – in the entire foundation soil).

<u>Example No.1</u>: Boring WR0017_002B – Crest of levee.

This boring is located in Unit RD 17 at Station 1007+42. The liquefaction triggering evaluation at this location is presented in Figure C-9. There are two relatively low SPT blowcounts that may potentially correspond to liquefaction: at elevations 9.2 and -30.8. The ground surface elevation is 12.7 and the top of levee at elevation 20.2. Figure E-1 show the variation of FS_{liq} with the assumed CWE at the two elevations where liquefiability was suspected.

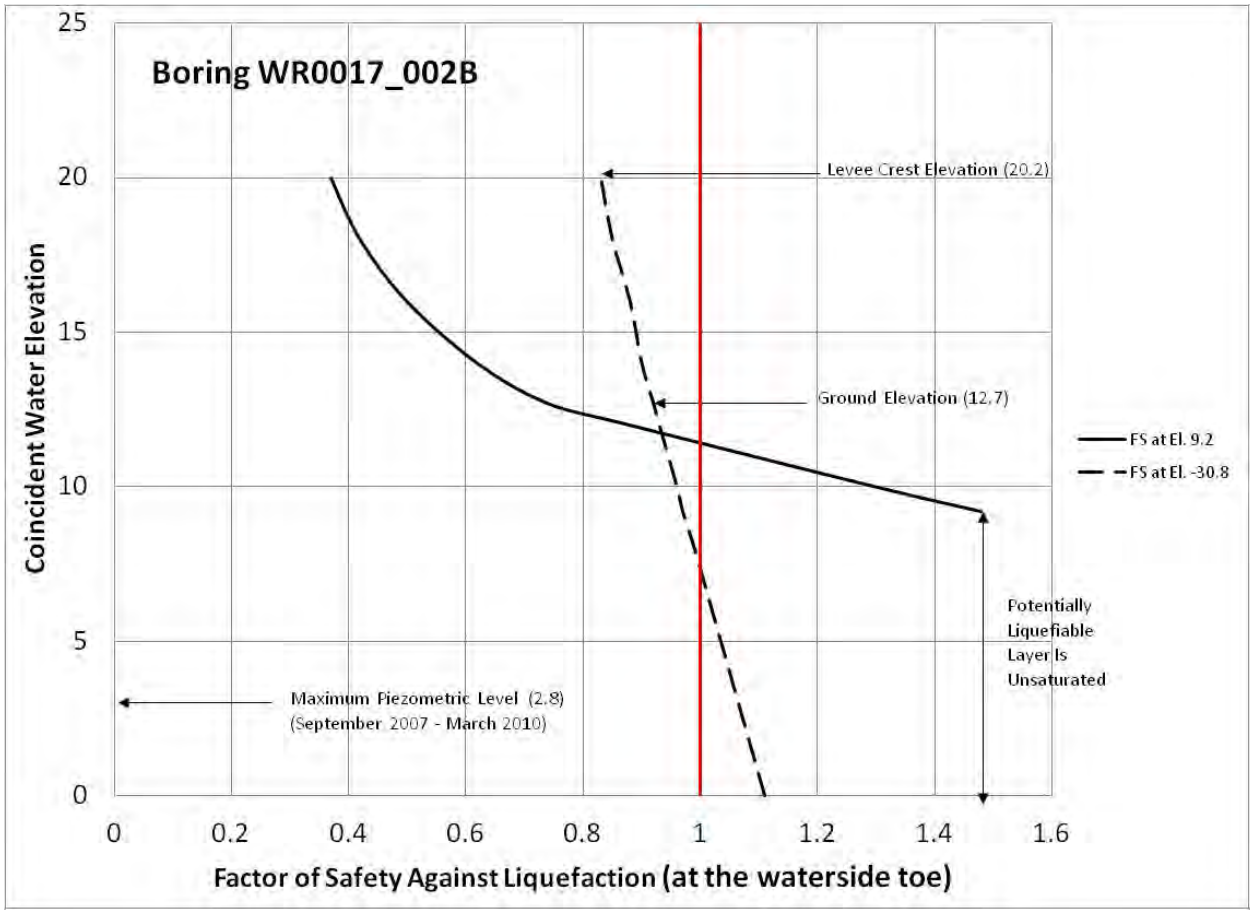


Figure E-1. Variation of FS_{liq} with the assumed CWE at the two elevations in boring WR0017_002B.

The effect of CWE on the calculated FS_{liq} is very important with the shallower potentially liquefiable layer:

- if CWE < 9.2, the layer is non-saturated and, therefore, non-liquefiable;
- for CWE = 9.2, FS_{liq} = 1.48, still non-liquefiable, although saturated;
- with higher CWE, FS_{liq} significantly decreases;
- it becomes FS_{liq} = 0.74 with CWE = 12.7, the ground surface elevation;
- and FS_{liq} = 0.37 with CWE = 20.0, close to top of the levee.

The deeper potentially liquefiable layer is less affected by the CWE selection, but still significantly:

- FS_{liq} = 1.11 for CWE = 0.0;
- FS_{liq} = 0.99 for CWE = 8.0;
- FS_{liq} = 0.92 for CWE = 12.7, the ground surface elevation;
- FS_{liq} = 0.83 for CWE = 20.0, close to top of the levee;

There is a piezometer (WR0017_001M) installed at Station 1048+84, close to the location of interest. Readings were available between September 2007 and March 2010. The maximum ground water level within this interval was 2.8. Assuming CWE = 2.8, it resulted FS_{liq} = 1.07 for the deeper layer; the shallower layer was determined to be well above CWE and, therefore, non-saturated.

Consequently, the location of Boring WR0017_002B was considered non-liquefiable. It is noted that the conservative assumption of water at the ground surface (CWE = 12.7) would imply the conclusion that both two layers were liquefiable.

Example No. 2: Boring WR0017_007B – Crest of levee.

This boring is located in Unit RD 17 at Station 1048+79. The liquefaction triggering evaluation at this location is presented in Figure C-10. There are two relatively low SPT blowcounts that may potentially correspond to liquefaction: at elevations -0.8 and -4.3, probably within the same geologic unit. The ground surface elevation is 8.7 and the top of levee at elevation 21.7. Figure E-2 show the variation of FS_{liq} with the assumed CWE at the two elevations where liquefiability was suspected.

The evaluated location is practically the same where piezometer readings were available: piezometer WR0017_001M installed at Station 1048+84 showed the maximum ground water level within a 2.5-year interval of 2.8. With CWE = 2.8 it resulted FS_{liq} of the order of 1.8 to 2.0 at the potentially liquefiable elevations.

It is noted that considering CWE at the ground surface elevation would still correspond to FS_{liq} in excess of 1.0 at both evaluated depths. Because CWE was credibly defined, this location was not considered seismically vulnerable.

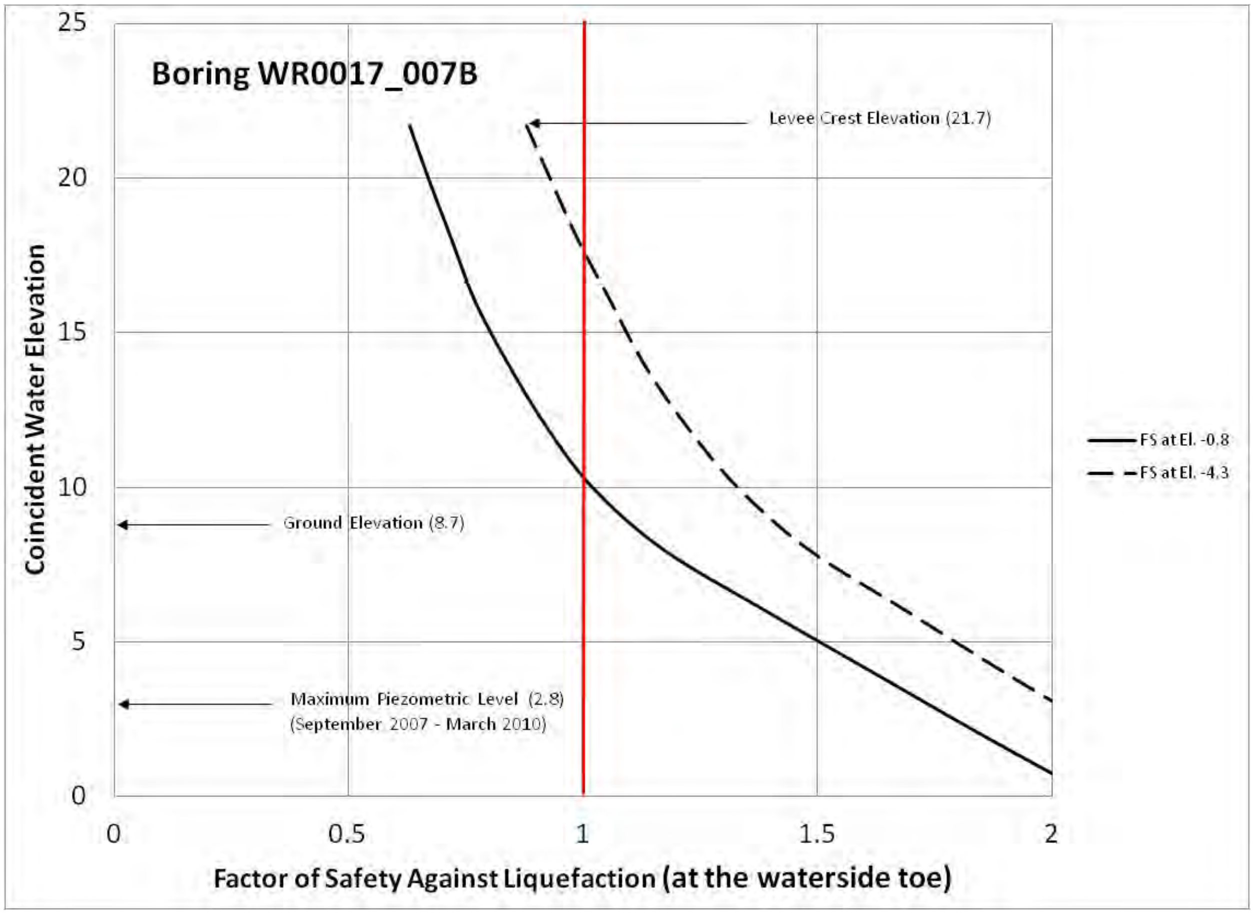


Figure E-2. Variation of FS_{liq} with the assumed CWE at the two elevations in boring WR0017_007B.

<u>Example No. 3</u>: Boring WR0017_041B – Crest of levee.

This boring is located in Unit RD 17 at Station 1330+01. The liquefaction triggering evaluation at this location is presented in Figure C-16. Five depths where SPT blowcounts were available have been examined in detail: 8.2 (not shown in Figure C-16, being in the unsaturated zone), 4.7, -0.3, -5.3, and -10.3. The ground surface elevation is 14.2 and the top of levee at elevation 25.7. Figure E-3 shows the variation of FS_{liq} with the assumed CWE at these five elevations.

The multi-annual maximum piezometric level (no flood events between September 2007 and March 2010) was available in Piezometers WR0017_005M & 006M at Station 1301+04 (maximum water elevation 4.8) and WR0017_ 008M & 009M at Station 1417+01 (maximum water elevation 5.5). The interpolated CWE = 5.0 was considered for Station 1330+01.

From Figure E-3 it is evident that no liquefaction is expected at any depth, with FS_{liq} of at least 1.4. If the CWE at ground elevation had been conservatively assumed, liquefaction would have been predicted at two shallower depths. Assuming all evaluated depths within the same geologic unit, variable CWE would correspond to different thickness of liquefiable layer, as shown in Figure E-4.

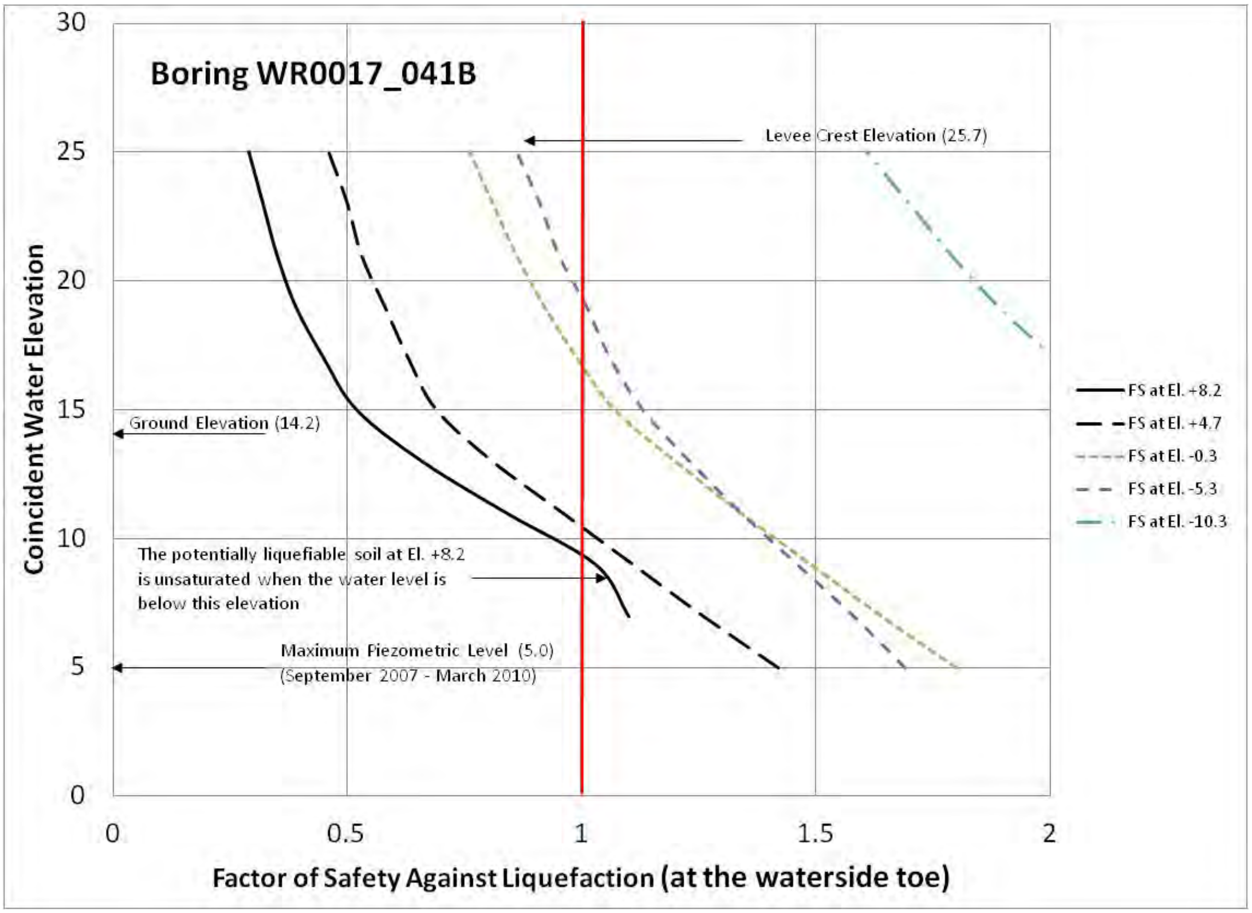


Figure E-3. Variation of FS_{liq} with the assumed CWE at the two elevations in boring WR0017_041B.

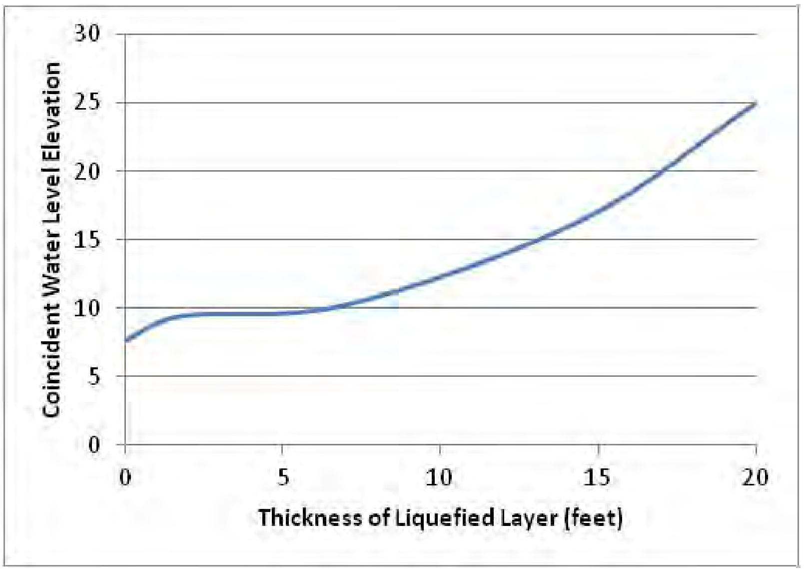


Figure E-4. Effect of assumed CWE on the thickness of layer determined as liquefiable at Boring WR0017_041B location.

<u>Example No. 4</u>: Boring WR0017_047B – Crest of levee.

This boring is located in Unit RD 17 at Station 1377+73. The liquefaction triggering evaluation at this location is presented in Figure C-17. Eight depths located probably within the same geologic unit have been examined in detail. The ground surface elevation is 14.2 and the top of levee at elevation 27.2. Figure E-5 shows the variation of FS_{liq} with the assumed CWE at these eight elevations.

The multi-annual maximum piezometric level (no flood events between September 2007 and March 2010) was available in Piezometers WR0017_005M & 006M at Station 1301+04 (maximum water elevation 4.8) and WR0017_ 008M & 009M at Station 1417+01 (maximum water elevation 5.5). The interpolated CWE = 5.3 was considered for Station 1377+73.

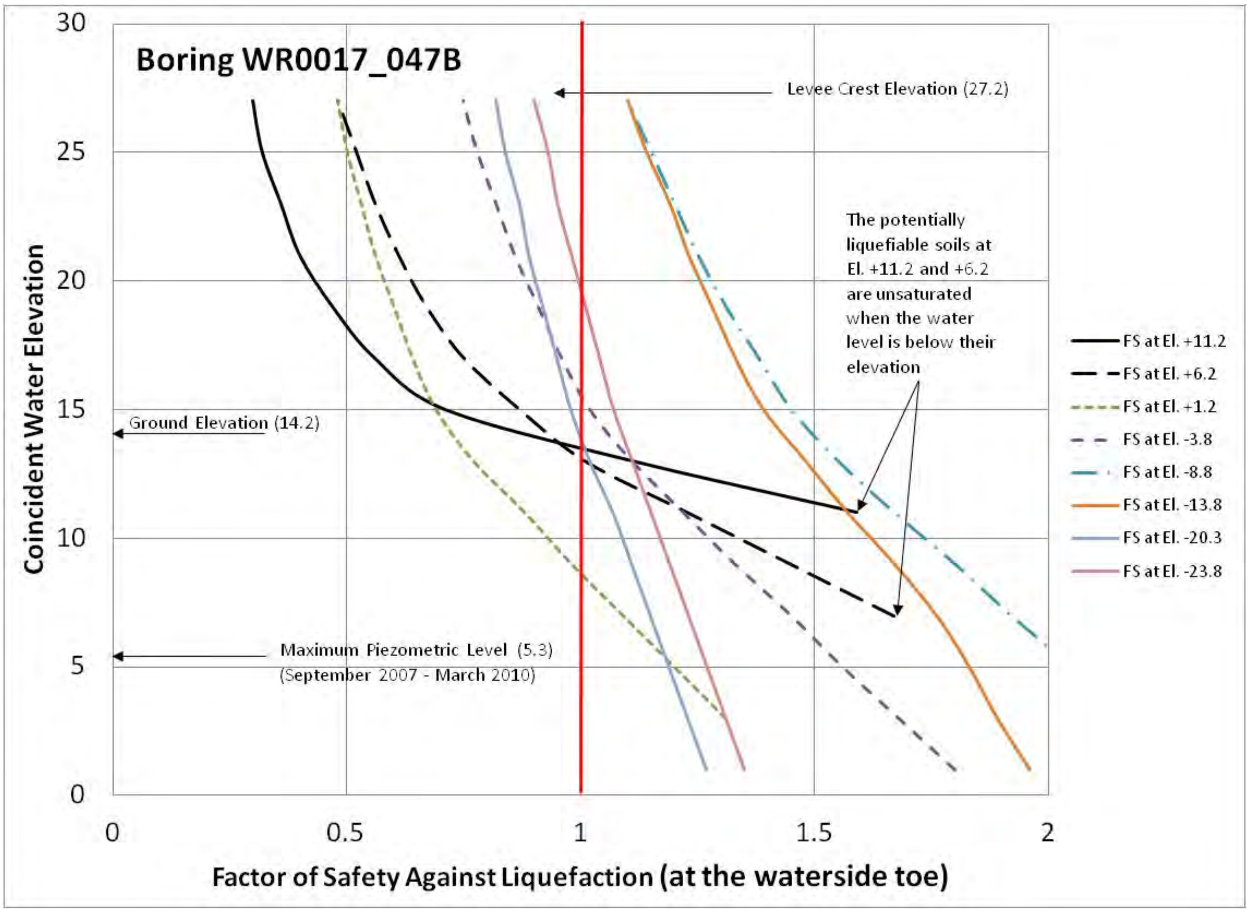


Figure E-5. Variation of FS_{liq} with the assumed CWE at the two elevations in boring WR0017_047B.

No liquefaction was predicted at this location when CWE = 5.3 was considered. However, if CWE = 14.2 (ground surface elevation) were conservatively assumed, a potential liquefiable layer of about 15 feet in thickness would have been assumed.

<u>Example No. 5</u>: Boring WR0017_102B – Crest of levee.

This boring is located in Unit RD 17 at Station 1825+94. The liquefaction triggering evaluation at this location is presented in Figure C-8. Six depths located probably within the same geologic unit have been examined in detail. The ground surface elevation is 14.0 and the top of levee at elevation 34.5. Figure E-6 shows the variation of FS_{liq} with the assumed CWE at these eight elevations.

The multi-annual maximum piezometric level (no flood events between September 2007 and March 2010) was available in Piezometers WR0017_022M & 023M at Station 1784+89 equal to 6.8, which was assumed CWE for Station 1825+94 too.

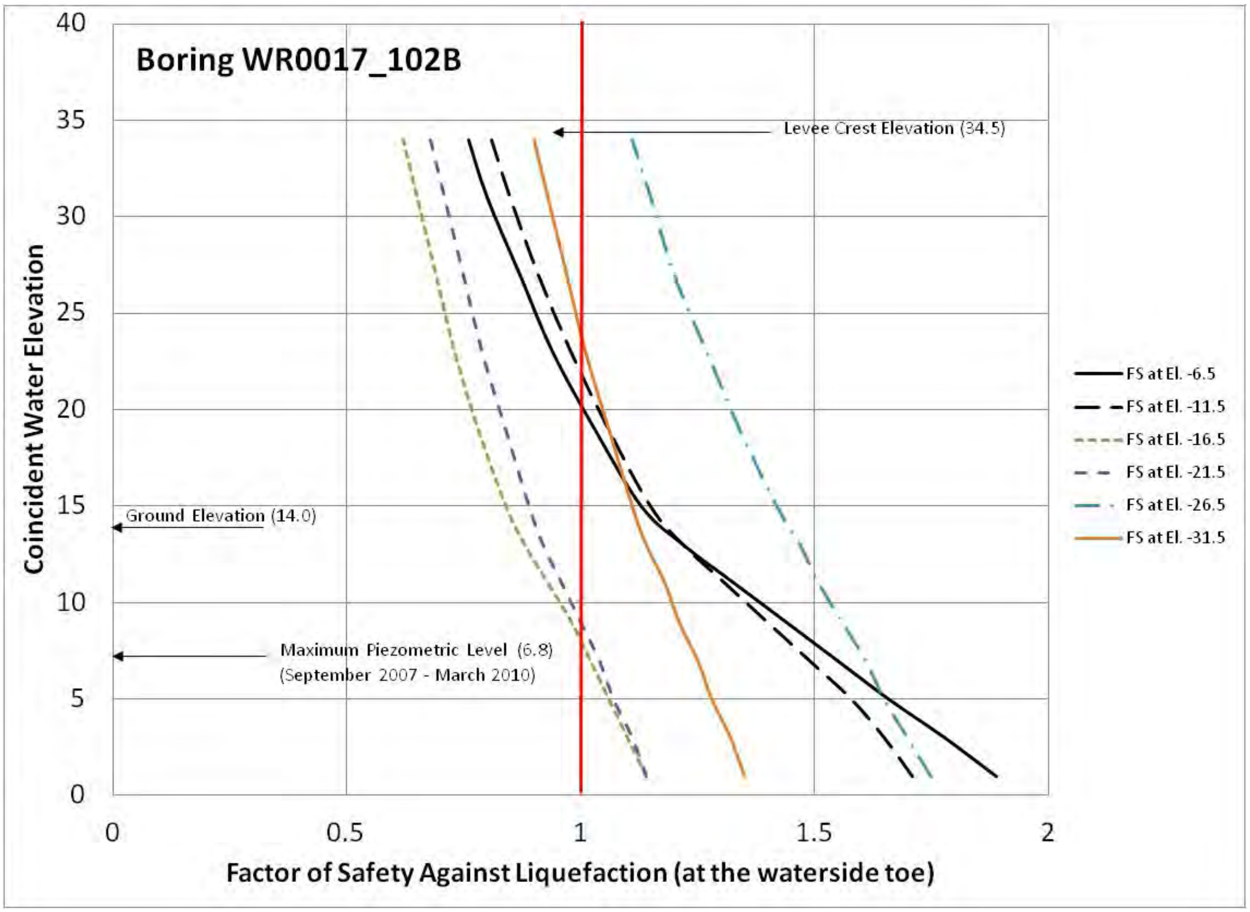


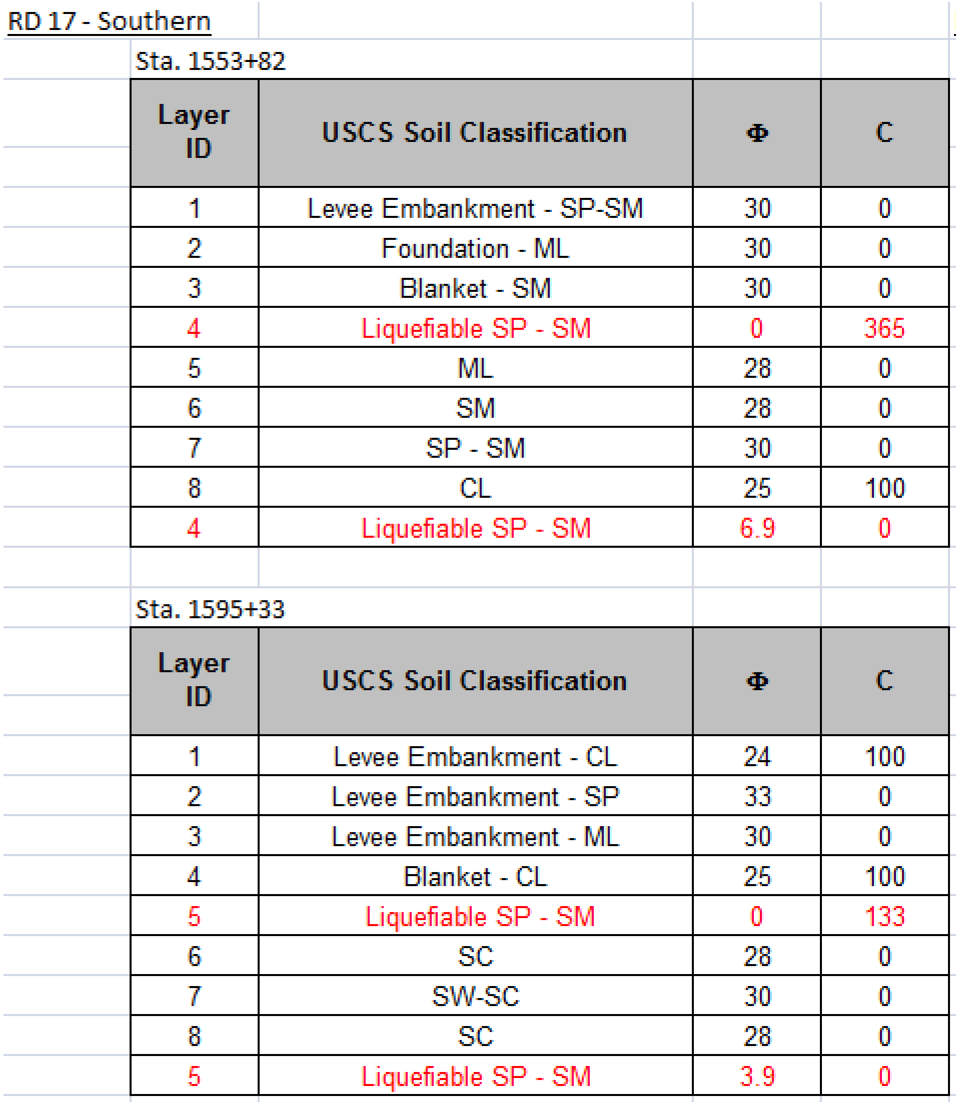
Figure E-6. Variation of FS_{liq} with the assumed CWE at the two elevations in boring WR0017_102B.

For CWE = 6.8 no liquefaction was predicted at this location. It is noted however, that the factor of safety against liquefaction in a 10-foot layer (approximately between elevations -14.0 and -24.0) was of the order of 1.02 – 1.05. With CWE as low as elevation 9.0 (5 feet below the ground surface elevation) liquefaction of this layer would have been predicted.

Appendix F

UTEXAS4 Post-Earthquake Stability Analyses

SOIL PARAMETERS – Post-liquefaction residual strength is shown in red.



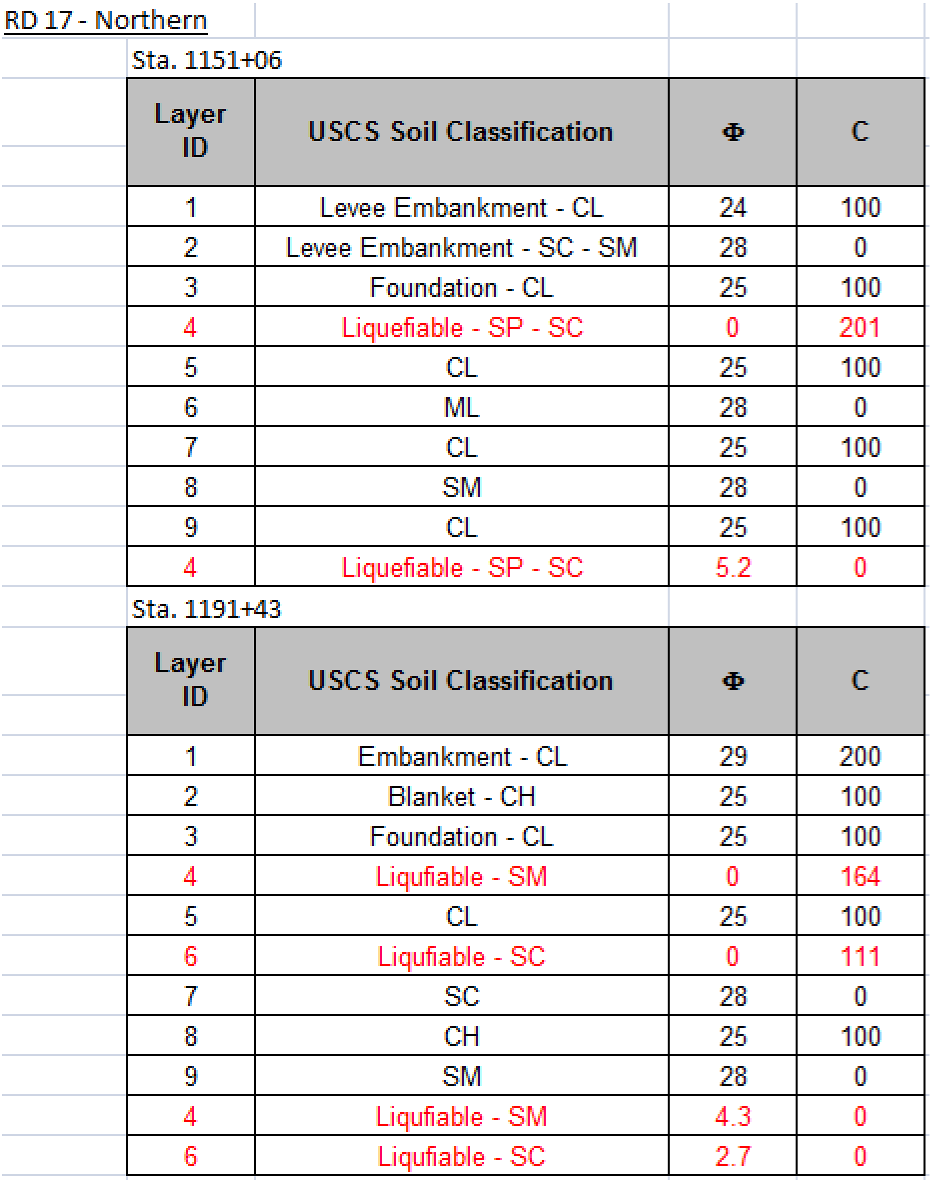
RD 17 - Southern

Sta. 1553+82

Layer ID	USCS Soil Classification	Φ	C
1	Levee Embankment - SP-SM	30	0
2	Foundation - ML	30	0
3	Blanket - SM	30	0
4	Liquefiable SP - SM	0	365
5	ML	28	0
6	SM	28	0
7	SP - SM	30	0
8	CL	25	100
4	Liquefiable SP - SM	6.9	0

Sta. 1595+33

Layer ID	USCS Soil Classification	Φ	C
1	Levee Embankment - CL	24	100
2	Levee Embankment - SP	33	0
3	Levee Embankment - ML	30	0
4	Blanket - CL	25	100
5	Liquefiable SP - SM	0	133
6	SC	28	0
7	SW-SC	30	0
8	SC	28	0
5	Liquefiable SP - SM	3.9	0



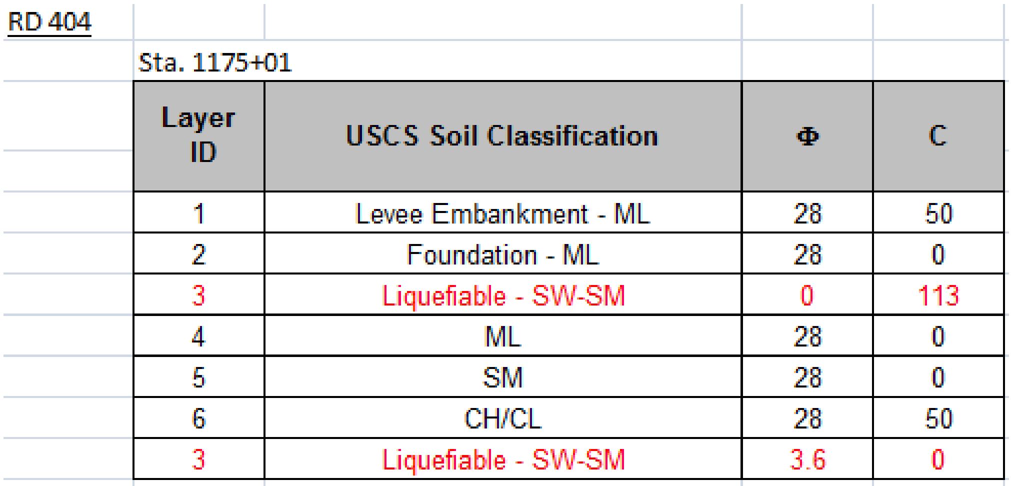
RD 17 - Northern

Sta. 1151+06

Layer ID	USCS Soil Classification	Φ	C
1	Levee Embankment - CL	24	100
2	Levee Embankment - SC - SM	28	0
3	Foundation - CL	25	100
4	Liquefiable - SP - SC	0	201
5	CL	25	100
6	ML	28	0
7	CL	25	100
8	SM	28	0
9	CL	25	100
4	Liquefiable - SP - SC	5.2	0

Sta. 1191+43

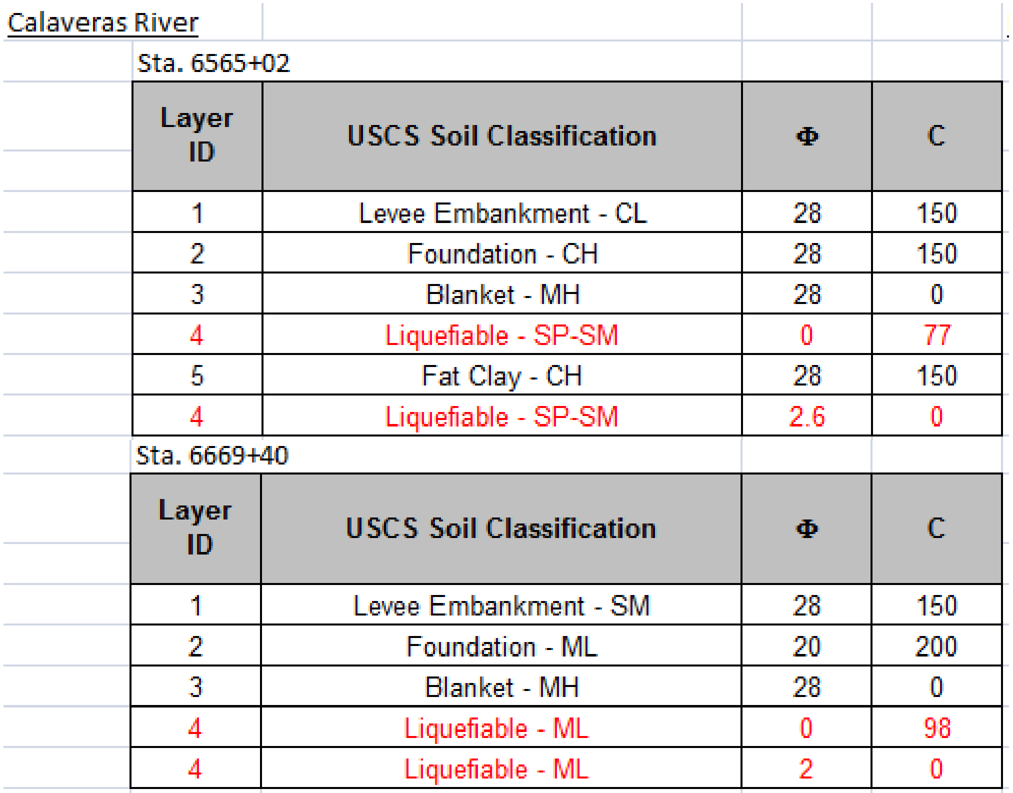
Layer ID	USCS Soil Classification	Φ	C
1	Embankment - CL	29	200
2	Blanket - CH	25	100
3	Foundation - CL	25	100
4	Liqufiable - SM	0	164
5	CL	25	100
6	Liqufiable - SC	0	111
7	SC	28	0
8	CH	25	100
9	SM	28	0
4	Liqufiable - SM	4.3	0
6	Liqufiable - SC	2.7	0



RD 404

Sta. 1175+01

Layer ID	USCS Soil Classification	Φ	C
1	Levee Embankment - ML	28	50
2	Foundation - ML	28	0
3	Liquefiable - SW-SM	0	113
4	ML	28	0
5	SM	28	0
6	CH/CL	28	50
3	Liquefiable - SW-SM	3.6	0



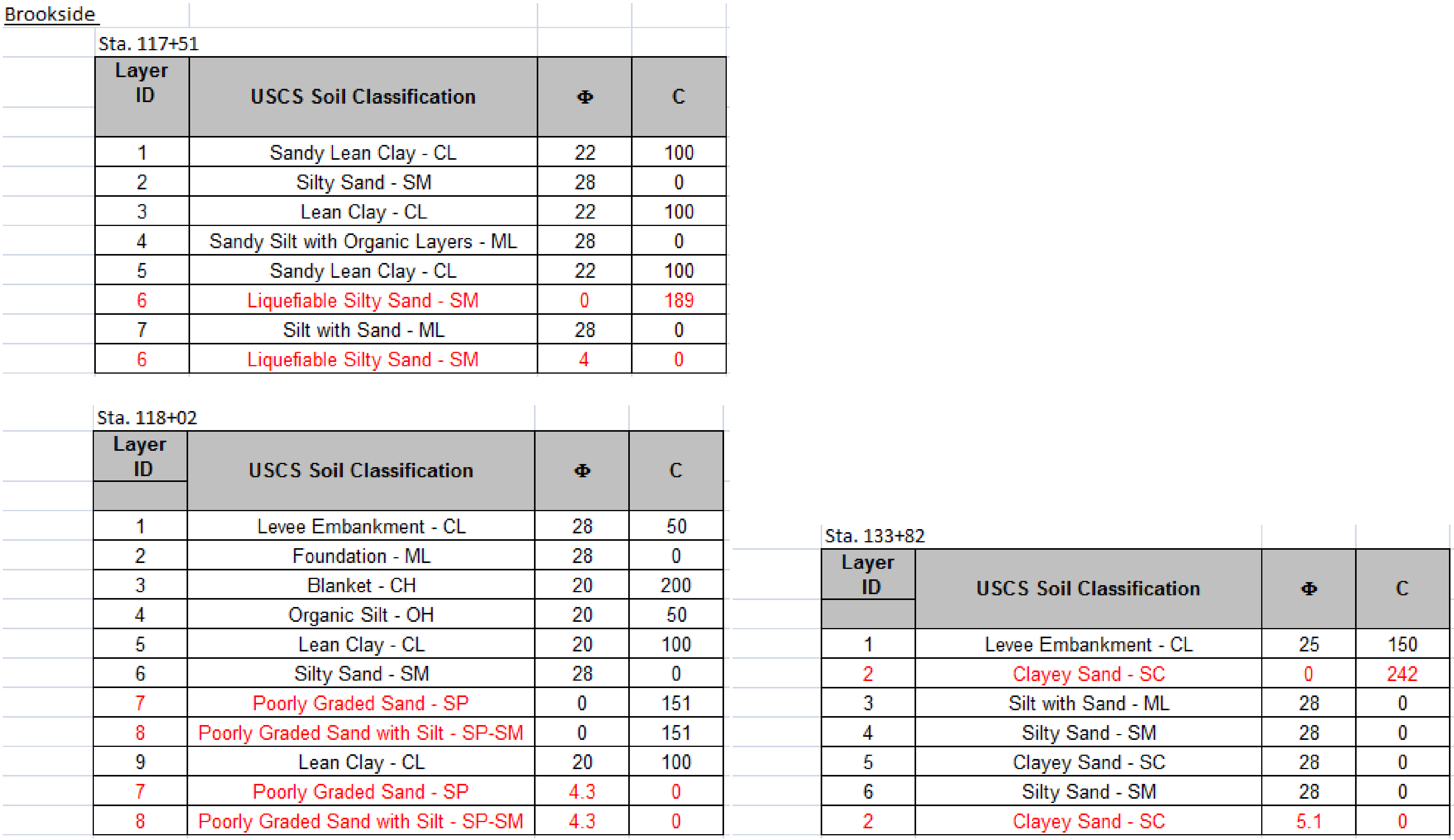
Calaveras River

Sta. 6565+02

Layer ID	USCS Soil Classification	Φ	C
1	Levee Embankment - CL	28	150
2	Foundation - CH	28	150
3	Blanket - MH	28	0
4	Liquefiable - SP-SM	0	77
5	Fat Clay - CH	28	150
4	Liquefiable - SP-SM	2.6	0

Sta. 6669+40

Layer ID	USCS Soil Classification	Φ	C
1	Levee Embankment - SM	28	150
2	Foundation - ML	20	200
3	Blanket - MH	28	0
4	Liquefiable - ML	0	98
4	Liquefiable - ML	2	0



Brookside

Sta. 117+51

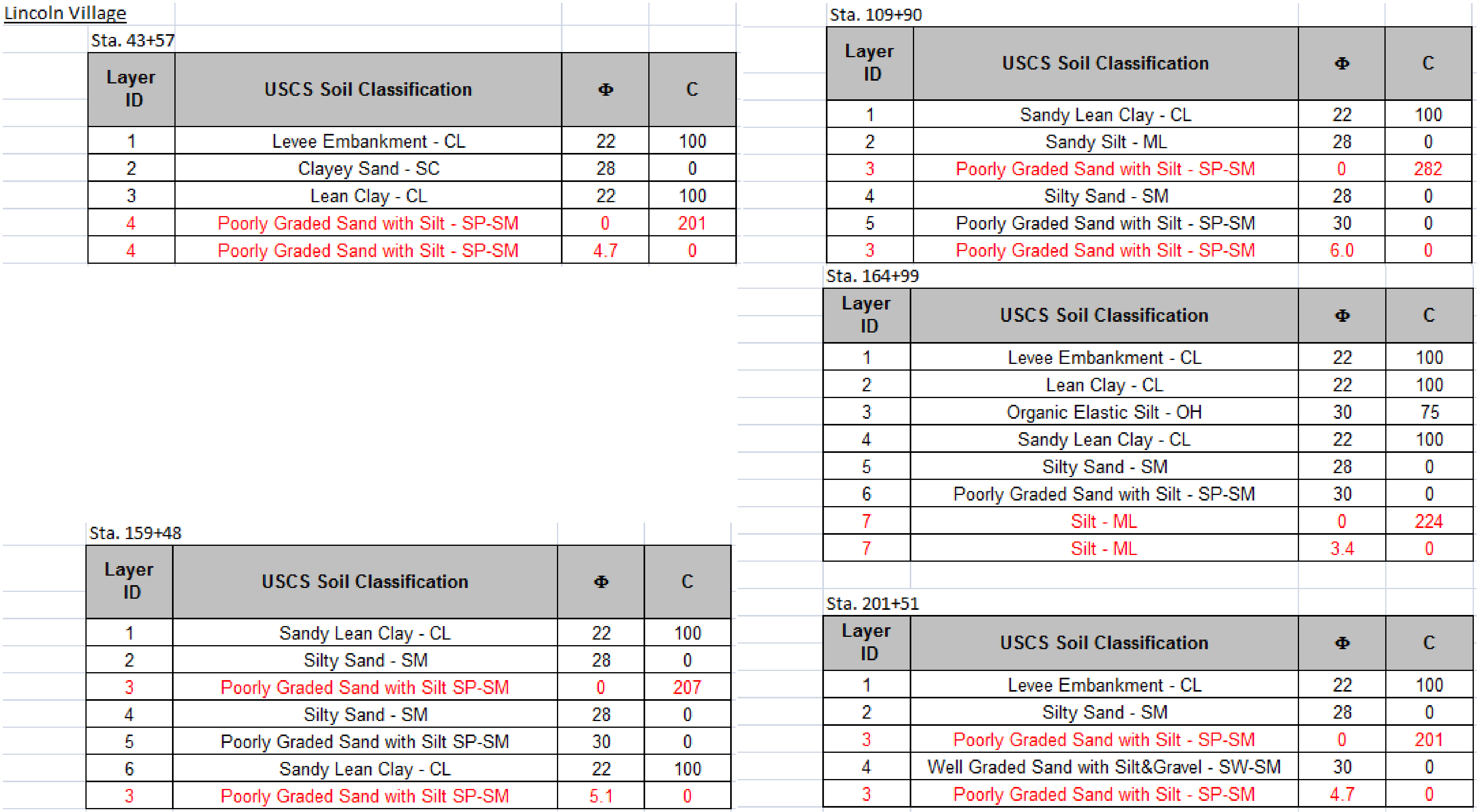
Layer ID	USCS Soil Classification	Φ	C
1	Sandy Lean Clay - CL	22	100
2	Silty Sand - SM	28	0
3	Lean Clay - CL	22	100
4	Sandy Silt with Organic Layers - ML	28	0
5	Sandy Lean Clay - CL	22	100
6	Liquefiable Silty Sand - SM	0	189
7	Silt with Sand - ML	28	0
6	Liquefiable Silty Sand - SM	4	0

Sta. 118+02

Layer ID	USCS Soil Classification	Φ	C
1	Levee Embankment - CL	28	50
2	Foundation - ML	28	0
3	Blanket - CH	20	200
4	Organic Silt - OH	20	50
5	Lean Clay - CL	20	100
6	Silty Sand - SM	28	0
7	Poorly Graded Sand - SP	0	151
8	Poorly Graded Sand with Silt - SP-SM	0	151
9	Lean Clay - CL	20	100
7	Poorly Graded Sand - SP	4.3	0
8	Poorly Graded Sand with Silt - SP-SM	4.3	0

Sta. 133+82

Layer ID	USCS Soil Classification	Φ	C
1	Levee Embankment - CL	25	150
2	Clayey Sand - SC	0	242
3	Silt with Sand - ML	28	0
4	Silty Sand - SM	28	0
5	Clayey Sand - SC	28	0
6	Silty Sand - SM	28	0
2	Clayey Sand - SC	5.1	0



Lincoln Village

Sta. 43+57

Layer ID	USCS Soil Classification	Φ	C
1	Levee Embankment - CL	22	100
2	Clayey Sand - SC	28	0
3	Lean Clay - CL	22	100
4	Poorly Graded Sand with Silt - SP-SM	0	201
4	Poorly Graded Sand with Silt - SP-SM	4.7	0

Sta. 159+48

Layer ID	USCS Soil Classification	Φ	C
1	Sandy Lean Clay - CL	22	100
2	Silty Sand - SM	28	0
3	Poorly Graded Sand with Silt SP-SM	0	207
4	Silty Sand - SM	28	0
5	Poorly Graded Sand with Silt SP-SM	30	0
6	Sandy Lean Clay - CL	22	100
3	Poorly Graded Sand with Silt SP-SM	5.1	0

Sta. 109+90

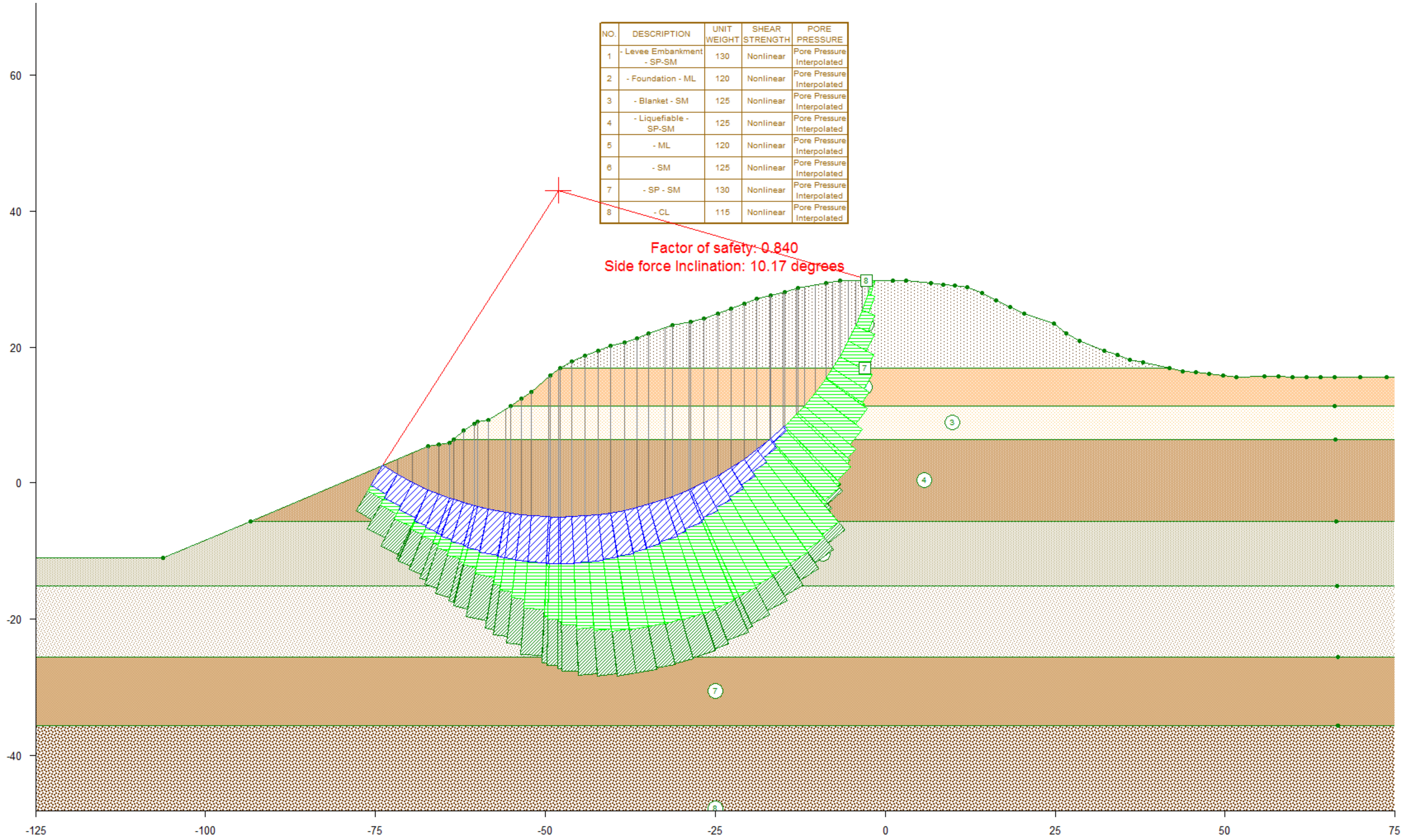
Layer ID	USCS Soil Classification	Φ	C
1	Sandy Lean Clay - CL	22	100
2	Sandy Silt - ML	28	0
3	Poorly Graded Sand with Silt - SP-SM	0	282
4	Silty Sand - SM	28	0
5	Poorly Graded Sand with Silt - SP-SM	30	0
3	Poorly Graded Sand with Silt - SP-SM	6.0	0

Sta. 164+99

Layer ID	USCS Soil Classification	Φ	C
1	Levee Embankment - CL	22	100
2	Lean Clay - CL	22	100
3	Organic Elastic Silt - OH	30	75
4	Sandy Lean Clay - CL	22	100
5	Silty Sand - SM	28	0
6	Poorly Graded Sand with Silt - SP-SM	30	0
7	Silt - ML	0	224
7	Silt - ML	3.4	0

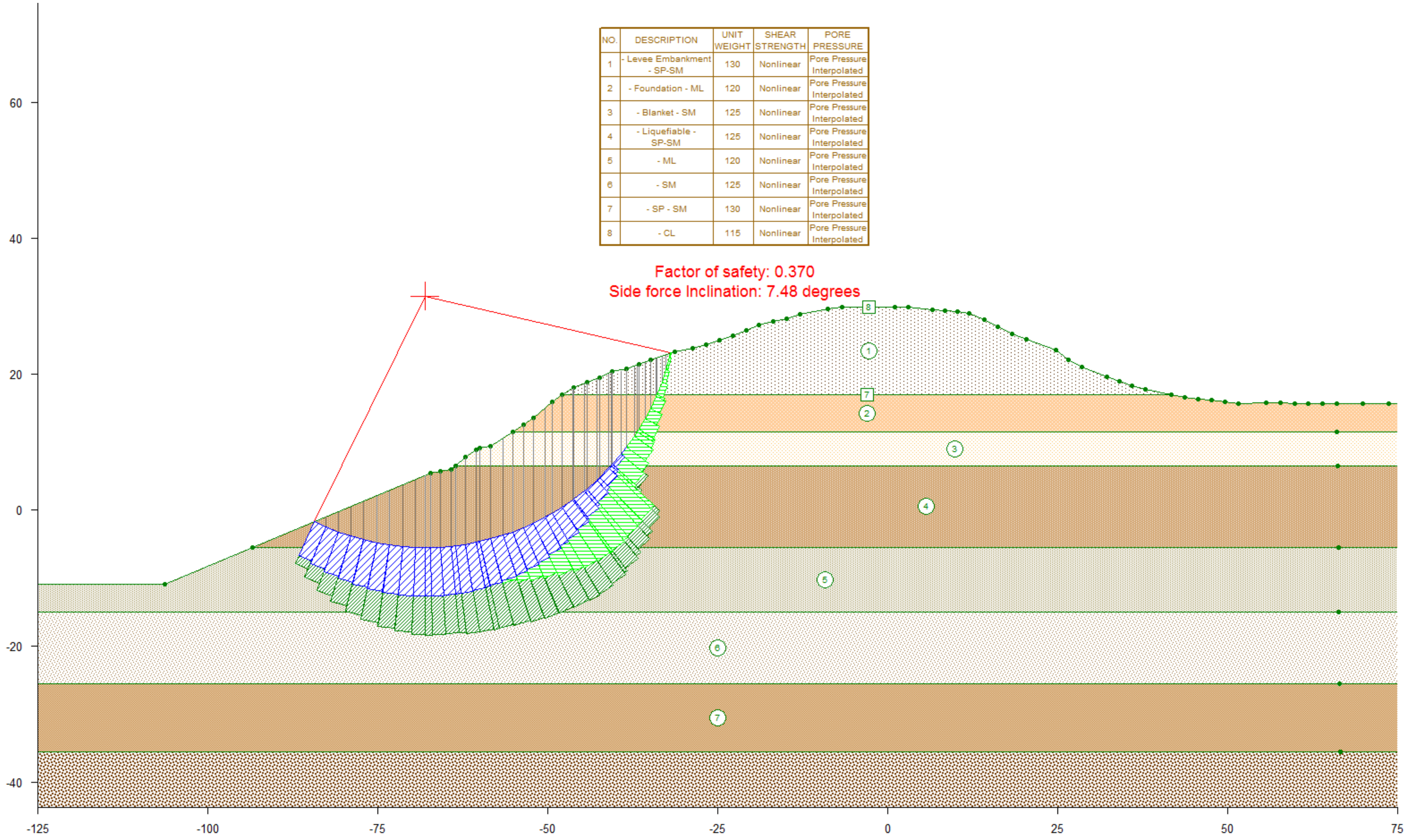
Sta. 201+51

Layer ID	USCS Soil Classification	Φ	C
1	Levee Embankment - CL	22	100
2	Silty Sand - SM	28	0
3	Poorly Graded Sand with Silt - SP-SM	0	201
4	Well Graded Sand with Silt&Gravel - SW-SM	30	0
3	Poorly Graded Sand with Silt - SP-SM	4.7	0



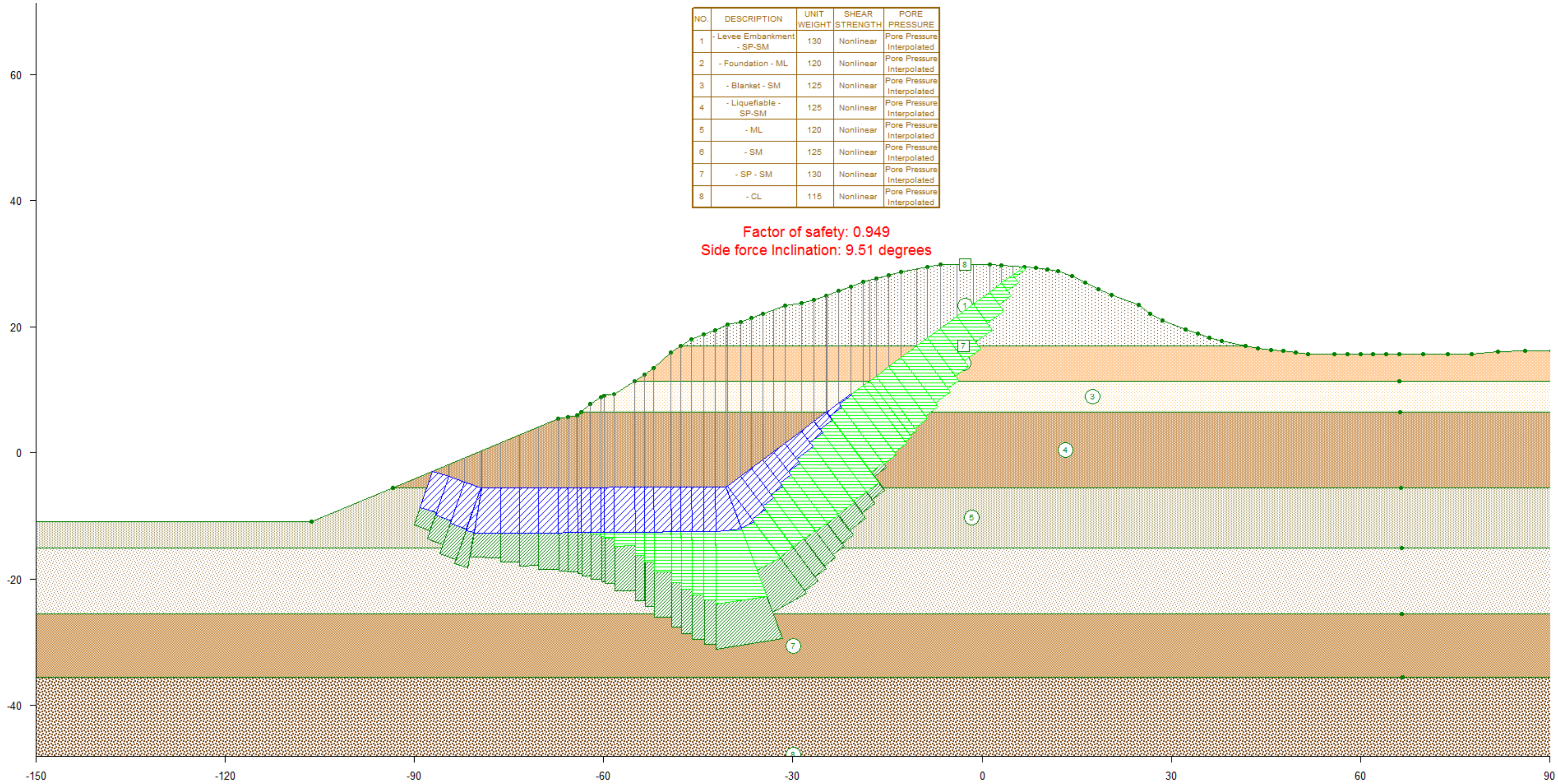
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - SP-SM	130	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Blanket - SM	125	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP-SM	125	Nonlinear	Pore Pressure Interpolated
5	- ML	120	Nonlinear	Pore Pressure Interpolated
6	- SM	125	Nonlinear	Pore Pressure Interpolated
7	- SP - SM	130	Nonlinear	Pore Pressure Interpolated
8	- CL	115	Nonlinear	Pore Pressure Interpolated

Fig F-1(a). RD 17 Southern, Station 1553+82 – Waterside – Option 1: Circular (Sr = 365 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - SP-SM	130	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Blanket - SM	125	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP-SM	125	Nonlinear	Pore Pressure Interpolated
5	- ML	120	Nonlinear	Pore Pressure Interpolated
6	- SM	125	Nonlinear	Pore Pressure Interpolated
7	- SP - SM	130	Nonlinear	Pore Pressure Interpolated
8	- CL	115	Nonlinear	Pore Pressure Interpolated

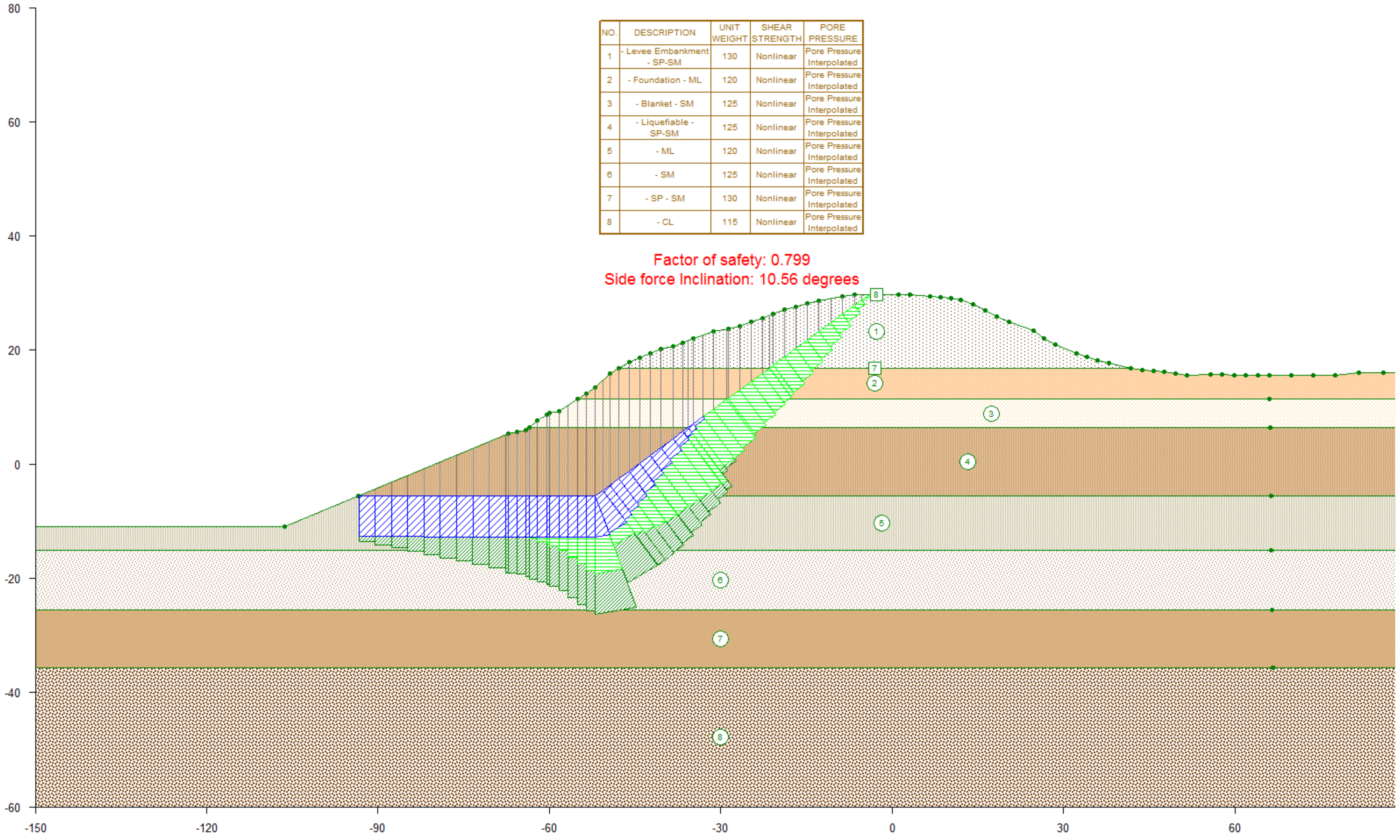
Fig F-1(b). RD 17 Station 1553+82 – Waterside – Option 1: Circular (PHI = 6.9 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - SP-SM	130	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Blanket - SM	125	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP-SM	125	Nonlinear	Pore Pressure Interpolated
5	- ML	120	Nonlinear	Pore Pressure Interpolated
6	- SM	125	Nonlinear	Pore Pressure Interpolated
7	- SP - SM	130	Nonlinear	Pore Pressure Interpolated
8	- CL	115	Nonlinear	Pore Pressure Interpolated

Factor of safety: 0.949
Side force Inclination: 9.51 degrees

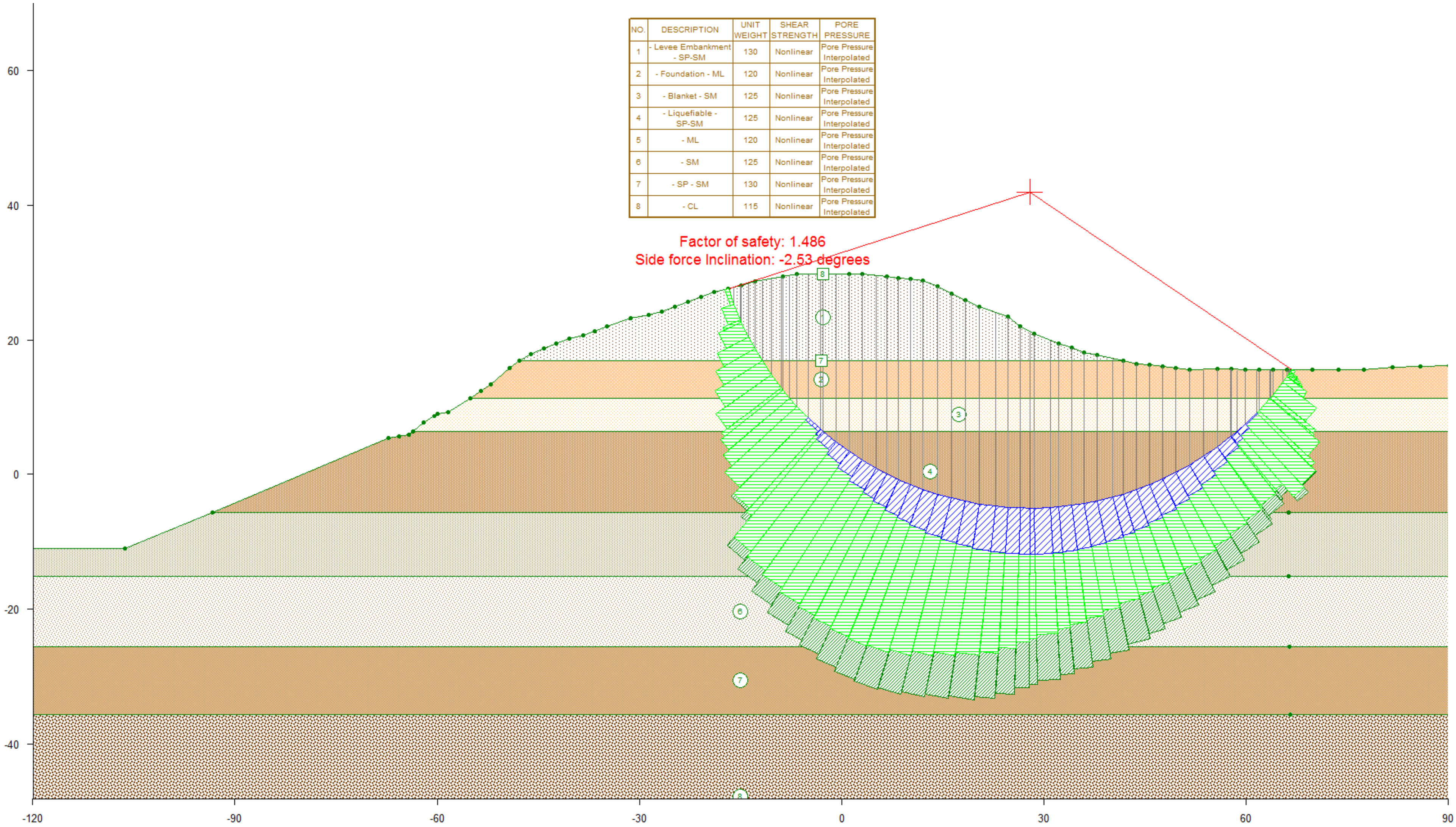
Fig F-2(a). RD 17 Southern, Station 1553+82 – Waterside – Option 2: Wedges (Sr = 365 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - SP-SM	130	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Blanket - SM	125	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP-SM	125	Nonlinear	Pore Pressure Interpolated
5	- ML	120	Nonlinear	Pore Pressure Interpolated
6	- SM	125	Nonlinear	Pore Pressure Interpolated
7	- SP - SM	130	Nonlinear	Pore Pressure Interpolated
8	- CL	115	Nonlinear	Pore Pressure Interpolated

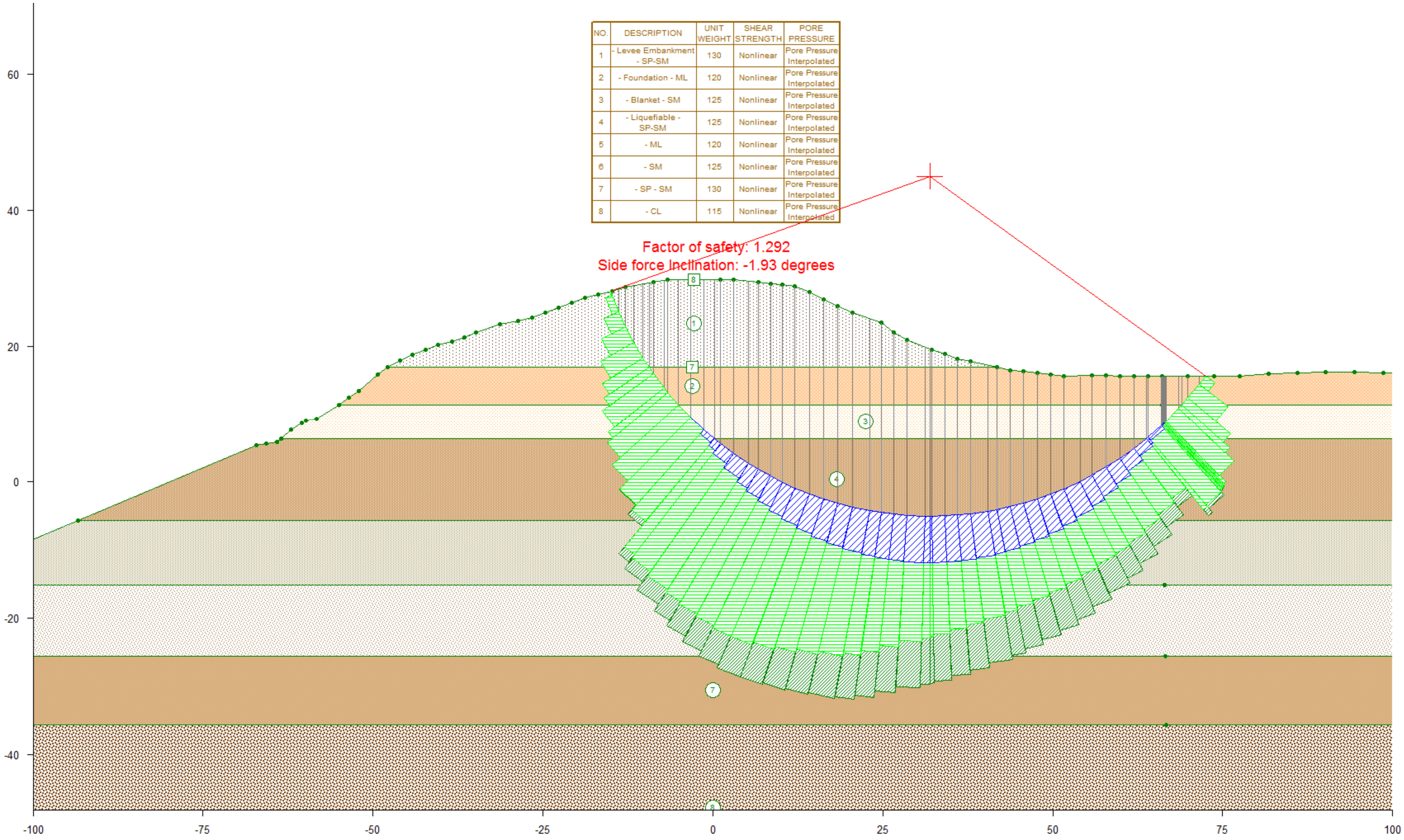
Factor of safety: 0.799
Side force Inclination: 10.56 degrees

Fig F-2(b). RD 17 Station 1553+82 – Waterside – Option 2: Wedges (PHI = 6.9 in liquefiable material)



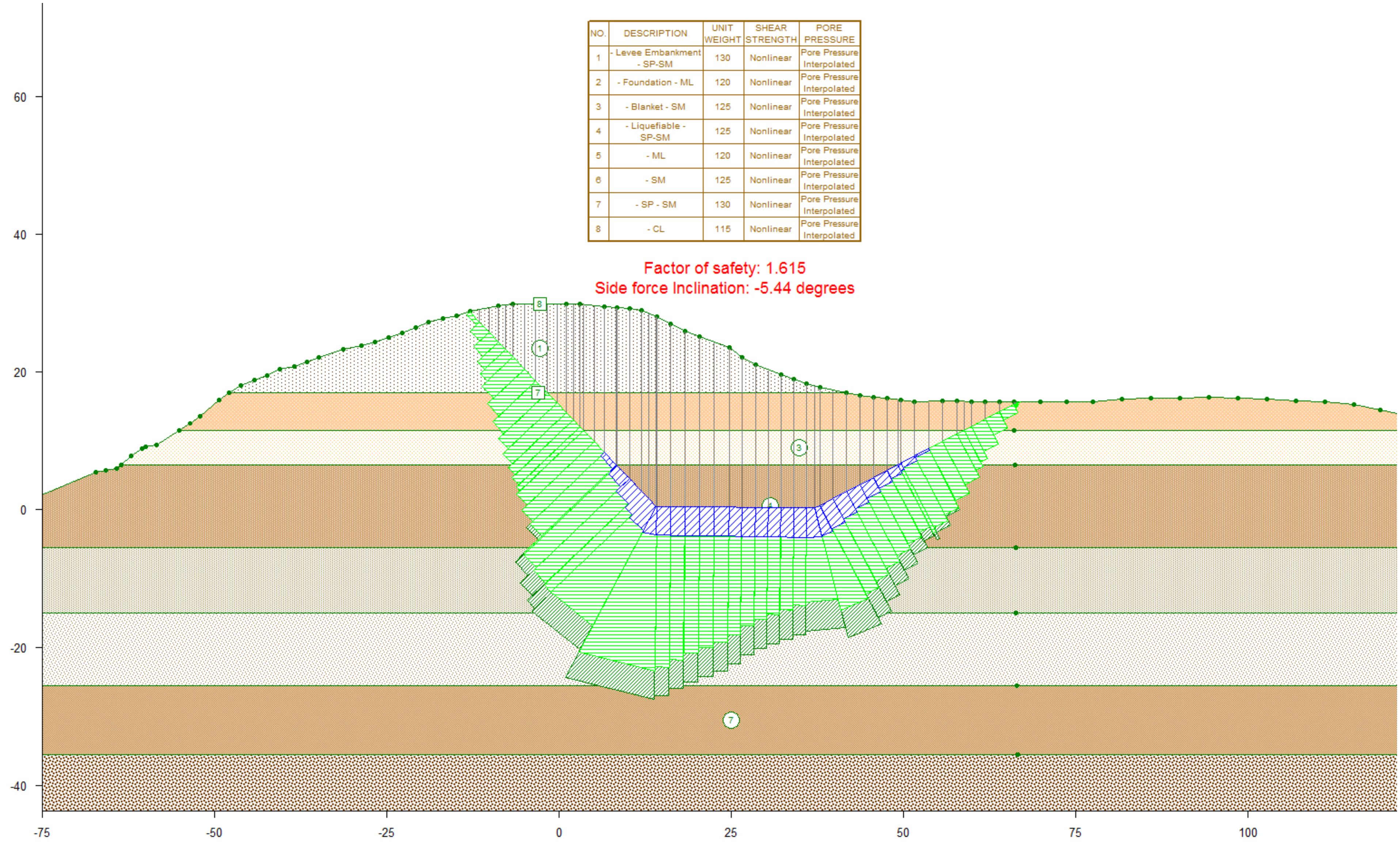
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - SP-SM	130	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Blanket - SM	125	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP-SM	125	Nonlinear	Pore Pressure Interpolated
5	- ML	120	Nonlinear	Pore Pressure Interpolated
6	- SM	125	Nonlinear	Pore Pressure Interpolated
7	- SP - SM	130	Nonlinear	Pore Pressure Interpolated
8	- CL	115	Nonlinear	Pore Pressure Interpolated

Fig F-3(a). RD 17 Southern, Station 1553+82 – Landside – Option 3: Circular (Sr = 365 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - SP-SM	130	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Blanket - SM	125	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP-SM	125	Nonlinear	Pore Pressure Interpolated
5	- ML	120	Nonlinear	Pore Pressure Interpolated
6	- SM	125	Nonlinear	Pore Pressure Interpolated
7	- SP - SM	130	Nonlinear	Pore Pressure Interpolated
8	- CL	115	Nonlinear	Pore Pressure Interpolated

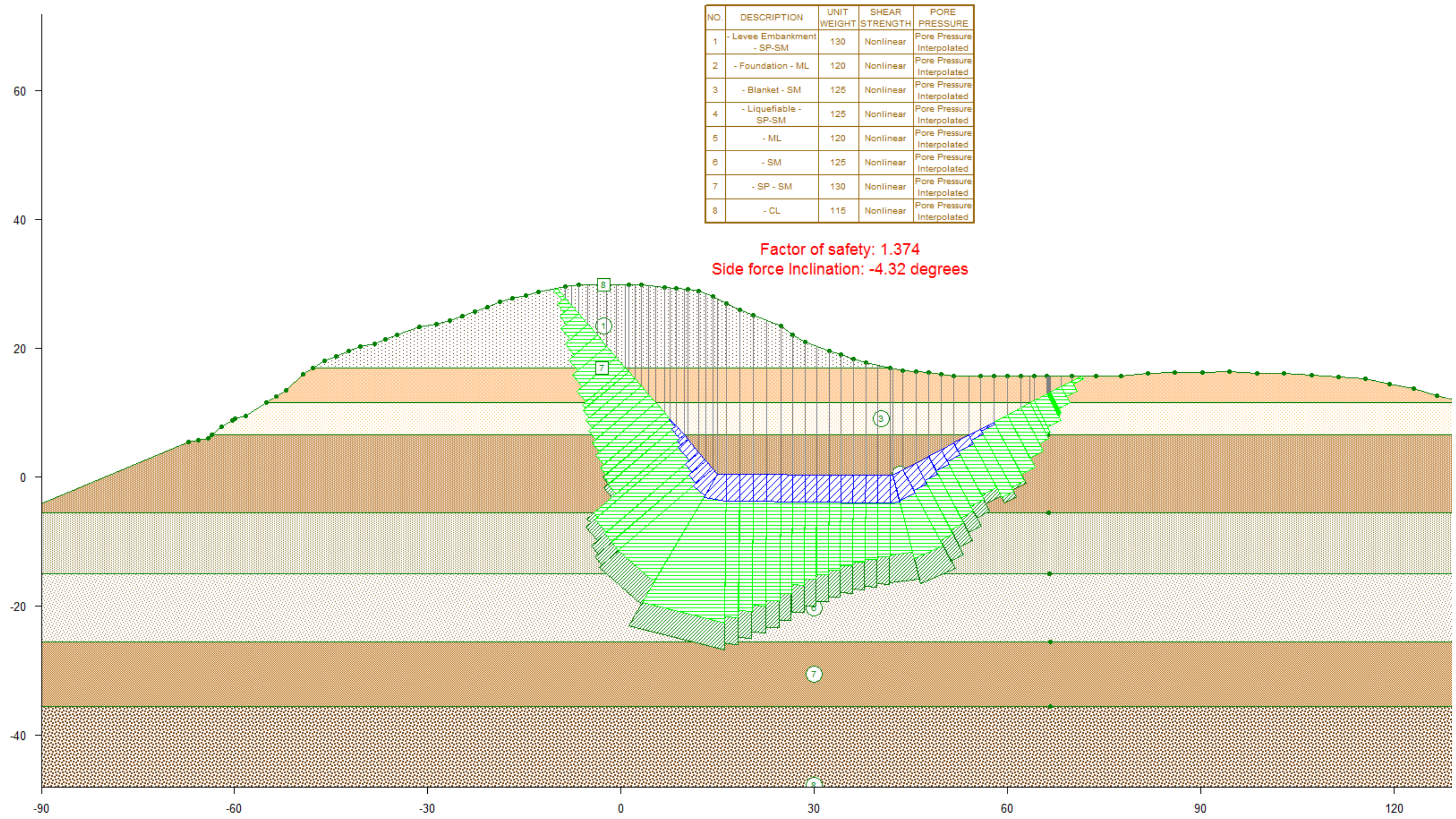
Fig F-3(b). RD 17 Station 1553+82 – Landside – Option 3: Circular (PHI = 6.9 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - SP-SM	130	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Blanket - SM	125	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP-SM	125	Nonlinear	Pore Pressure Interpolated
5	- ML	120	Nonlinear	Pore Pressure Interpolated
6	- SM	125	Nonlinear	Pore Pressure Interpolated
7	- SP - SM	130	Nonlinear	Pore Pressure Interpolated
8	- CL	115	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.615
Side force Inclination: -5.44 degrees

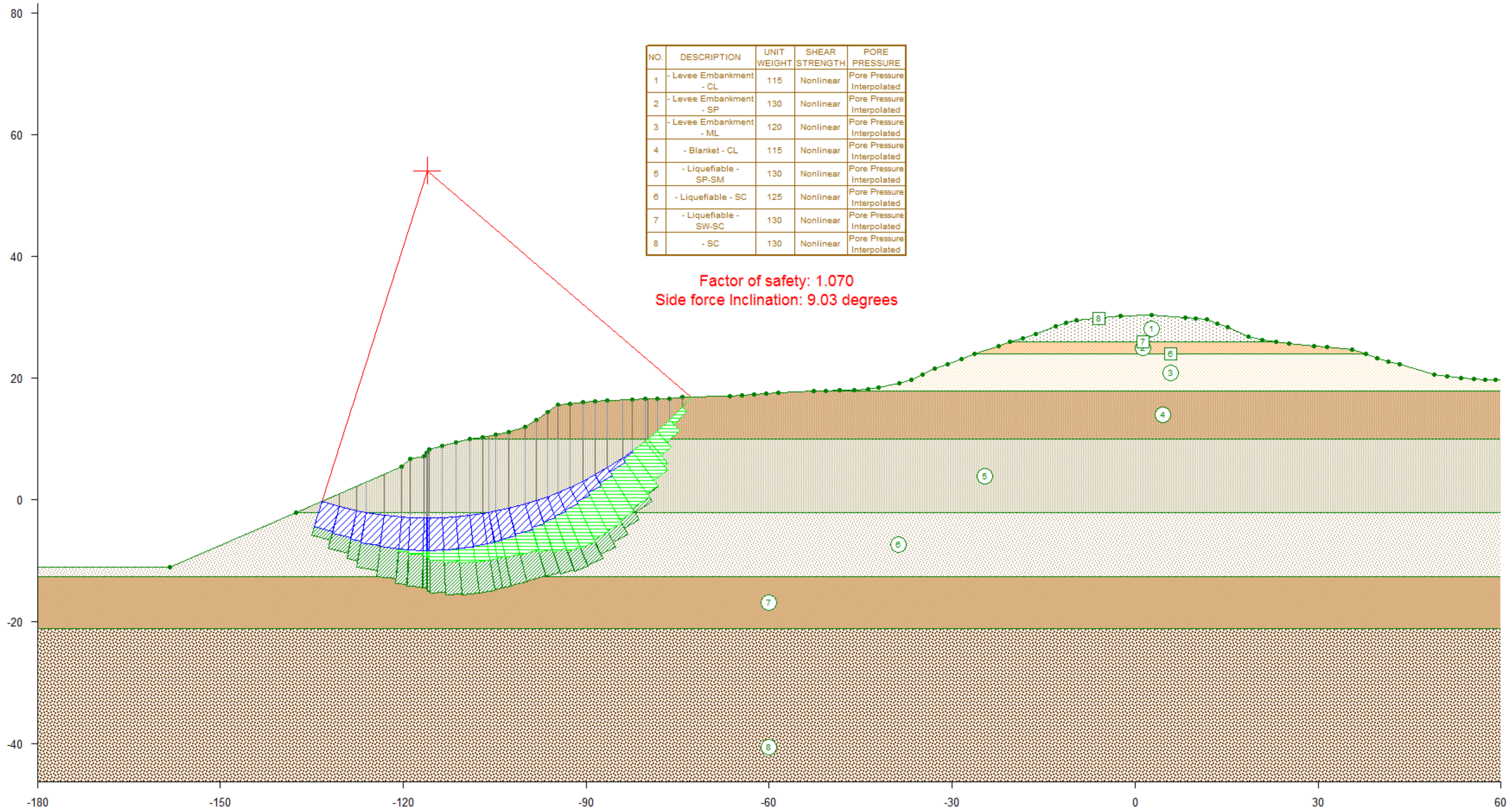
Fig F-4(a). RD 17 Southern, Station 1553+82 – Landside – Option 4: Wedge (Sr = 365 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - SP-SM	130	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Blanket - SM	125	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP-SM	125	Nonlinear	Pore Pressure Interpolated
5	- ML	120	Nonlinear	Pore Pressure Interpolated
6	- SM	125	Nonlinear	Pore Pressure Interpolated
7	- SP - SM	130	Nonlinear	Pore Pressure Interpolated
8	- CL	115	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.374
Side force Inclination: -4.32 degrees

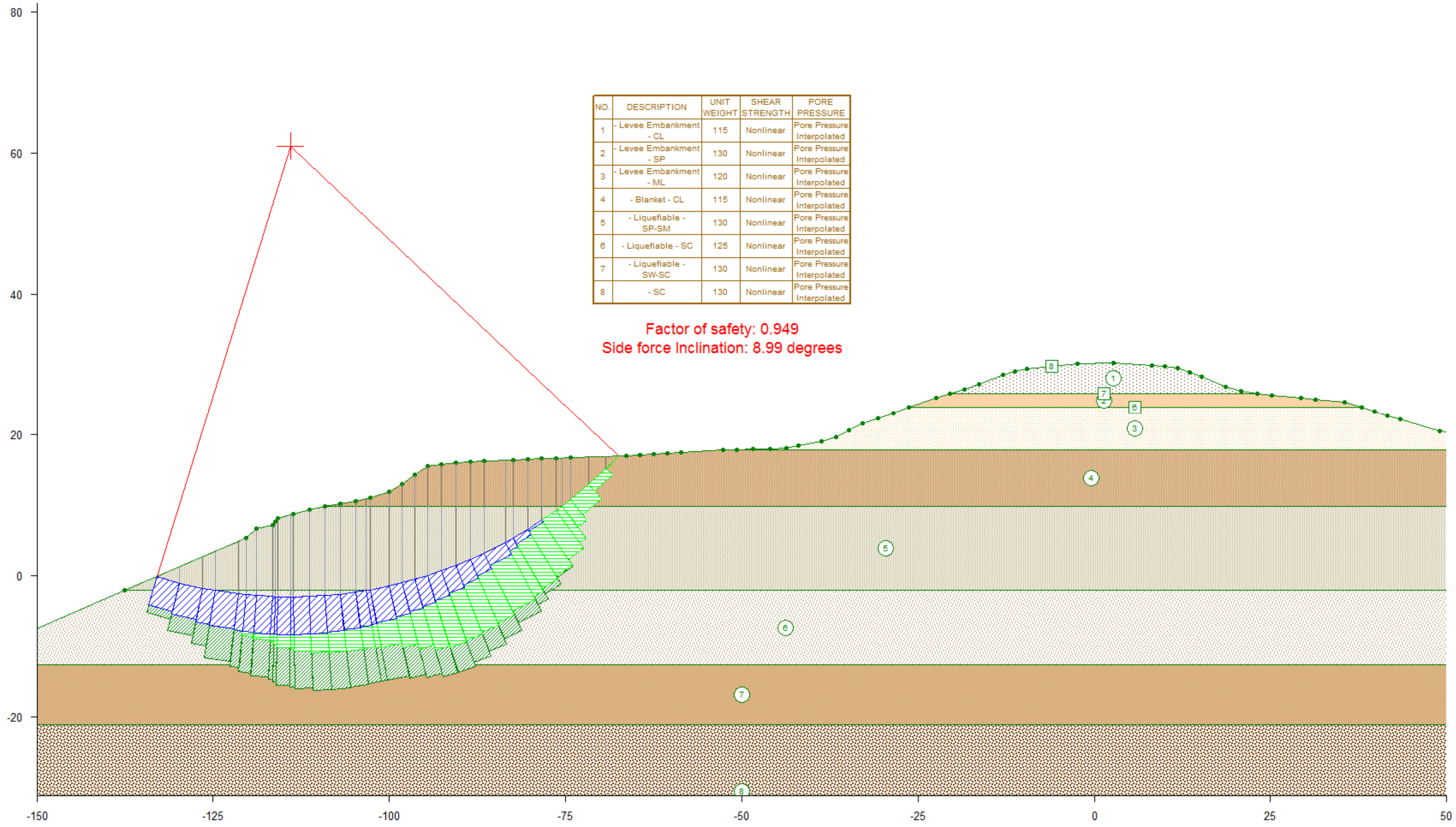
Fig F-4(b). RD 17 Station 1553+82 – Landside – Option 4: Wedge (PHI = 6.9 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SP	130	Nonlinear	Pore Pressure Interpolated
3	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
4	- Blanket - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Liquefiable - SP-SM	130	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable - SW-SC	130	Nonlinear	Pore Pressure Interpolated
8	- SC	130	Nonlinear	Pore Pressure Interpolated

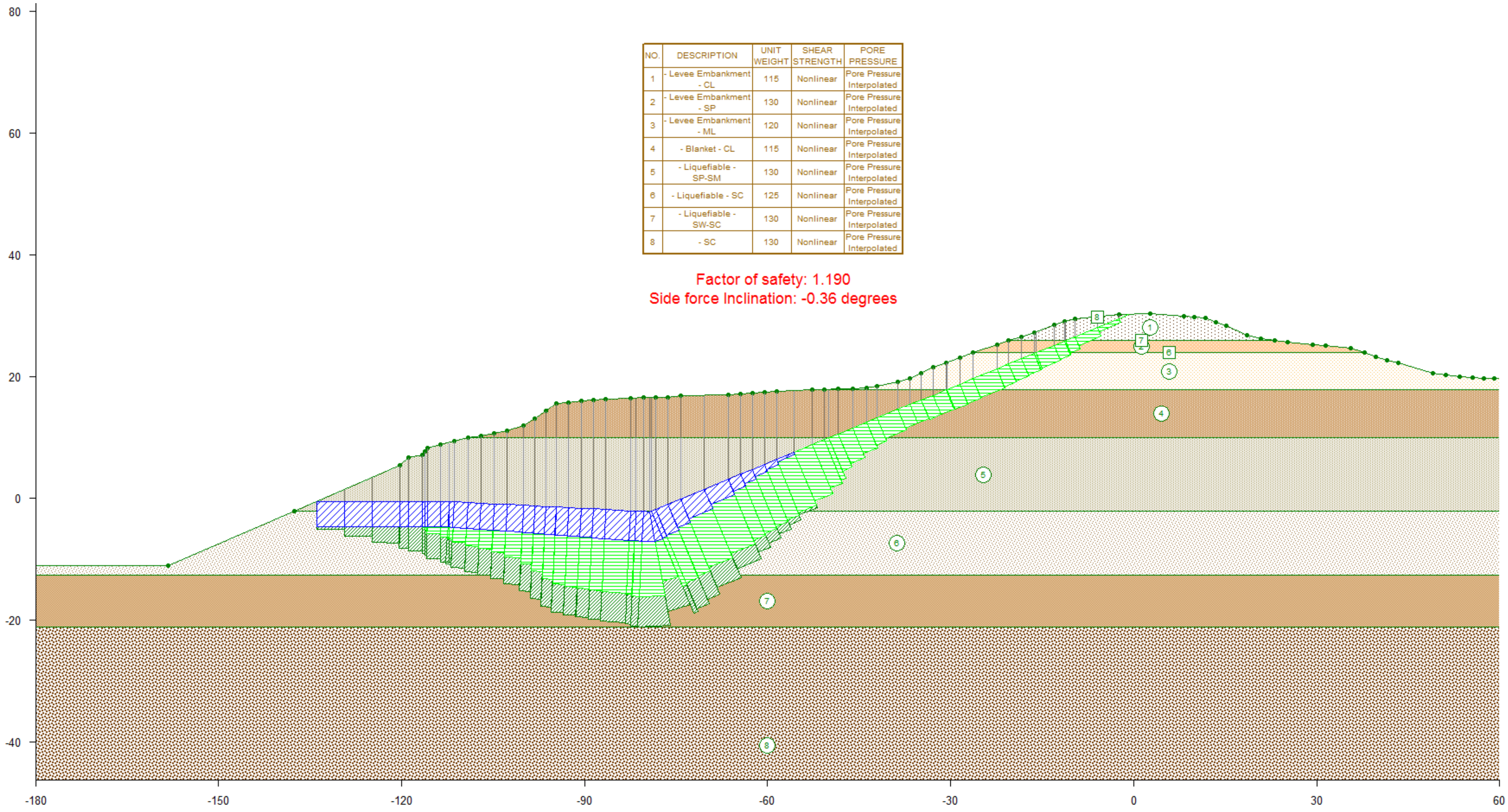
Factor of safety: 1.070
Side force Inclination: 9.03 degrees

Fig F-5(a). RD 17 Southern, Station 1595+33 – Waterside – Option 1: Circular (Sr = 133 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SP	130	Nonlinear	Pore Pressure Interpolated
3	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
4	- Blanket - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Liquefiable - SP-SM	130	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable - SW-SC	130	Nonlinear	Pore Pressure Interpolated
8	- SC	130	Nonlinear	Pore Pressure Interpolated

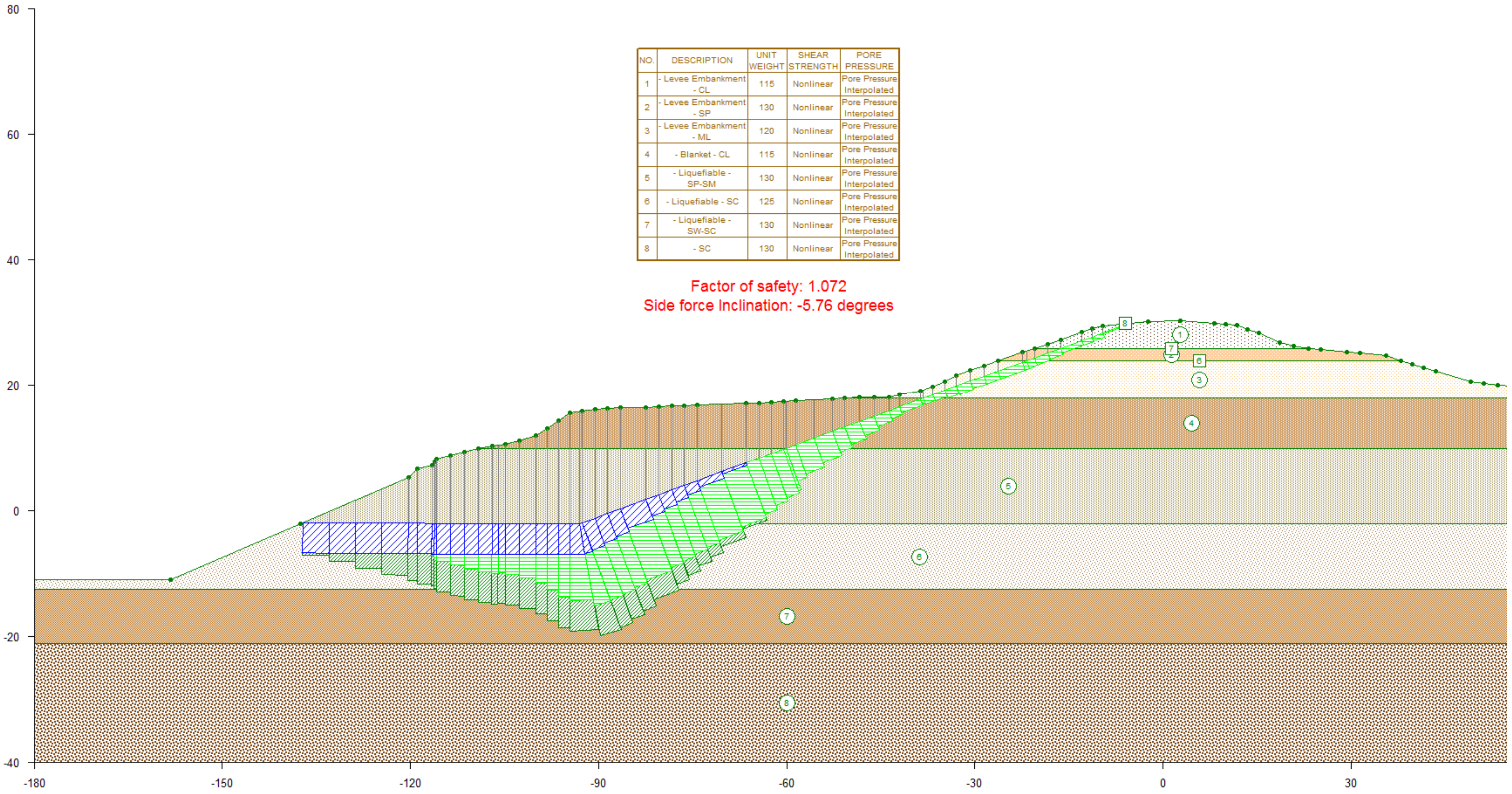
Fig F-5(b). RD 17 Station 1595+33 – Waterside – Option 1: Circular (PHI = 3.9 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SP	130	Nonlinear	Pore Pressure Interpolated
3	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
4	- Blanket - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Liquefiable - SP-SM	130	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable - SW-SC	130	Nonlinear	Pore Pressure Interpolated
8	- SC	130	Nonlinear	Pore Pressure Interpolated

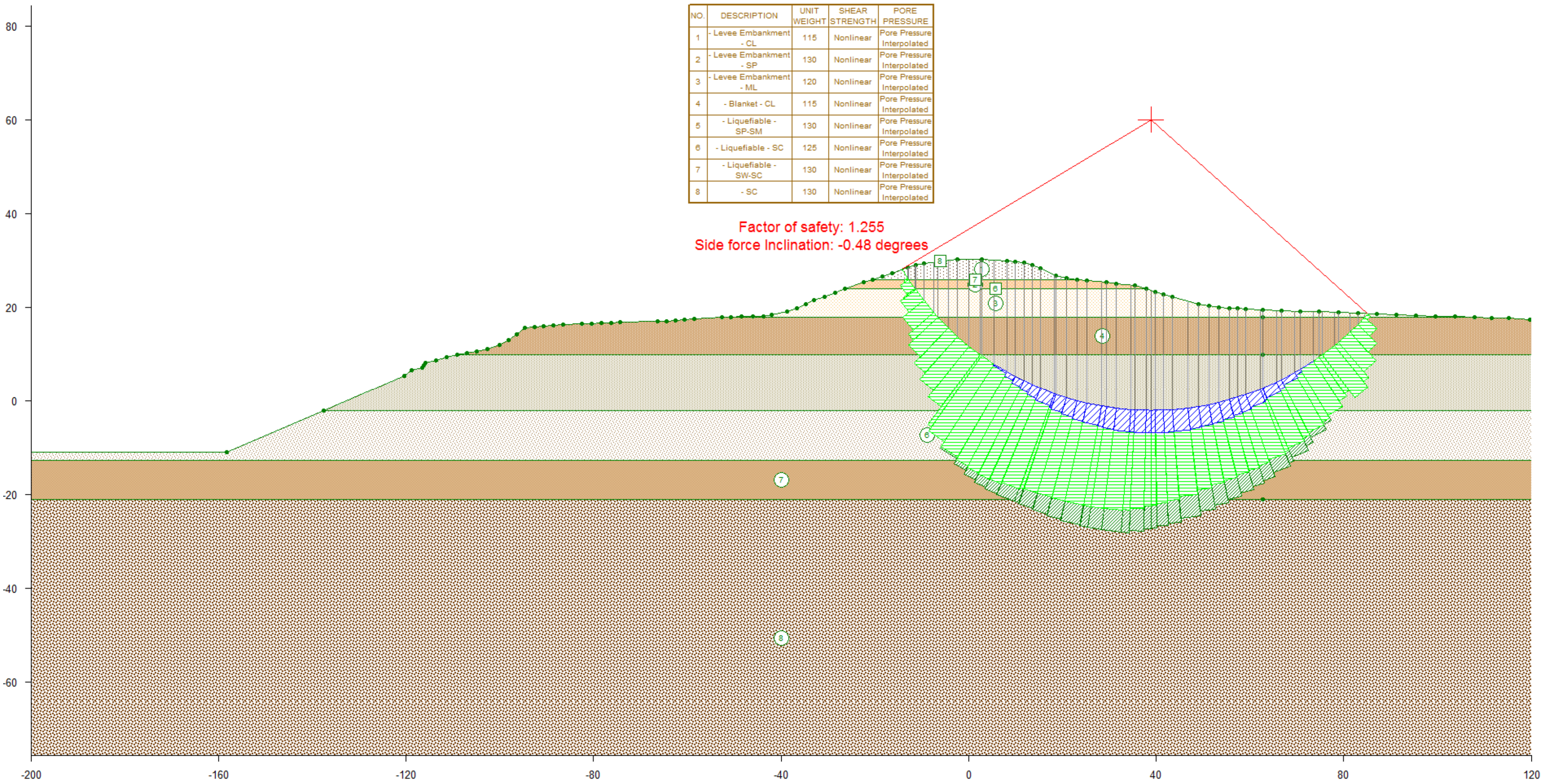
Factor of safety: 1.190
Side force Inclination: -0.36 degrees

Fig F-6(a). RD 17 Southern, Station 1595+33 – Waterside – Option 2: Wedges (Sr = 133 psf in liquefiable material)



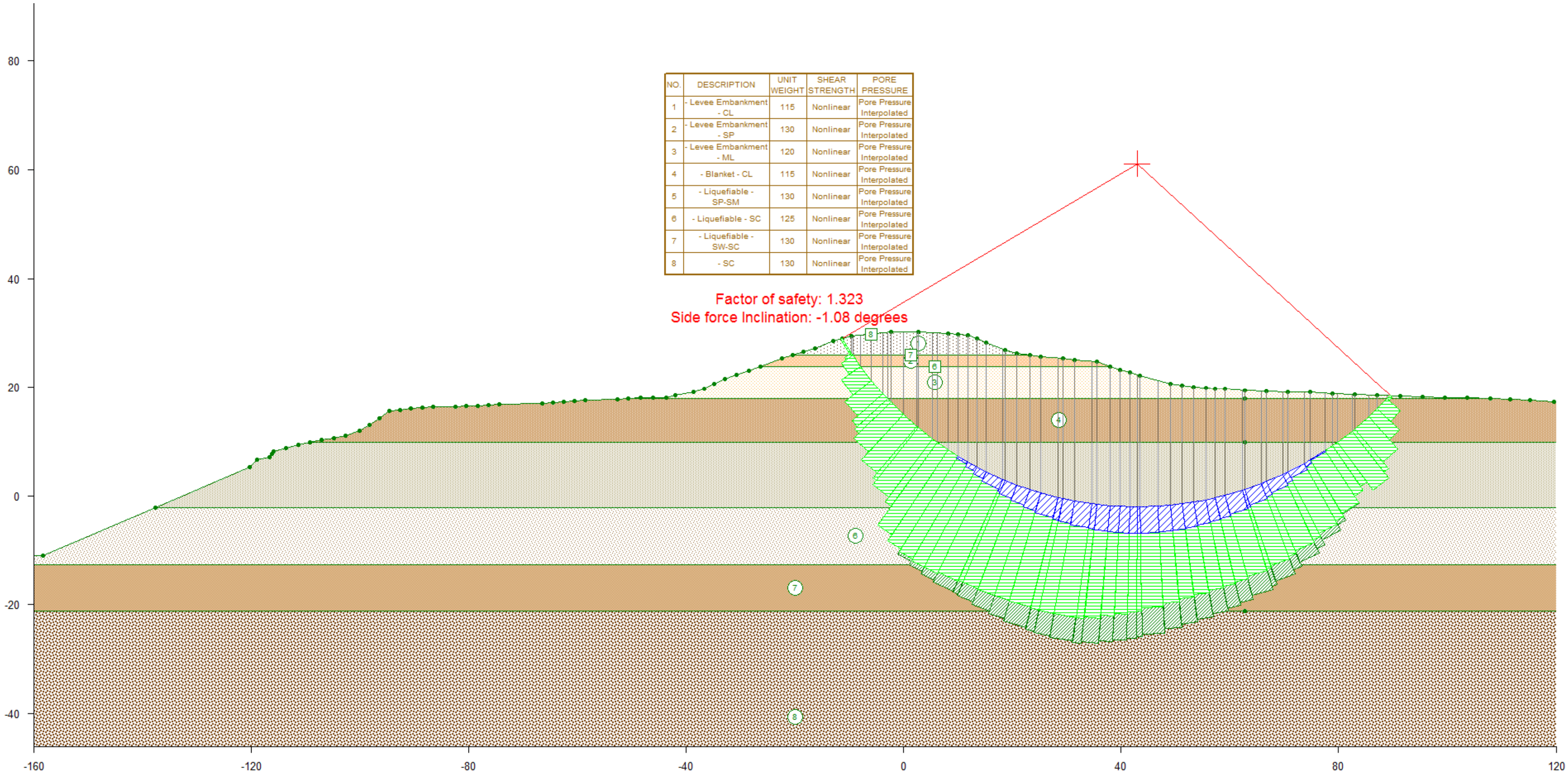
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SP	130	Nonlinear	Pore Pressure Interpolated
3	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
4	- Blanket - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Liquefiable - SP-SM	130	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable - SW-SC	130	Nonlinear	Pore Pressure Interpolated
8	- SC	130	Nonlinear	Pore Pressure Interpolated

Fig F-6(b). RD 17 Station 1595+33 – Waterside – Option 2: Wedges (PHI = 3.9 in liquefiable material)



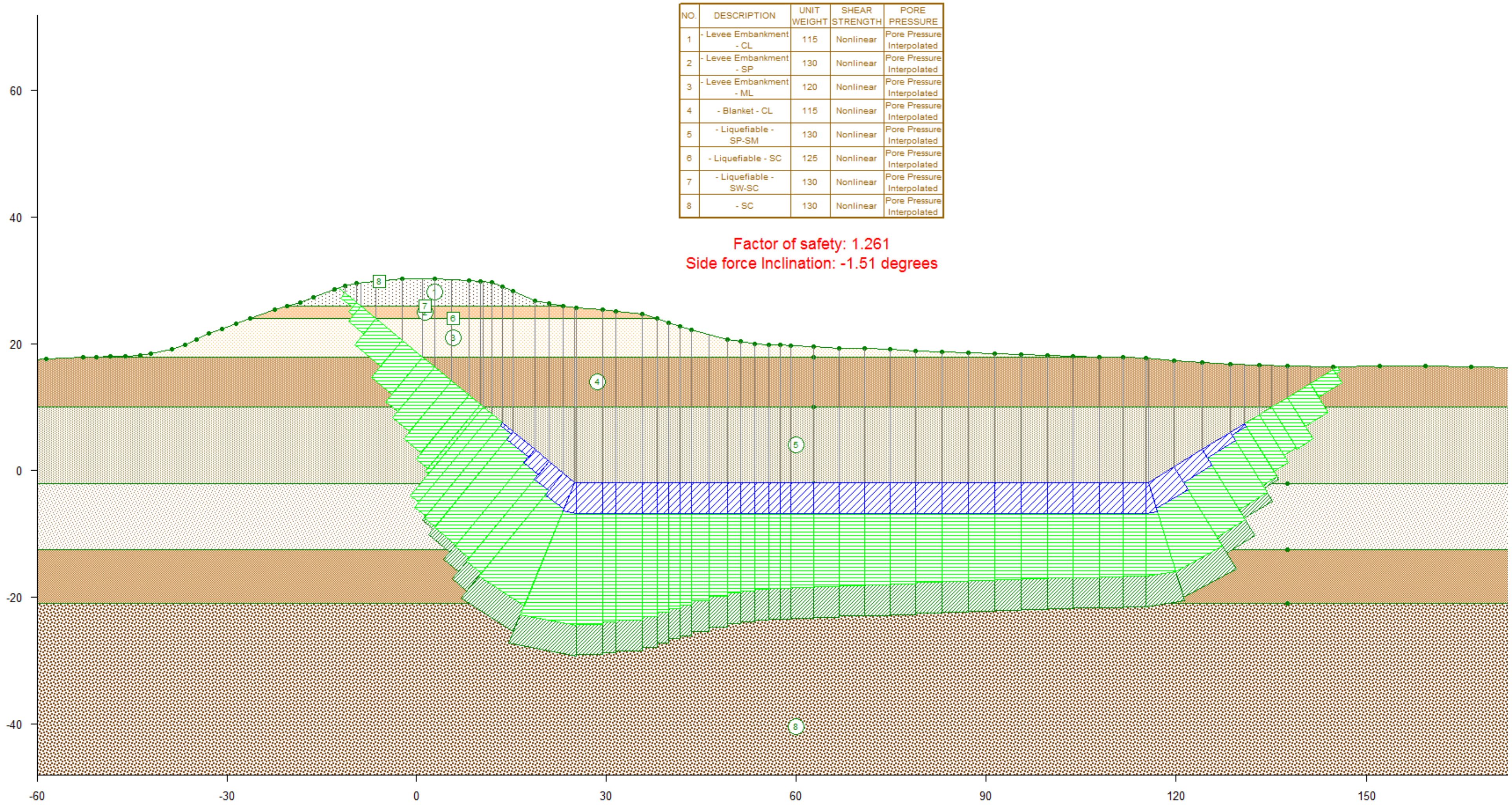
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SP	130	Nonlinear	Pore Pressure Interpolated
3	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
4	- Blanket - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Liquefiable - SP-SM	130	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable - SW-SC	130	Nonlinear	Pore Pressure Interpolated
8	- SC	130	Nonlinear	Pore Pressure Interpolated

Fig F-7(a). RD 17 Southern, Station 1595+33 – Landside – Option 3: Circular (Sr = 133 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SP	130	Nonlinear	Pore Pressure Interpolated
3	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
4	- Blanket - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Liquefiable - SP-SM	130	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable - SW-SC	130	Nonlinear	Pore Pressure Interpolated
8	- SC	130	Nonlinear	Pore Pressure Interpolated

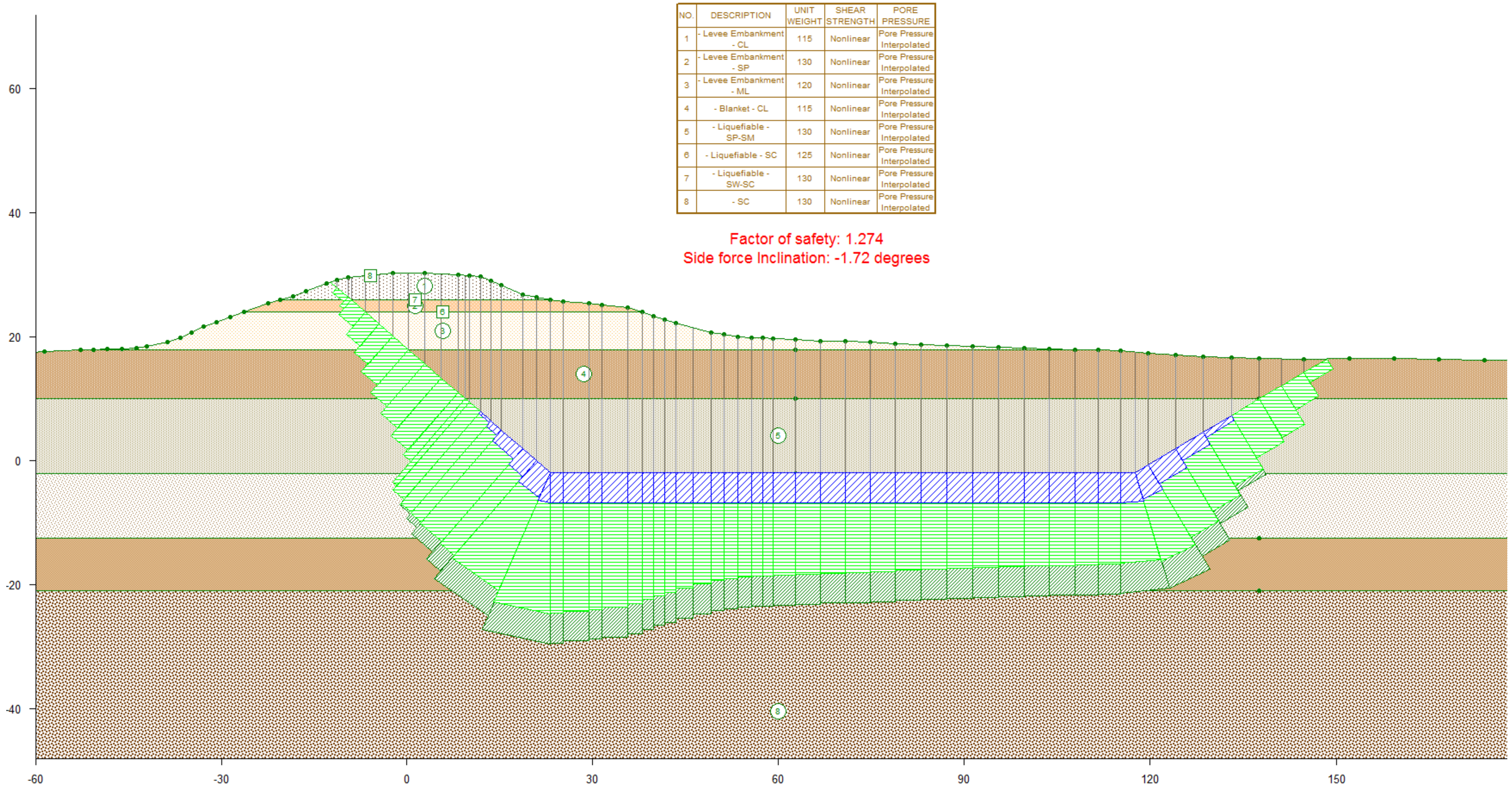
Fig F-(7). RD 17 Station 1595+33 – Landside – Option 3: Circular (PHI = 3.9 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SP	130	Nonlinear	Pore Pressure Interpolated
3	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
4	- Blanket - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Liquefiable - SP-SM	130	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable - SW-SC	130	Nonlinear	Pore Pressure Interpolated
8	- SC	130	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.261
Side force Inclination: -1.51 degrees

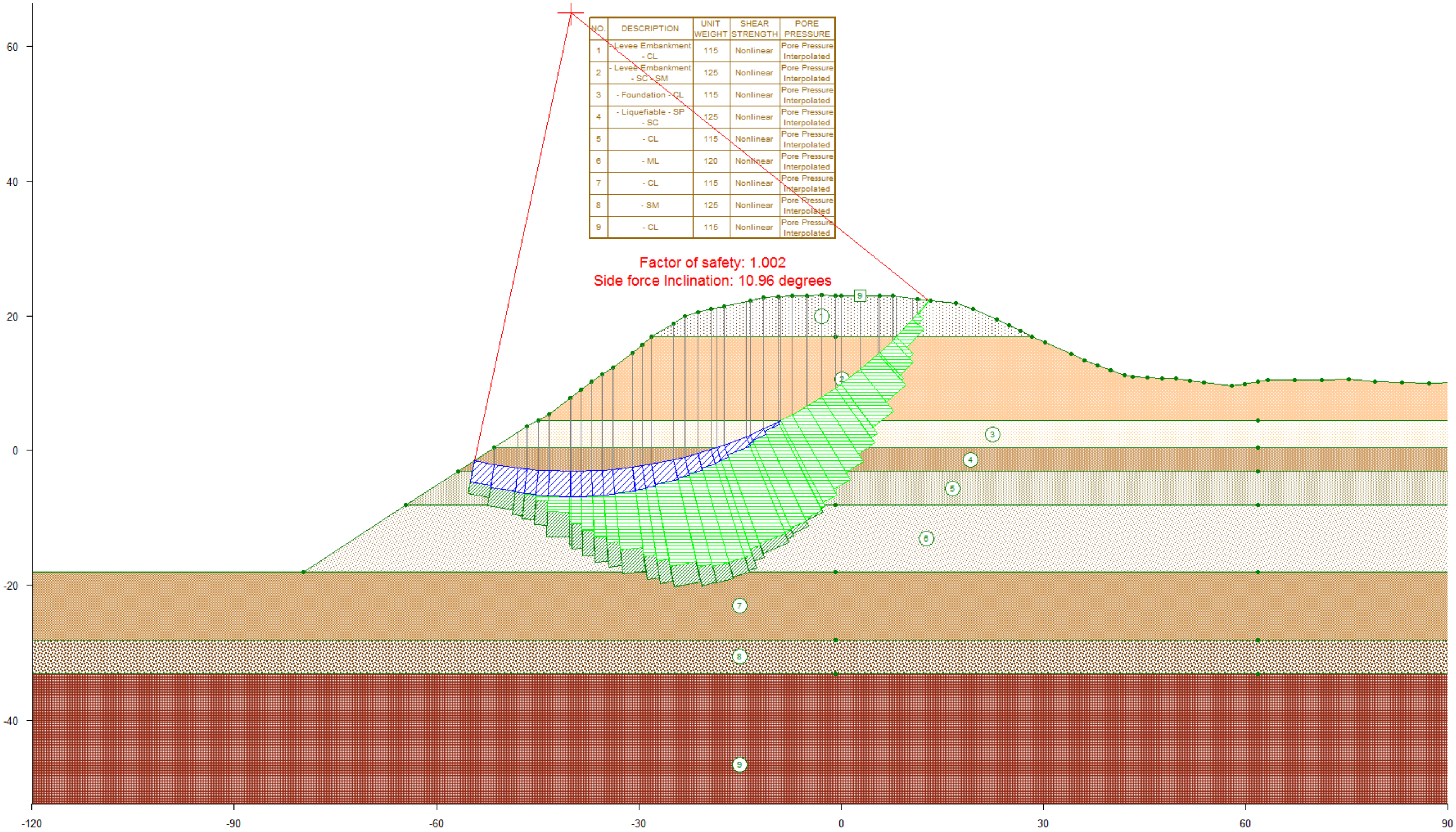
Fig F-8(a). RD 17 Southern, Station 1595+33 – Landside – Option 4: Wedge (Sr = 133 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SP	130	Nonlinear	Pore Pressure Interpolated
3	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
4	- Blanket - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Liquefiable - SP-SM	130	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable - SW-SC	130	Nonlinear	Pore Pressure Interpolated
8	- SC	130	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.274
Side force Inclination: -1.72 degrees

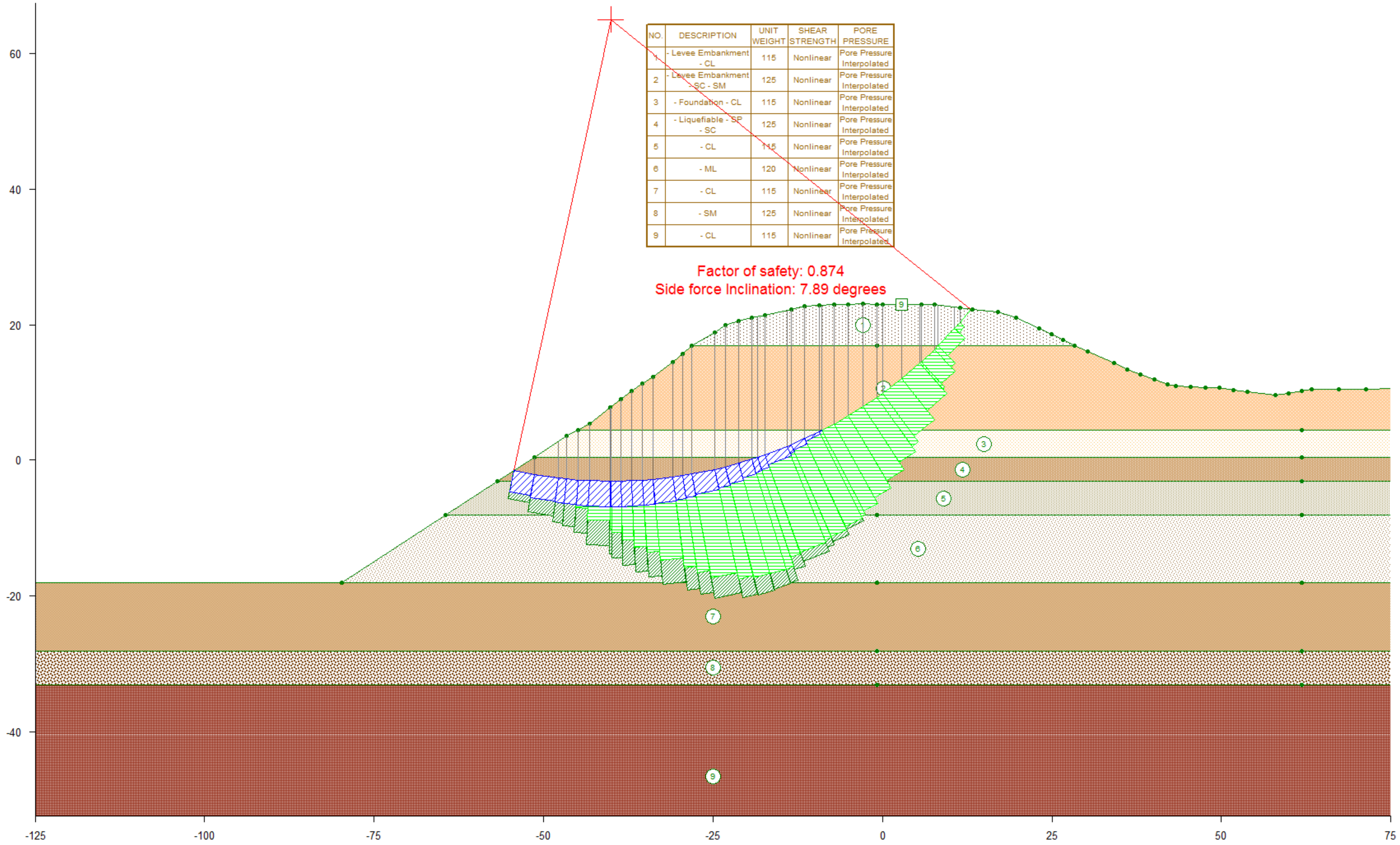
Fig F-8(b). RD 17 Station 1595+33 – Landside – Option 4: Wedge (PHI = 3.9 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SC - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP - SC	125	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- ML	120	Nonlinear	Pore Pressure Interpolated
7	- CL	115	Nonlinear	Pore Pressure Interpolated
8	- SM	125	Nonlinear	Pore Pressure Interpolated
9	- CL	115	Nonlinear	Pore Pressure Interpolated

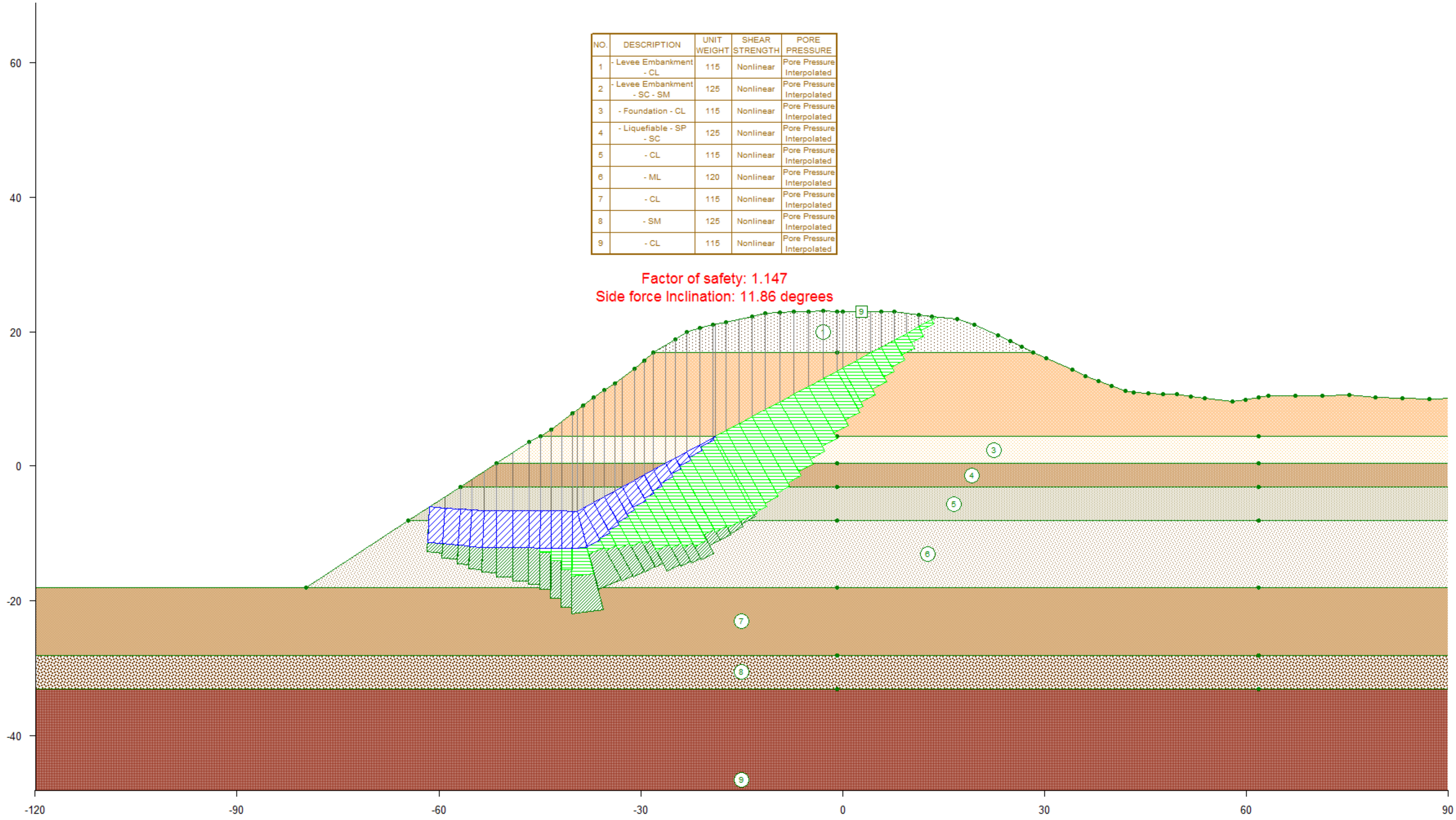
Factor of safety: 1.002
Side force Inclination: 10.96 degrees

Fig F-9(a). RD 17 Northern, Station 1151+06 – Waterside – Option 1: Circular (Sr = 201 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SC - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP - SC	125	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- ML	120	Nonlinear	Pore Pressure Interpolated
7	- CL	115	Nonlinear	Pore Pressure Interpolated
8	- SM	125	Nonlinear	Pore Pressure Interpolated
9	- CL	115	Nonlinear	Pore Pressure Interpolated

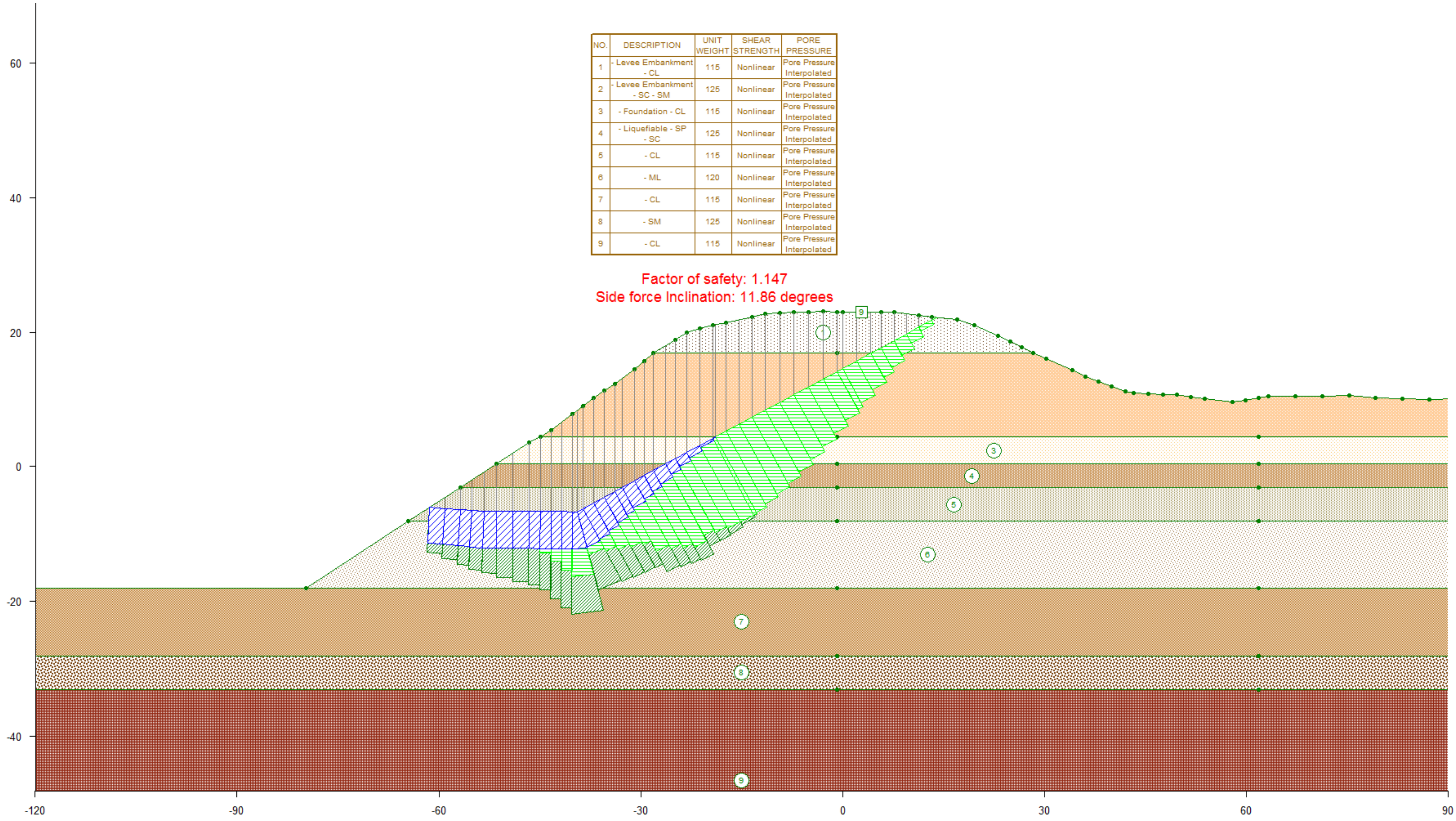
Fig F-9(b). RD 17 Station 1151+06 – Waterside – Option 1: Circular (PHI = 5.2 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SC - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP - SC	125	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- ML	120	Nonlinear	Pore Pressure Interpolated
7	- CL	115	Nonlinear	Pore Pressure Interpolated
8	- SM	125	Nonlinear	Pore Pressure Interpolated
9	- CL	115	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.147
Side force Inclination: 11.86 degrees

Fig F-10(a). RD 17 Northern, Station 1151+06– Waterside – Option 2: Wedges (Sr = 201 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SC - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP - SC	125	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- ML	120	Nonlinear	Pore Pressure Interpolated
7	- CL	115	Nonlinear	Pore Pressure Interpolated
8	- SM	125	Nonlinear	Pore Pressure Interpolated
9	- CL	115	Nonlinear	Pore Pressure Interpolated

Fig F-10(b). RD 17 Station 1151+06 – Waterside – Option 2: Wedges (PHI = 5.2 in liquefiable material)

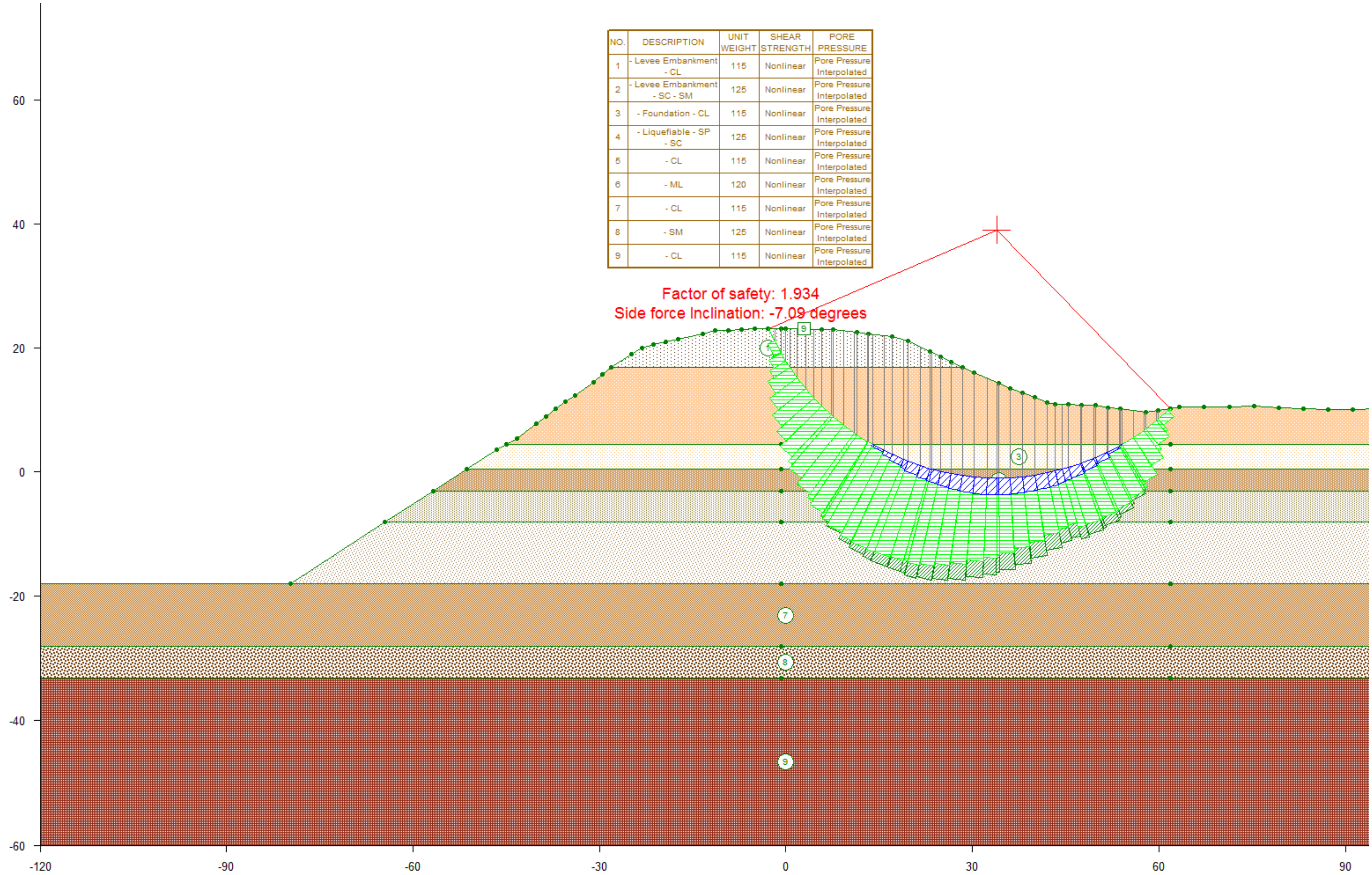
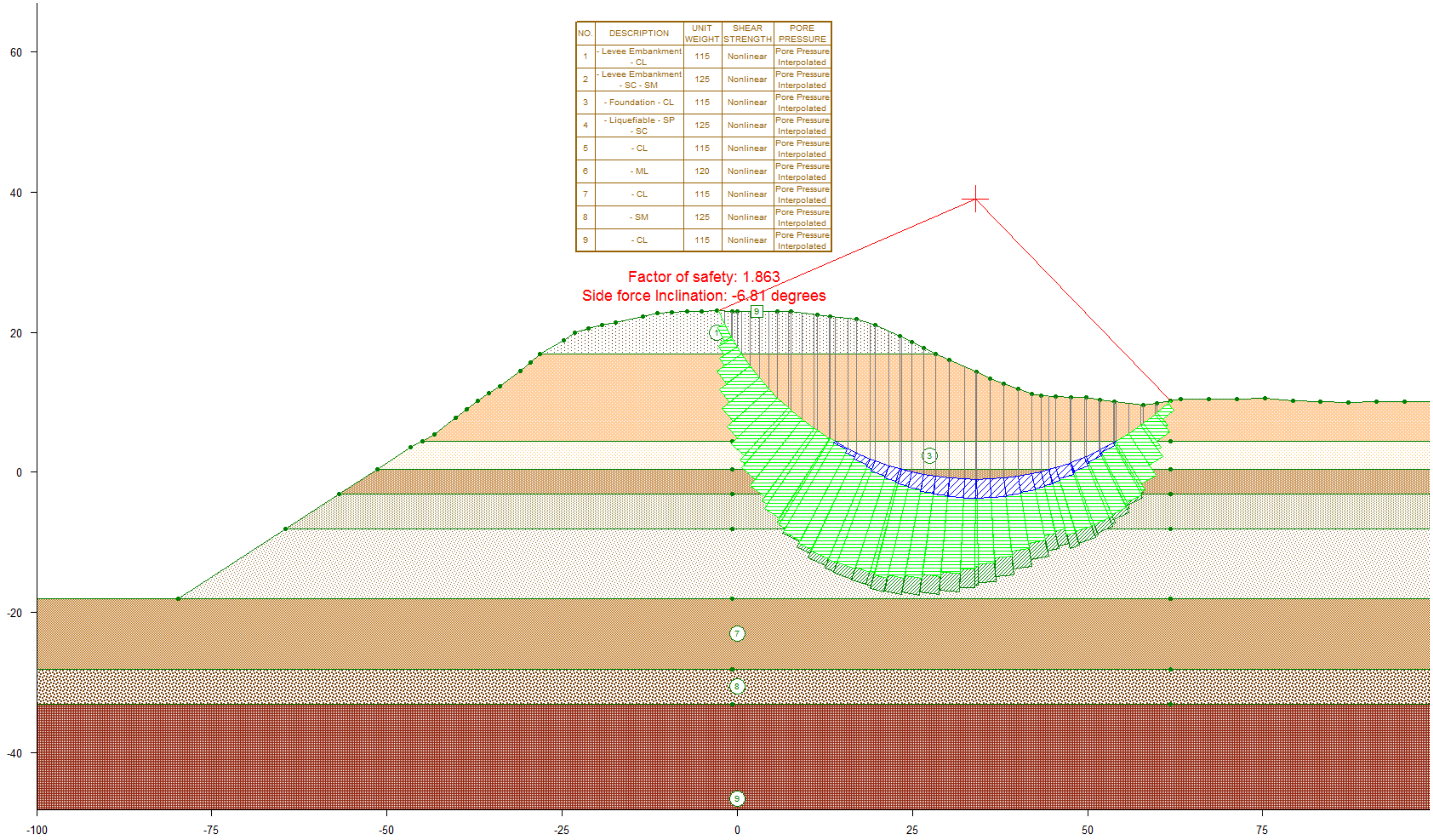
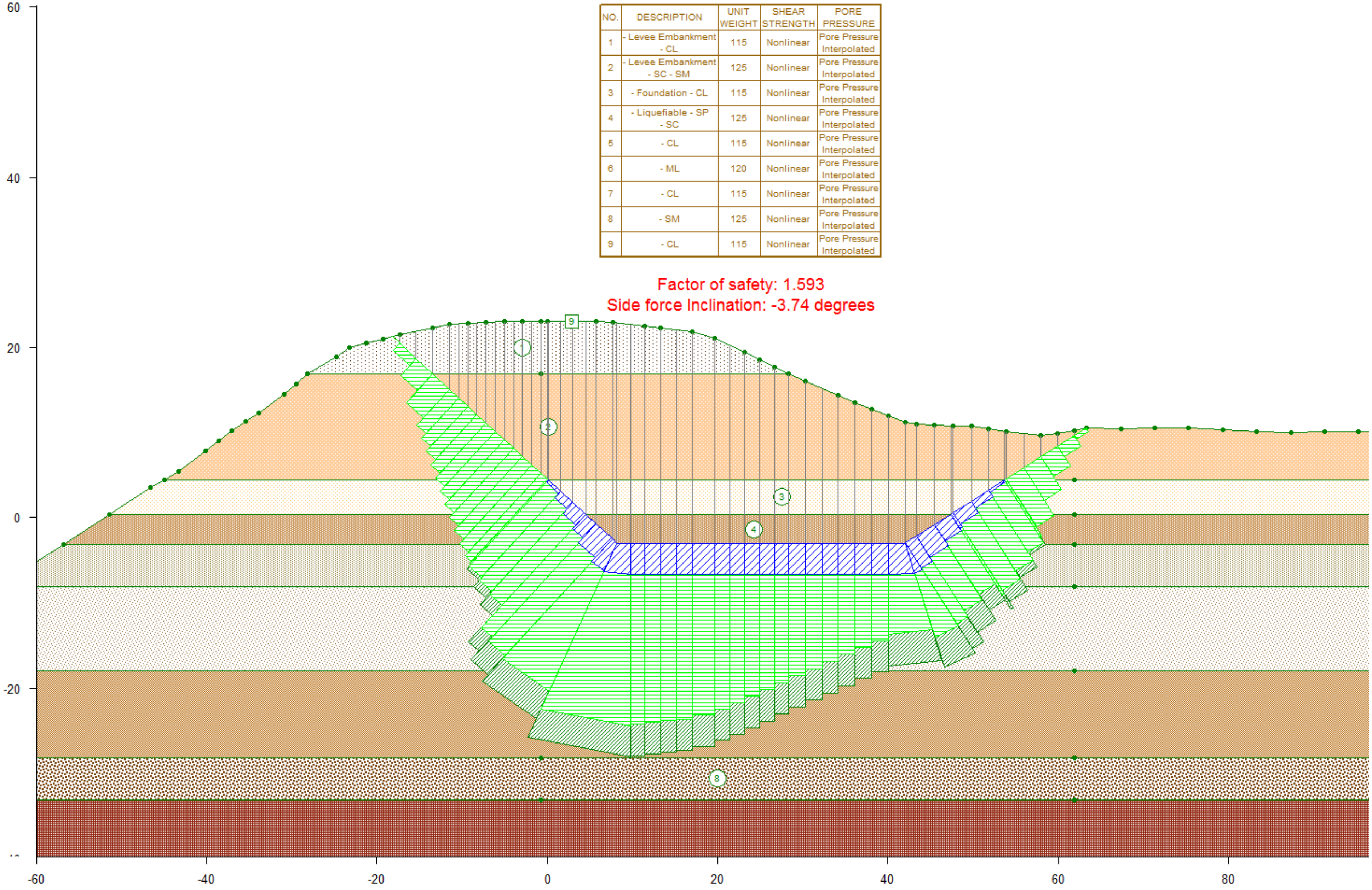


Fig F-11(a). RD 17 Northern, Station 1151+06– Landside – Option 3: Circular (Sr = 201 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SC - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP - SC	125	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- ML	120	Nonlinear	Pore Pressure Interpolated
7	- CL	115	Nonlinear	Pore Pressure Interpolated
8	- SM	125	Nonlinear	Pore Pressure Interpolated
9	- CL	115	Nonlinear	Pore Pressure Interpolated

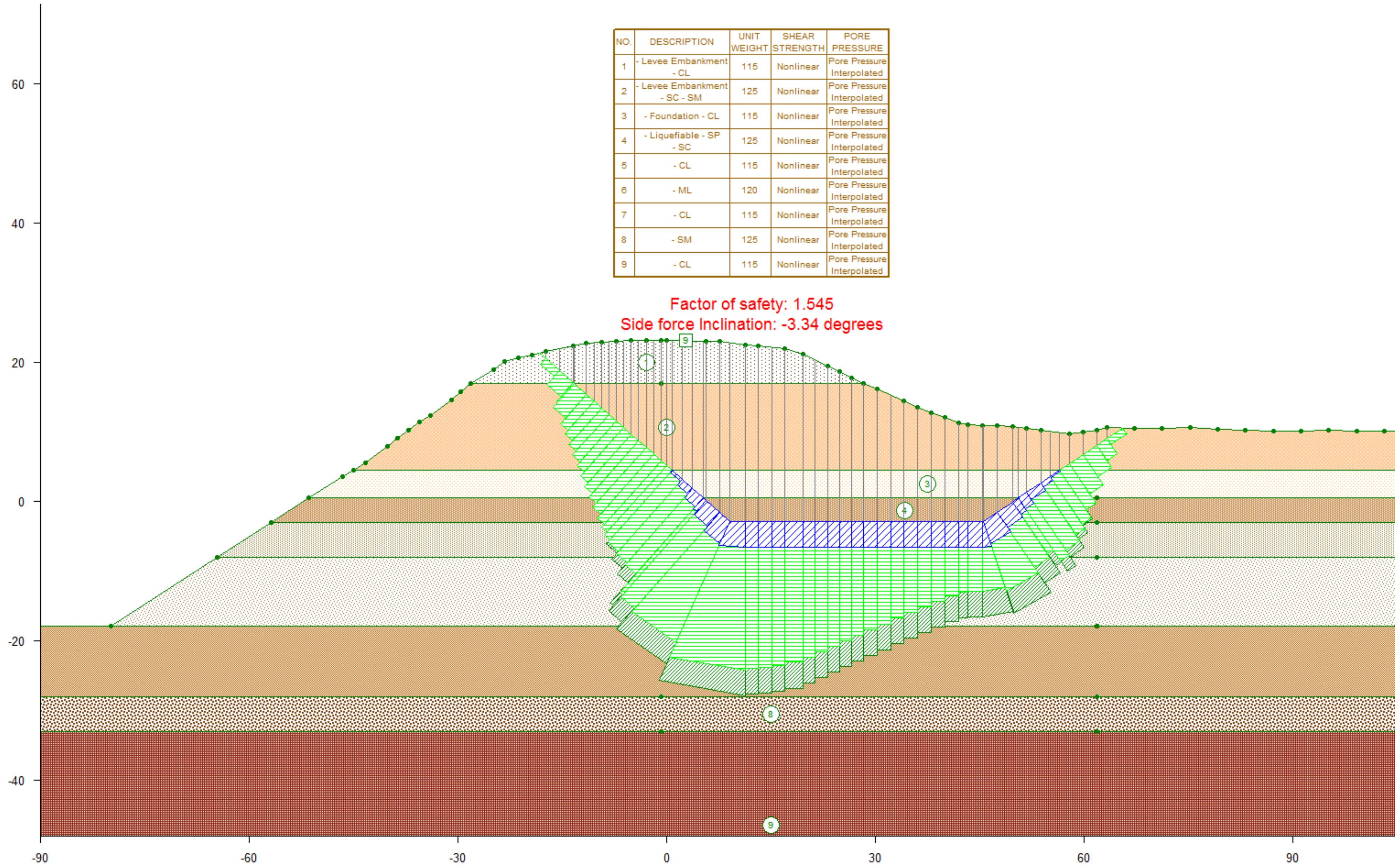
Fig F-11(b). RD 17 Station 1151+06 – Landside – Option 3: Circular (PHI = 5.2 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SC - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP - SC	125	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- ML	120	Nonlinear	Pore Pressure Interpolated
7	- CL	115	Nonlinear	Pore Pressure Interpolated
8	- SM	125	Nonlinear	Pore Pressure Interpolated
9	- CL	115	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.593
Side force Inclination: -3.74 degrees

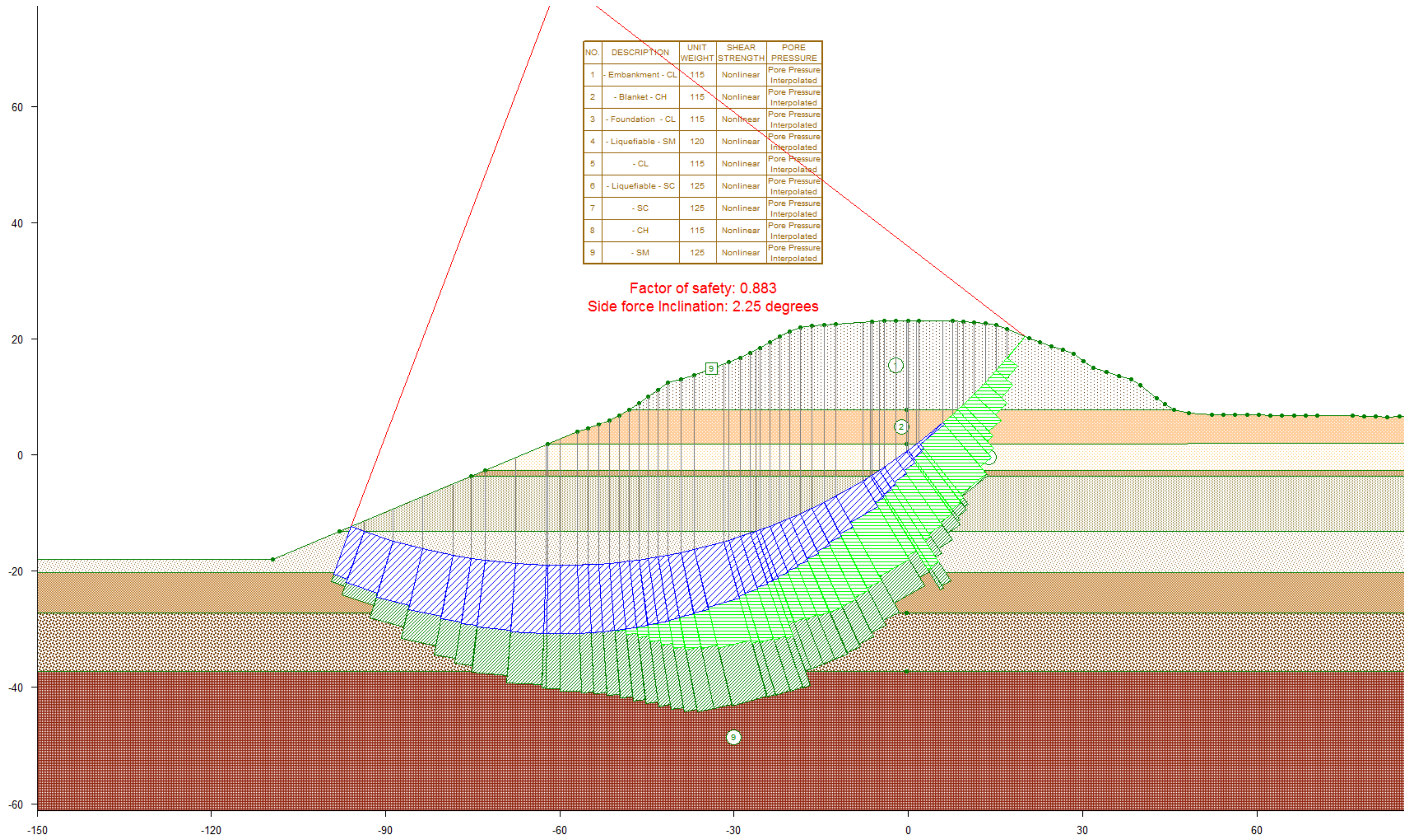
Fig F-12(a). RD 17 Northern, Station 1151+06– Landside – Option 4: Wedge (Sr = 201 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Levee Embankment - SC - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP - SC	125	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- ML	120	Nonlinear	Pore Pressure Interpolated
7	- CL	115	Nonlinear	Pore Pressure Interpolated
8	- SM	125	Nonlinear	Pore Pressure Interpolated
9	- CL	115	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.545
Side force Inclination: -3.34 degrees

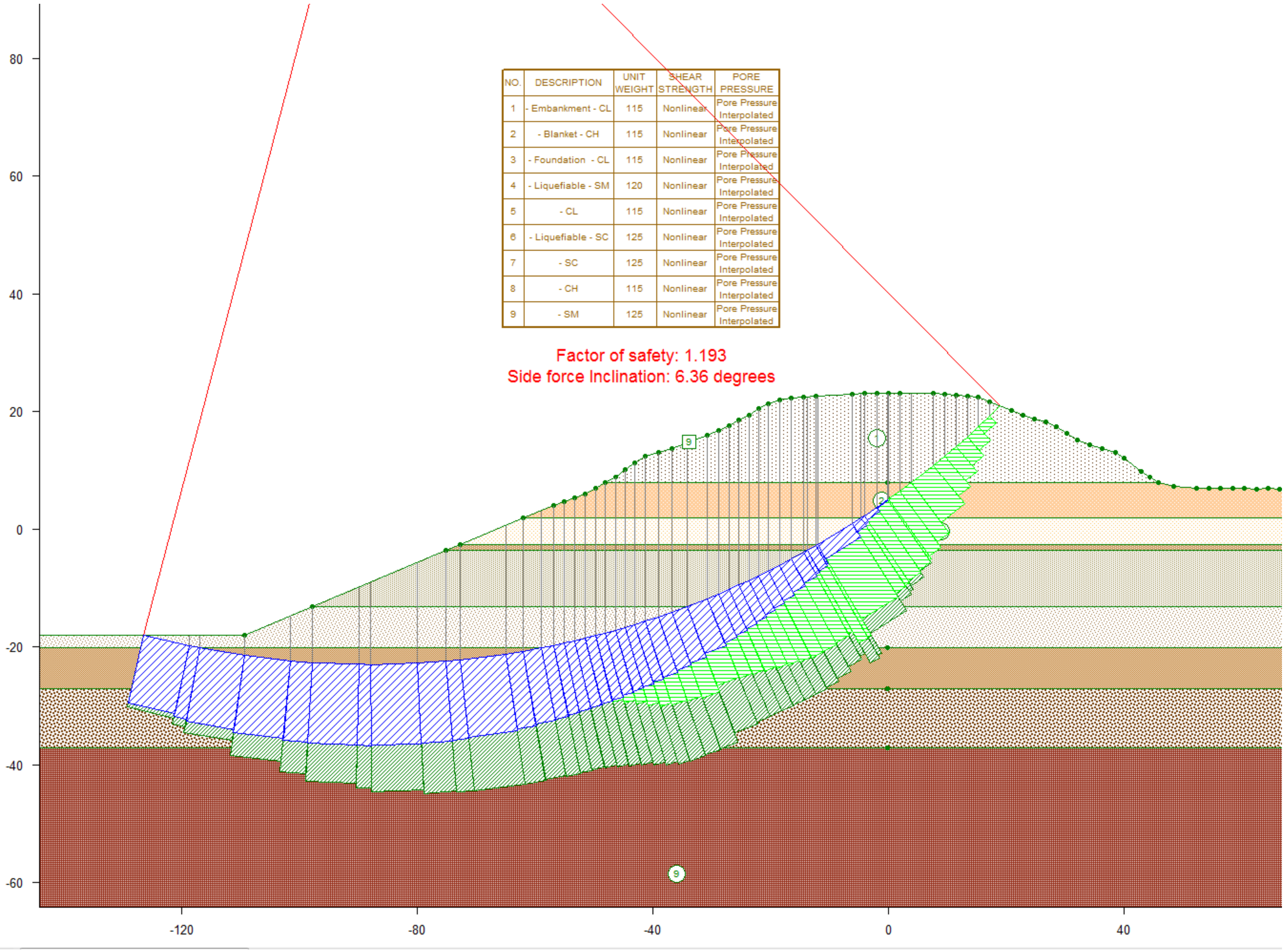
Fig F-12(b). RD 17 Northern, Station 1151+06– Landside – Option 4: Wedge (PHI = 5.2 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Blanket - CH	115	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SM	120	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- SC	125	Nonlinear	Pore Pressure Interpolated
8	- CH	115	Nonlinear	Pore Pressure Interpolated
9	- SM	125	Nonlinear	Pore Pressure Interpolated

Factor of safety: 0.883
Side force Inclination: 2.25 degrees

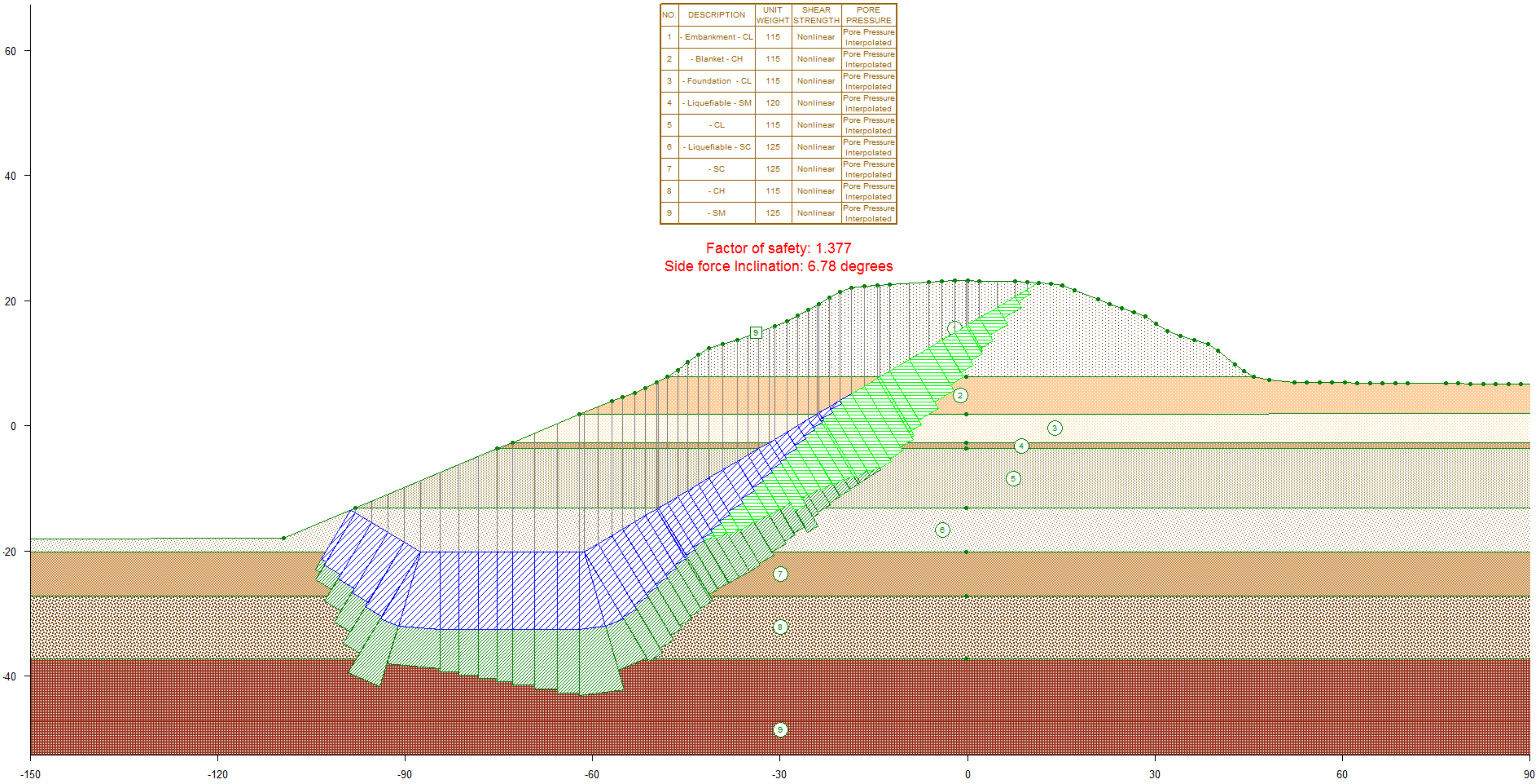
Fig F-13(a). RD 17 Northern, Station 1191+43 – Waterside – Option 1: Circular (Sr = 164 & 111 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Blanket - CH	115	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SM	120	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- SC	125	Nonlinear	Pore Pressure Interpolated
8	- CH	115	Nonlinear	Pore Pressure Interpolated
9	- SM	125	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.193
Side force Inclination: 6.36 degrees

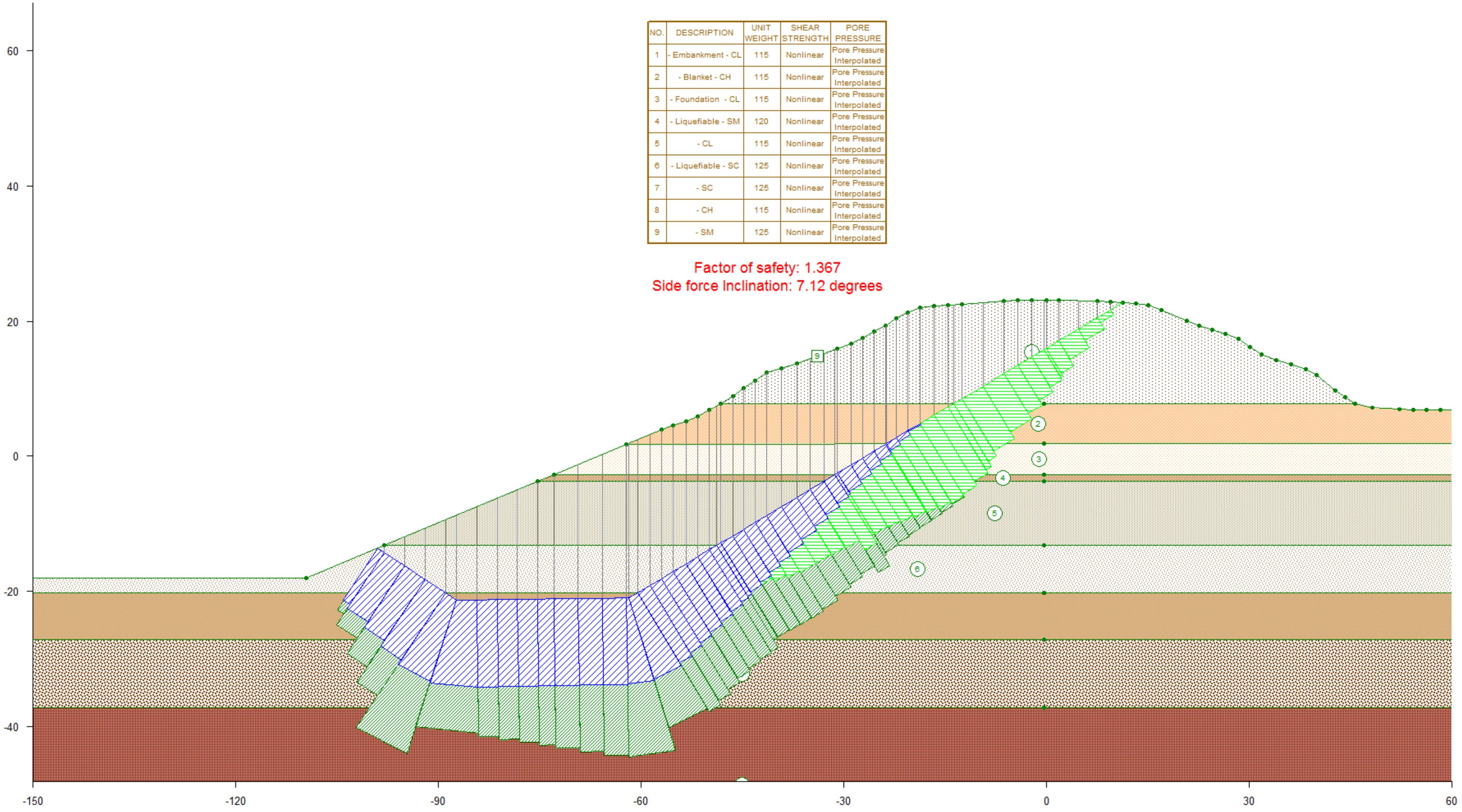
Fig F-13(b). RD 17 Station 1191+43 – Waterside – Option 1: Circular (PHI = 4.3 & 2.7 in liquefiable materials)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Blanket - CH	115	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SM	120	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- SC	125	Nonlinear	Pore Pressure Interpolated
8	- CH	115	Nonlinear	Pore Pressure Interpolated
9	- SM	125	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.377
Side force Inclination: 6.78 degrees

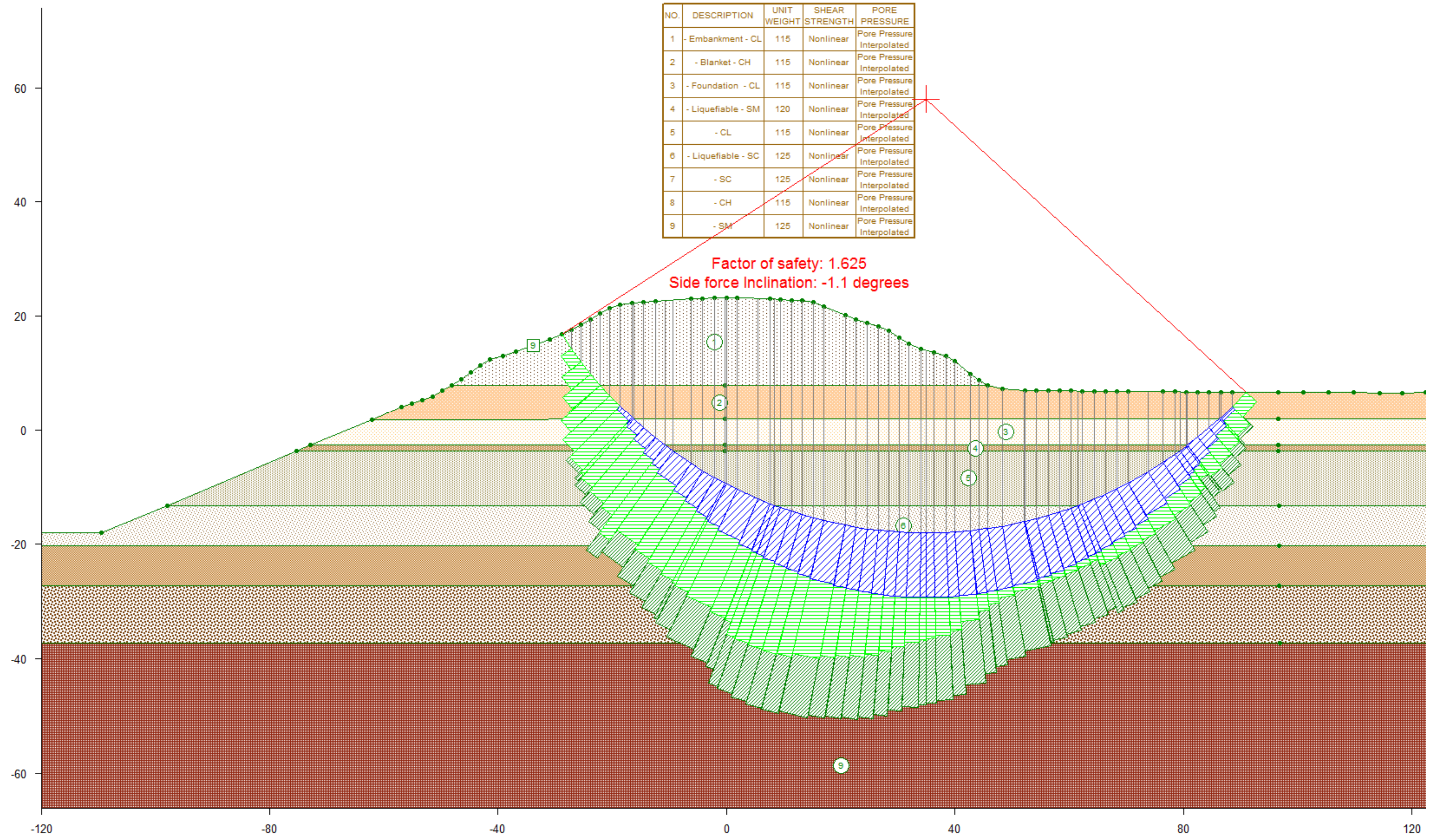
Fig F-14(a). RD 17 Northern, Station 1191+43– Waterside – Option 2: Wedges (Sr = 164 & 111 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Blanket - CH	115	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SM	120	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- SC	125	Nonlinear	Pore Pressure Interpolated
8	- CH	115	Nonlinear	Pore Pressure Interpolated
9	- SM	125	Nonlinear	Pore Pressure Interpolated

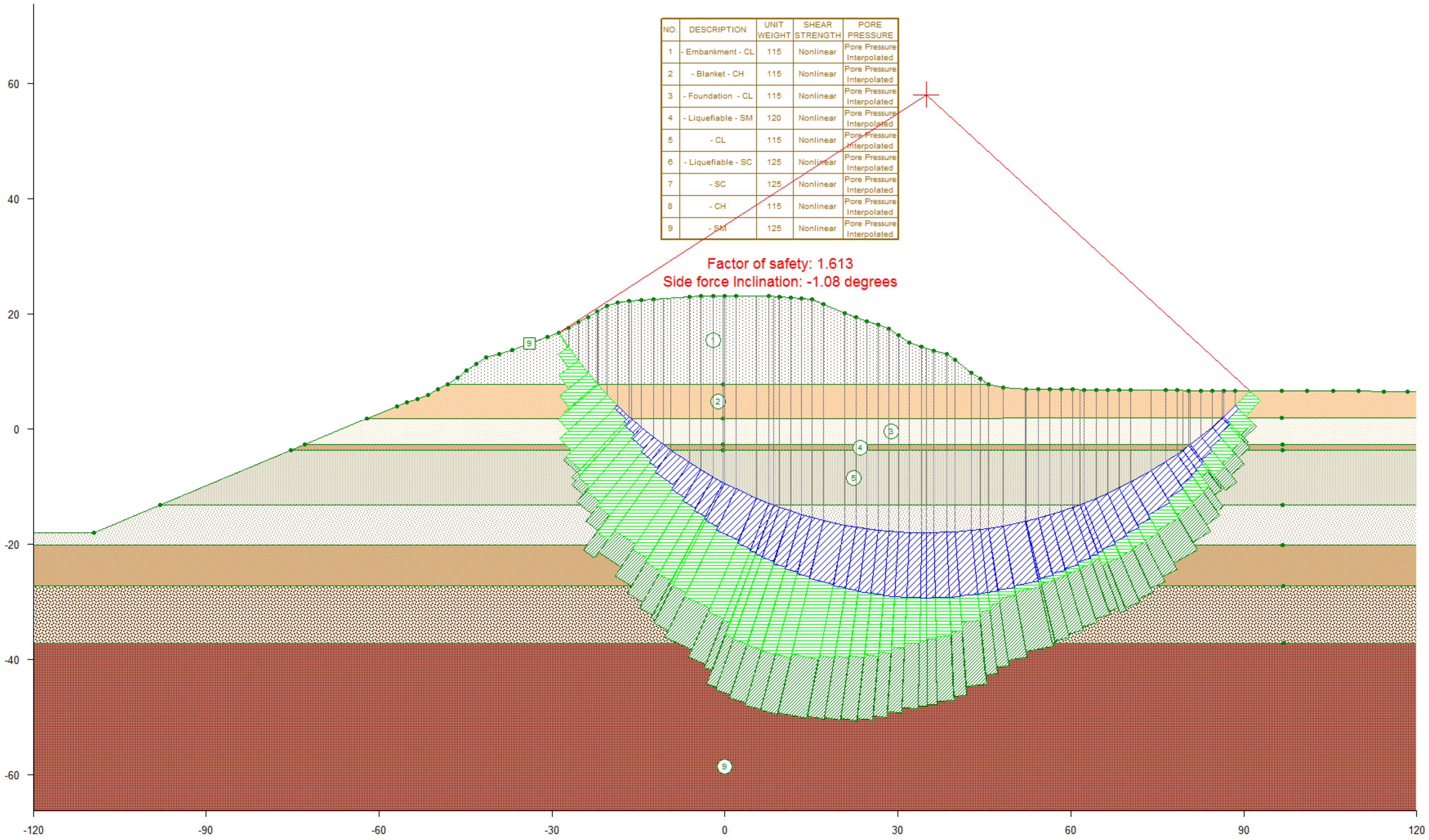
Factor of safety: 1.367
Side force Inclination: 7.12 degrees

Fig F-14(b). RD 17 Station 1191+43– Waterside – Option 2: Wedges (PHI = 4.3 & 2.7 in liquefiable materials)



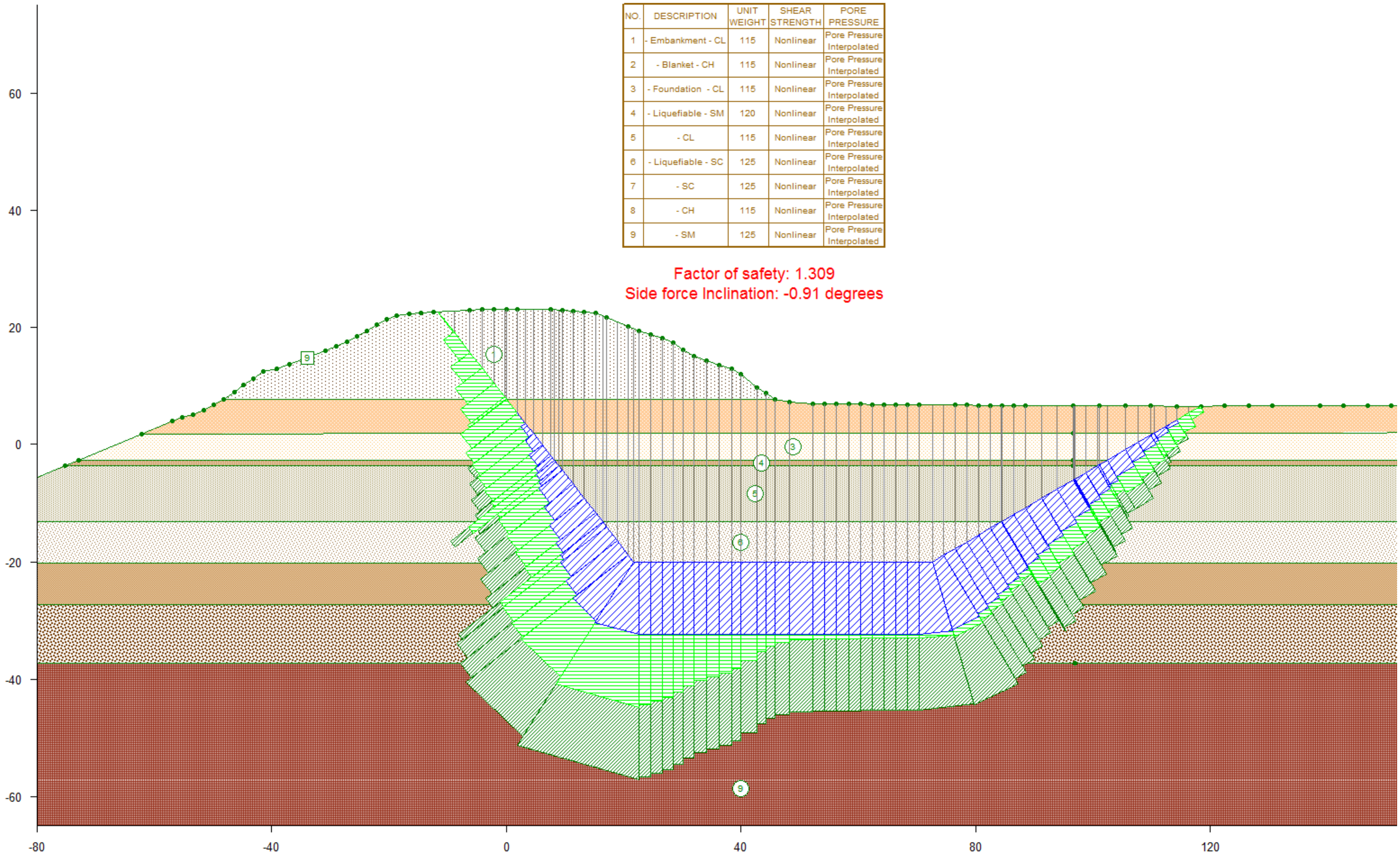
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Blanket - CH	115	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SM	120	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- SC	125	Nonlinear	Pore Pressure Interpolated
8	- CH	115	Nonlinear	Pore Pressure Interpolated
9	- SM	125	Nonlinear	Pore Pressure Interpolated

Fig F-15(a). RD 17 Northern, Station 1191+43 – Landside – Option 3: Circular (Sr = 164 & 111 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Blanket - CH	115	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SM	120	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- SC	125	Nonlinear	Pore Pressure Interpolated
8	- CH	115	Nonlinear	Pore Pressure Interpolated
9	- SM	125	Nonlinear	Pore Pressure Interpolated

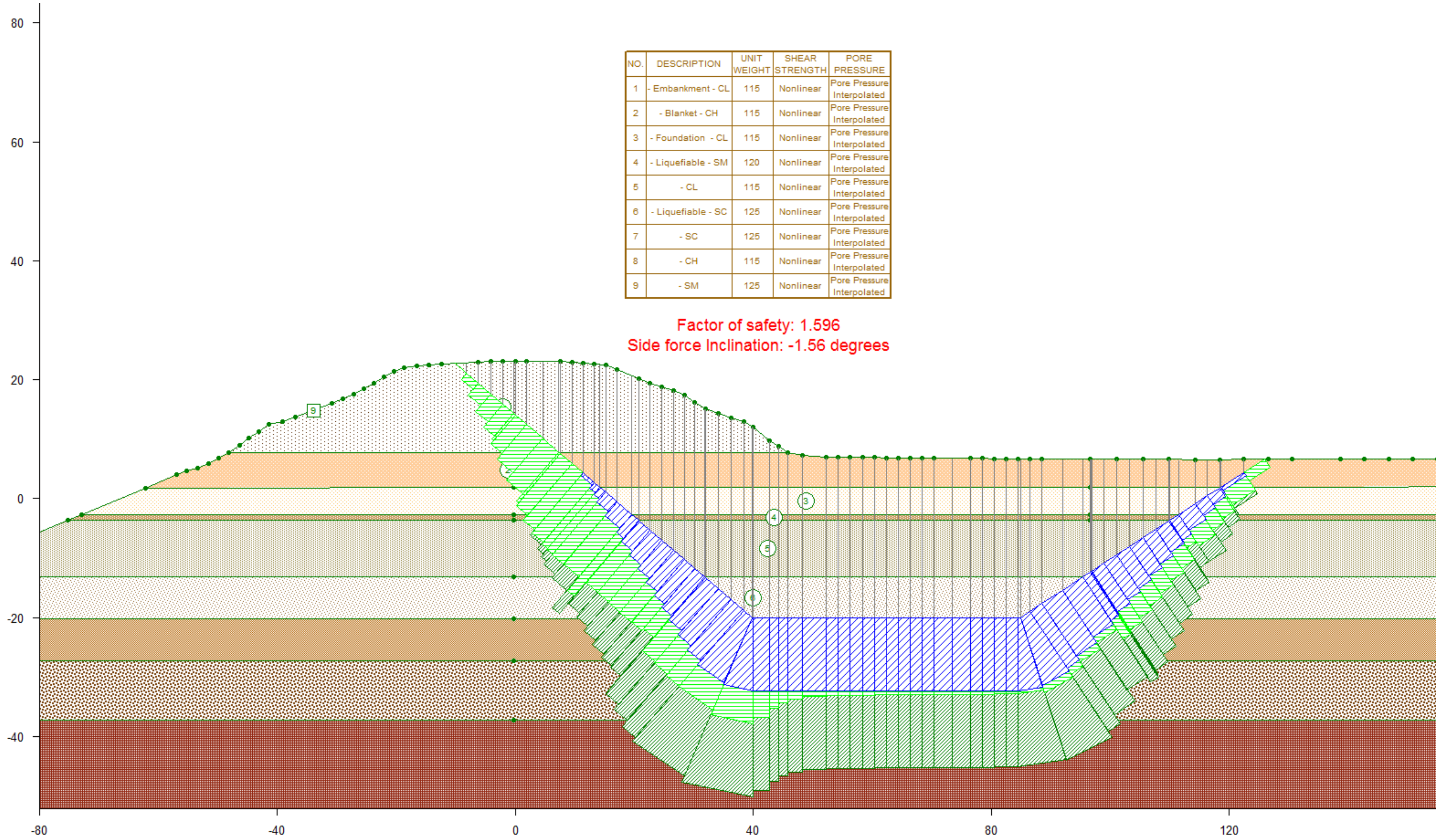
Fig F-15(b). RD 17 Station 1191+43 – Landside – Option 3: Circular (PHI = 4.3 & 2.7 in liquefiable materials)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Blanket - CH	115	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SM	120	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- SC	125	Nonlinear	Pore Pressure Interpolated
8	- CH	115	Nonlinear	Pore Pressure Interpolated
9	- SM	125	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.309
Side force Inclination: -0.91 degrees

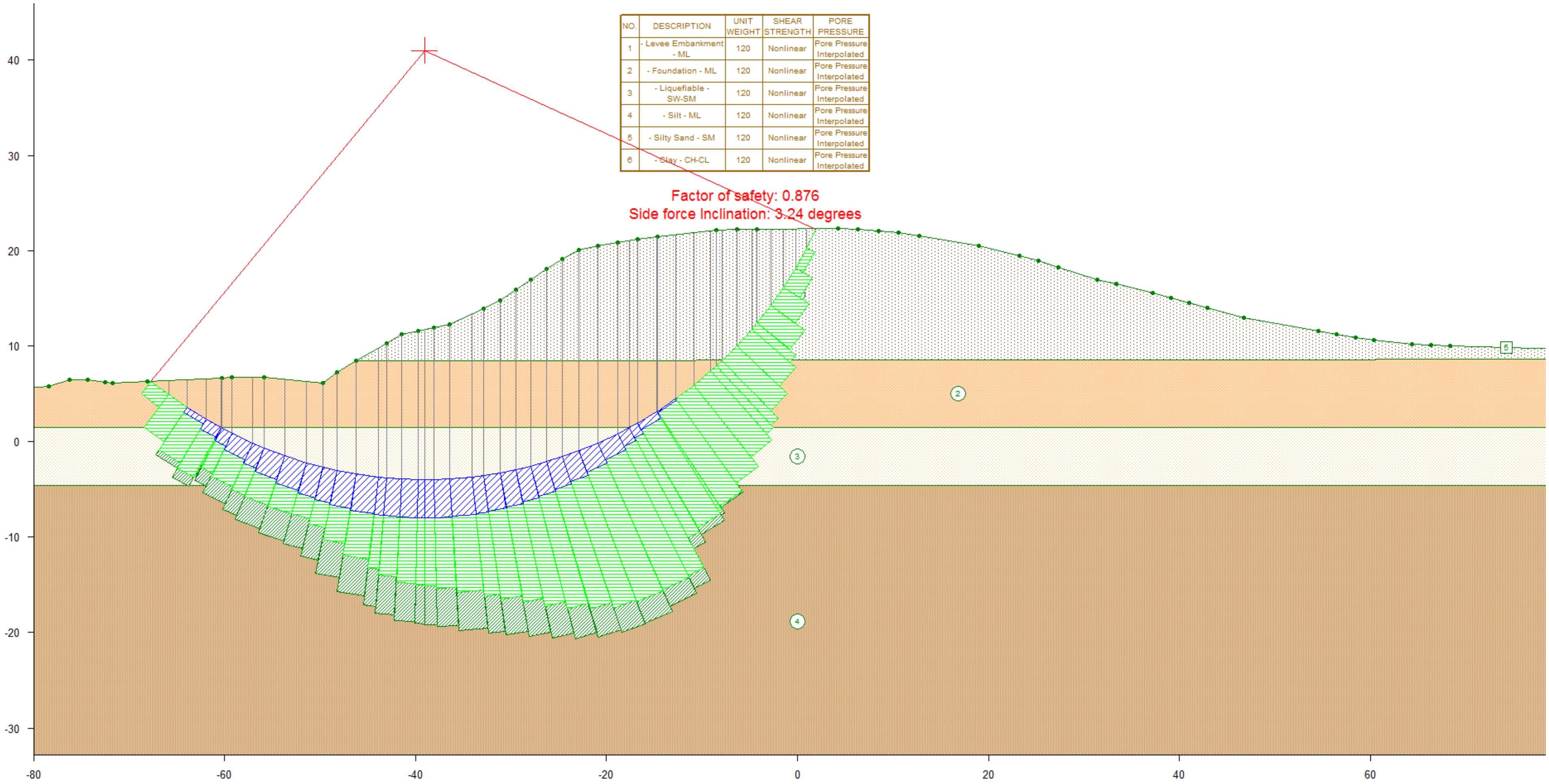
Fig F-16(a). RD 17 Northern, Station 1191+43 – Landside – Option 4: Wedge (Sr = 164 & 111 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Blanket - CH	115	Nonlinear	Pore Pressure Interpolated
3	- Foundation - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SM	120	Nonlinear	Pore Pressure Interpolated
5	- CL	115	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable - SC	125	Nonlinear	Pore Pressure Interpolated
7	- SC	125	Nonlinear	Pore Pressure Interpolated
8	- CH	115	Nonlinear	Pore Pressure Interpolated
9	- SM	125	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.596
Side force Inclination: -1.56 degrees

Fig F-16(b). RD 17 Station 1191+43 – Landside – Option 4: Wedge (PHI = 4.3 & 2.7 in liquefiable materials)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable - SW-SM	120	Nonlinear	Pore Pressure Interpolated
4	- Silt - ML	120	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	120	Nonlinear	Pore Pressure Interpolated
6	- Clay - CH-CL	120	Nonlinear	Pore Pressure Interpolated

Fig F-17(a). RD 404 Station 1175+01 – Waterside – Option 1: Circular (Sr = 113 psf in liquefiable material)

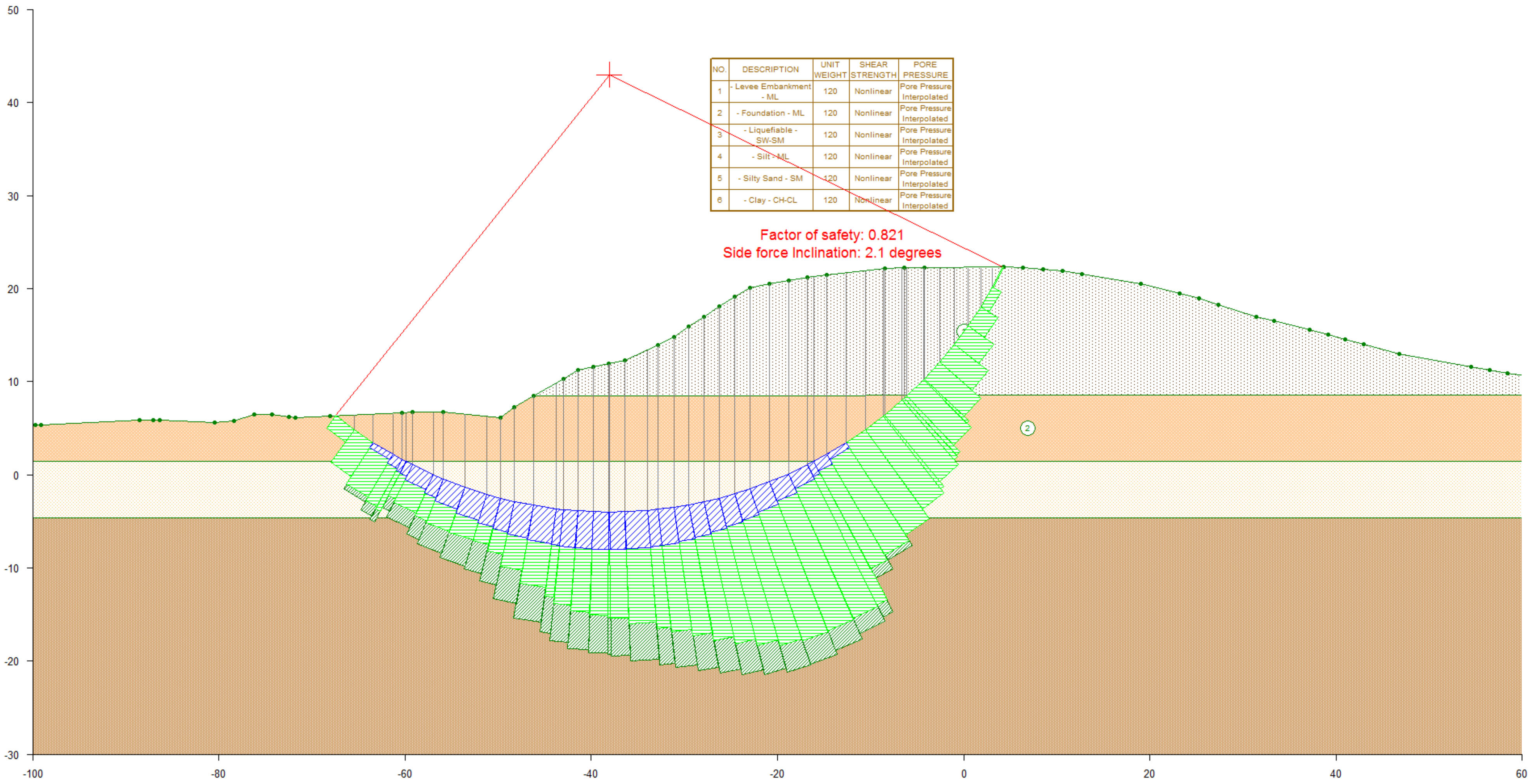
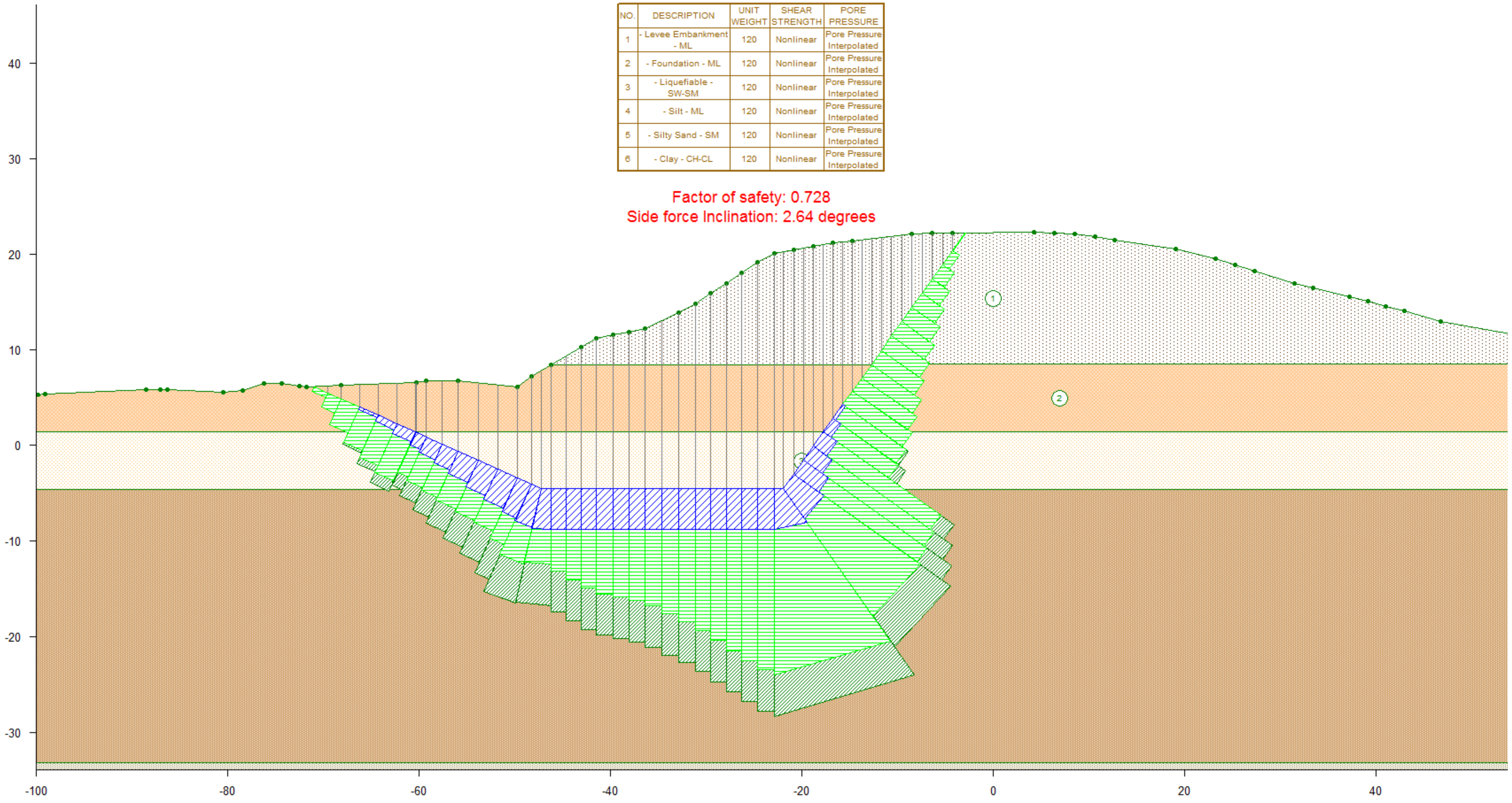


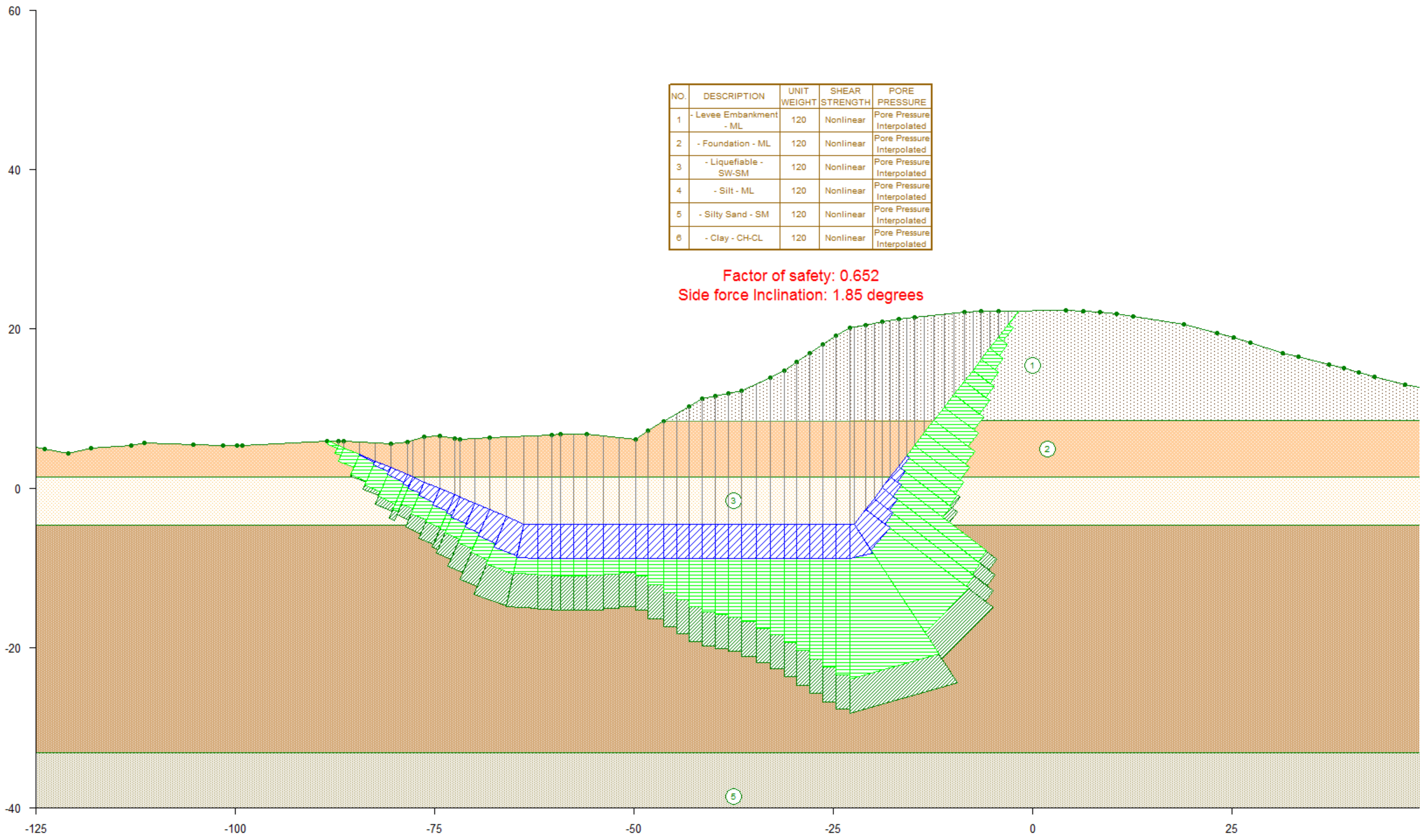
Fig F-17(b). RD 404 Station 1175+01 – Waterside – Option 1: Circular (PHI = 3.6 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable - SW-SM	120	Nonlinear	Pore Pressure Interpolated
4	- Silt - ML	120	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	120	Nonlinear	Pore Pressure Interpolated
6	- Clay - CH-CL	120	Nonlinear	Pore Pressure Interpolated

Factor of safety: 0.728
Side force Inclination: 2.64 degrees

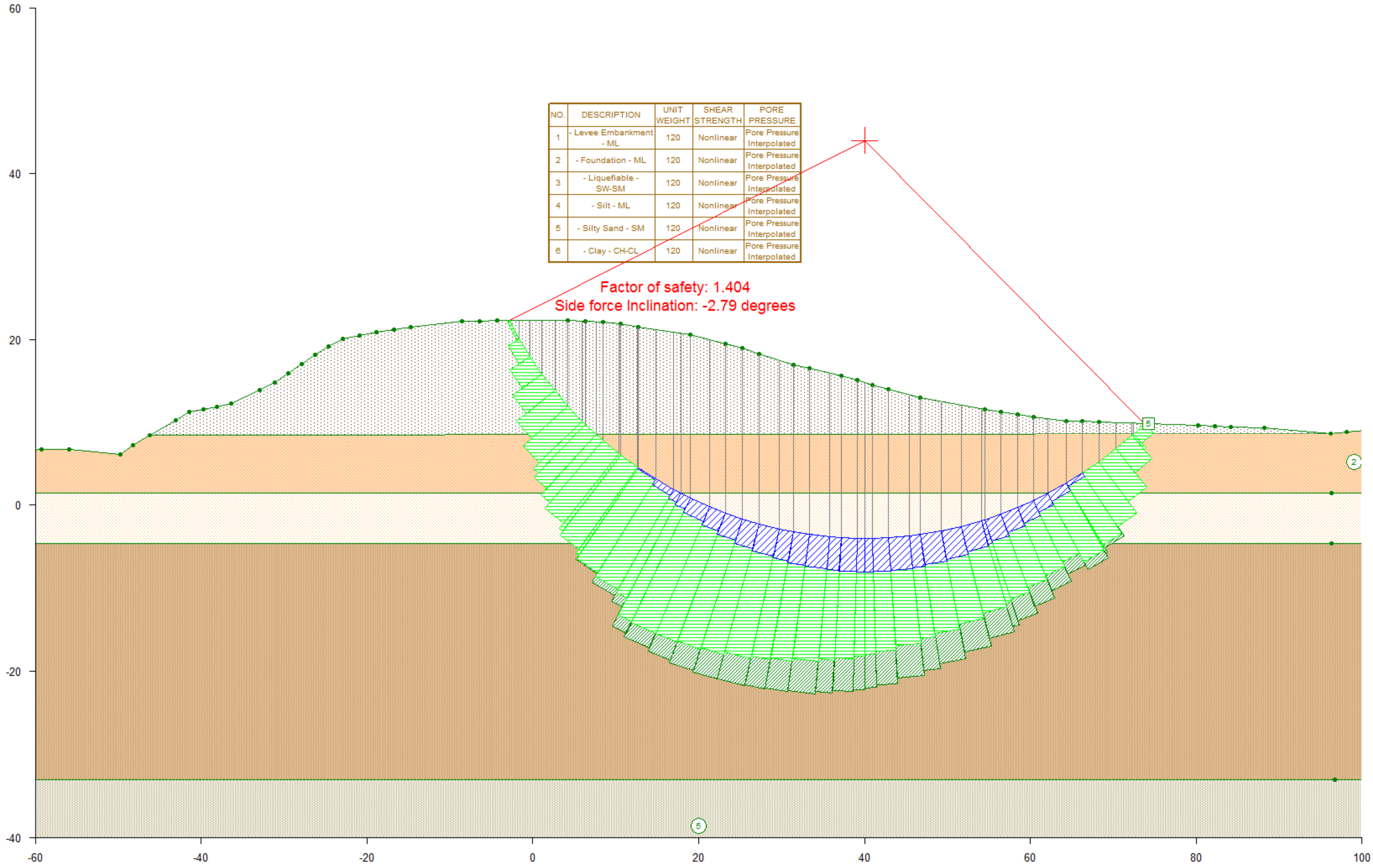
Fig F-18(a). RD 404 Station 1175+01 – Waterside – Option 2: Wedges (Sr = 113 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable - SW-SM	120	Nonlinear	Pore Pressure Interpolated
4	- Silt - ML	120	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	120	Nonlinear	Pore Pressure Interpolated
6	- Clay - CH-CL	120	Nonlinear	Pore Pressure Interpolated

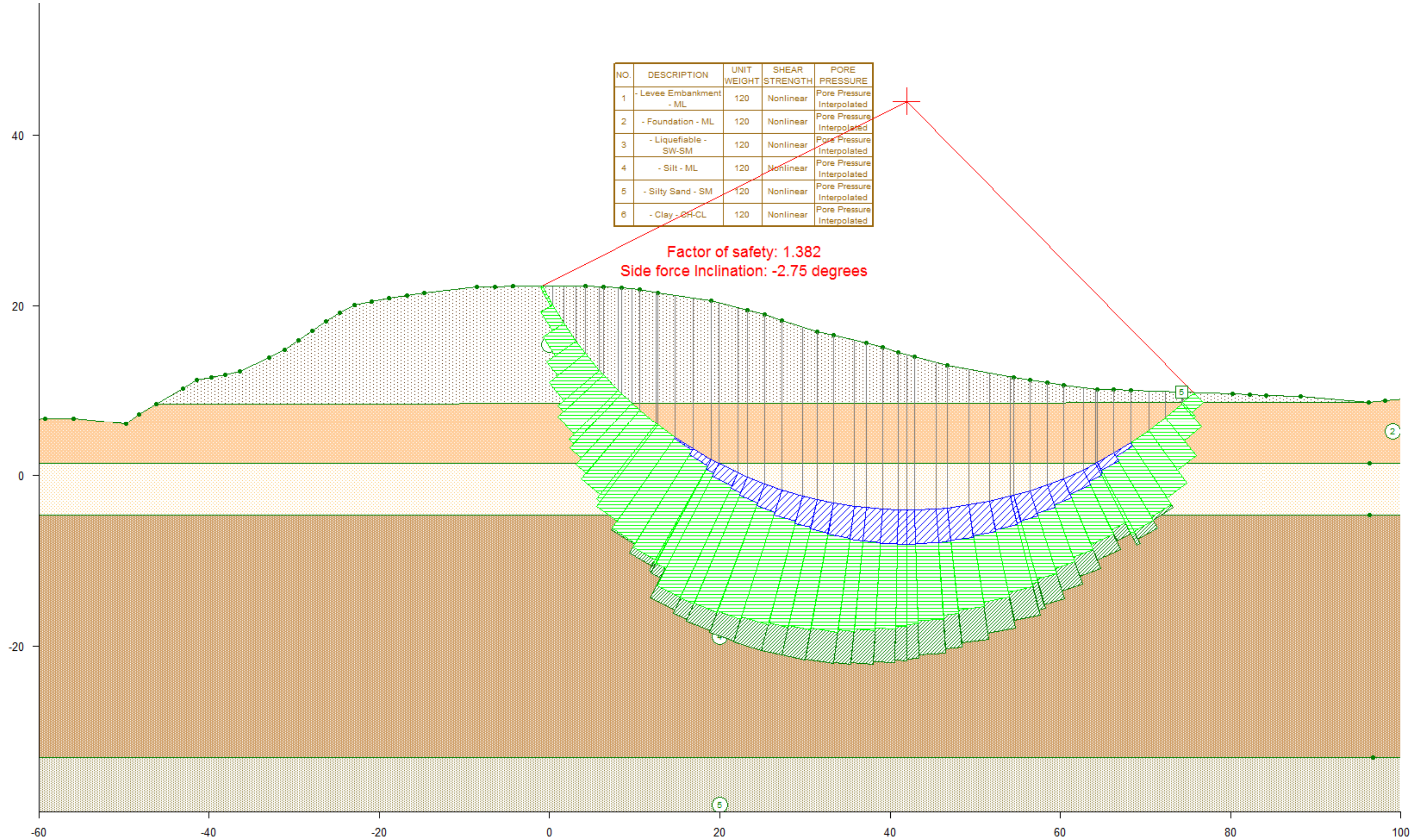
Factor of safety: 0.652
Side force Inclination: 1.85 degrees

Fig F-18(b). RD 404 Station 1175+01 – Waterside – Option 2: Wedges (PHI = 3.6 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable - SW-SM	120	Nonlinear	Pore Pressure Interpolated
4	- Silt - ML	120	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	120	Nonlinear	Pore Pressure Interpolated
6	- Clay - CH-CL	120	Nonlinear	Pore Pressure Interpolated

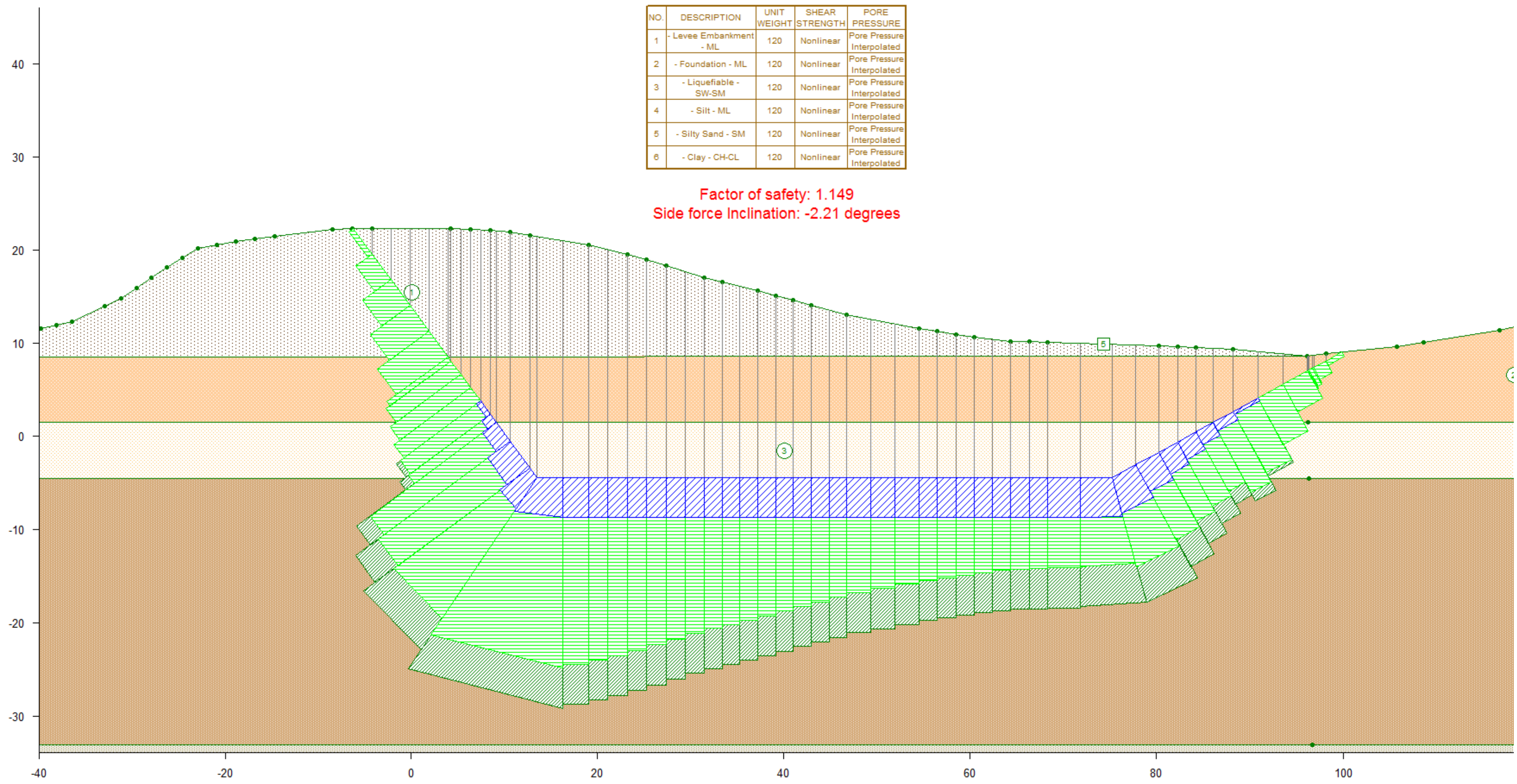
Fig F-19(a). RD 404 Station 1175+01 – Landside – Option 3: Circular (Sr = 113 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable - SW-SM	120	Nonlinear	Pore Pressure Interpolated
4	- Silt - ML	120	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	120	Nonlinear	Pore Pressure Interpolated
6	- Clay - CH-CL	120	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.382
Side force Inclination: -2.75 degrees

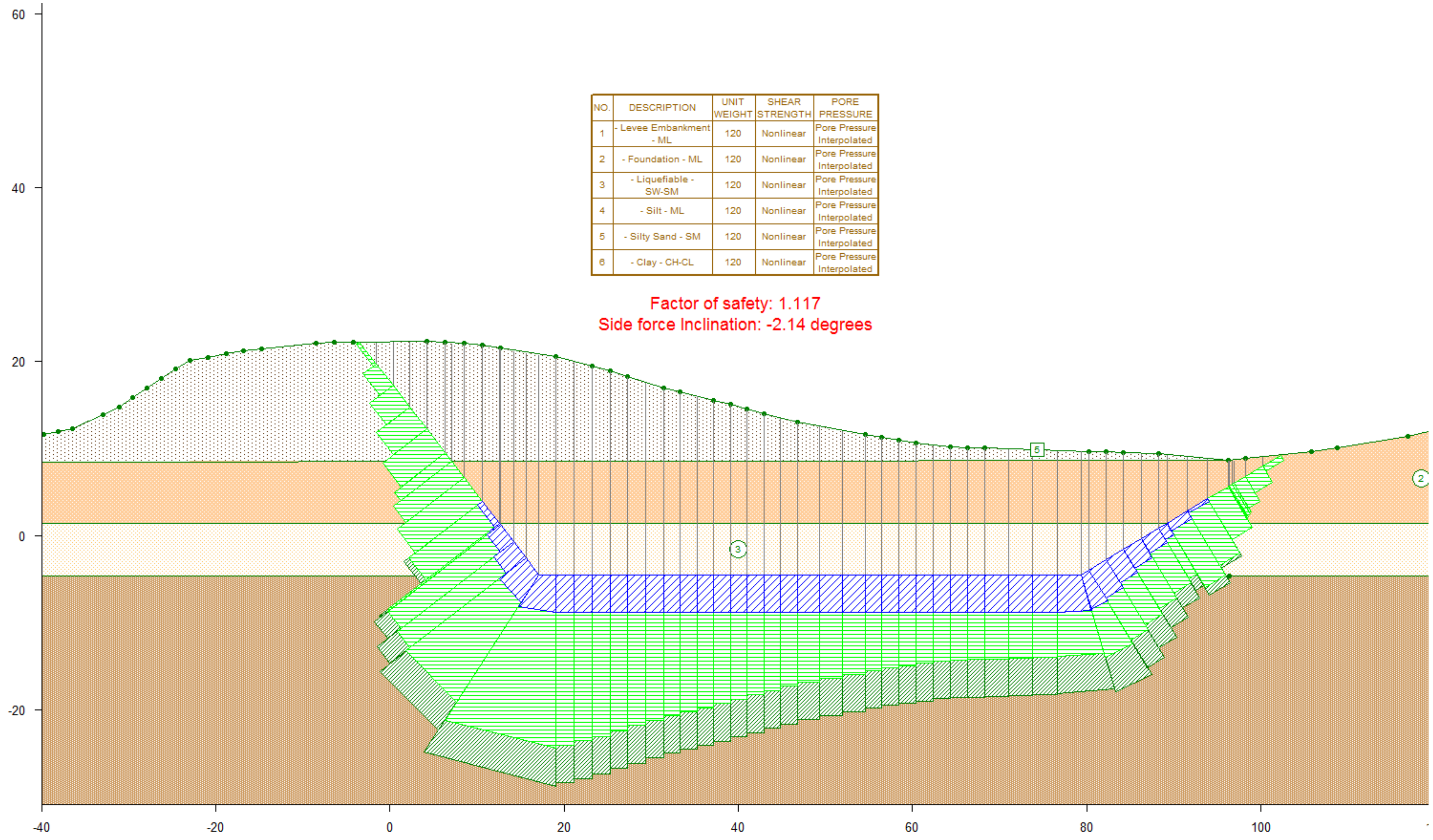
Fig F-19(b). RD 404 Station 1175+01 – Landside – Option 3: Circular (PHI = 3.6 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable - SW-SM	120	Nonlinear	Pore Pressure Interpolated
4	- Silt - ML	120	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	120	Nonlinear	Pore Pressure Interpolated
6	- Clay - CH-CL	120	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.149
Side force Inclination: -2.21 degrees

Fig F-20(a). RD 404 Station 1175+01 – Landside – Option 4: Wedge (Sr = 113 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - ML	120	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable - SW-SM	120	Nonlinear	Pore Pressure Interpolated
4	- Silt - ML	120	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	120	Nonlinear	Pore Pressure Interpolated
6	- Clay - CH-CL	120	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.117
Side force Inclination: -2.14 degrees

Fig F-20(b). RD 404 Station 1175+01 – Landside – Option 4: Wedge (PHI = 3.6 in liquefiable material)

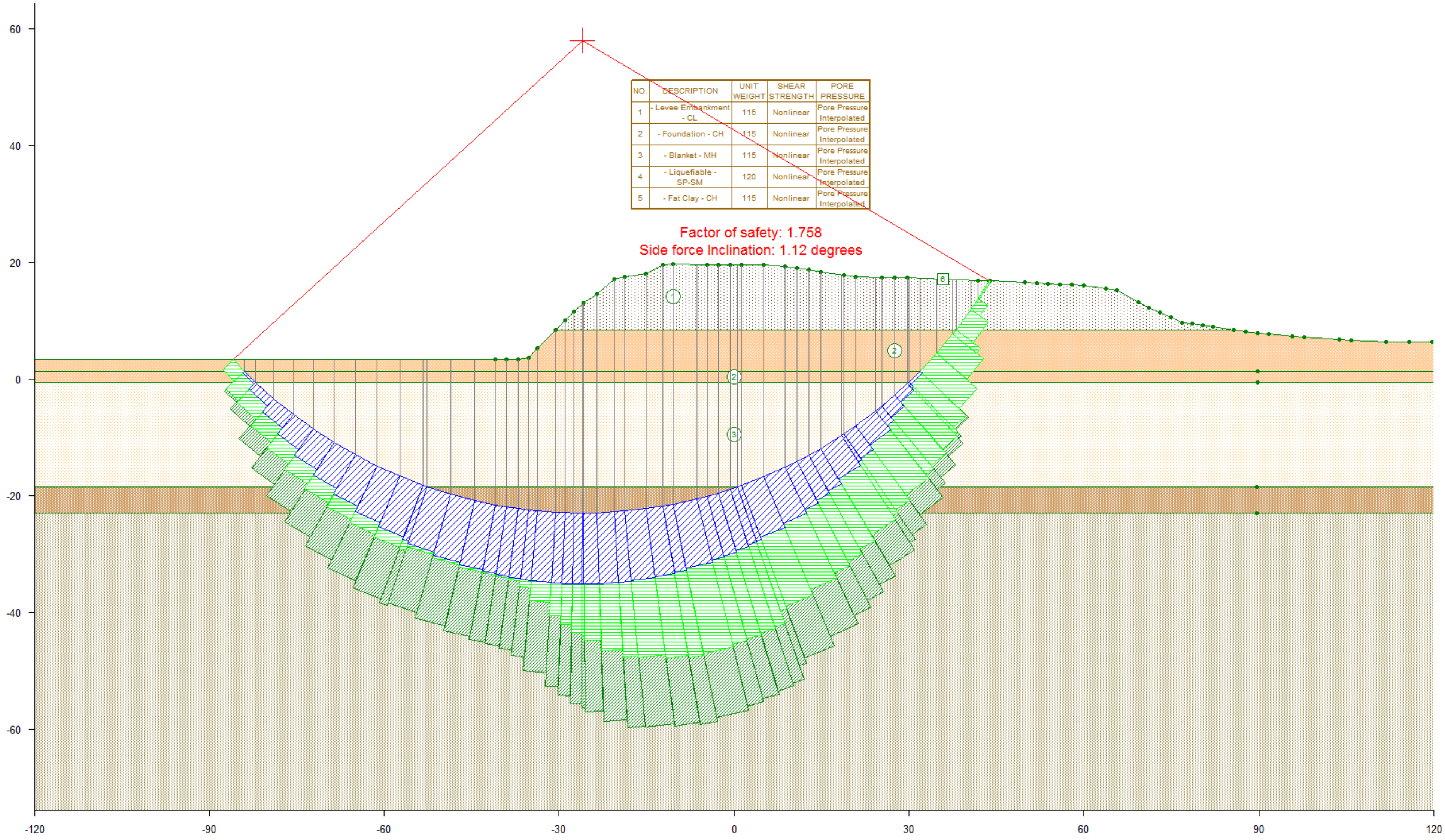
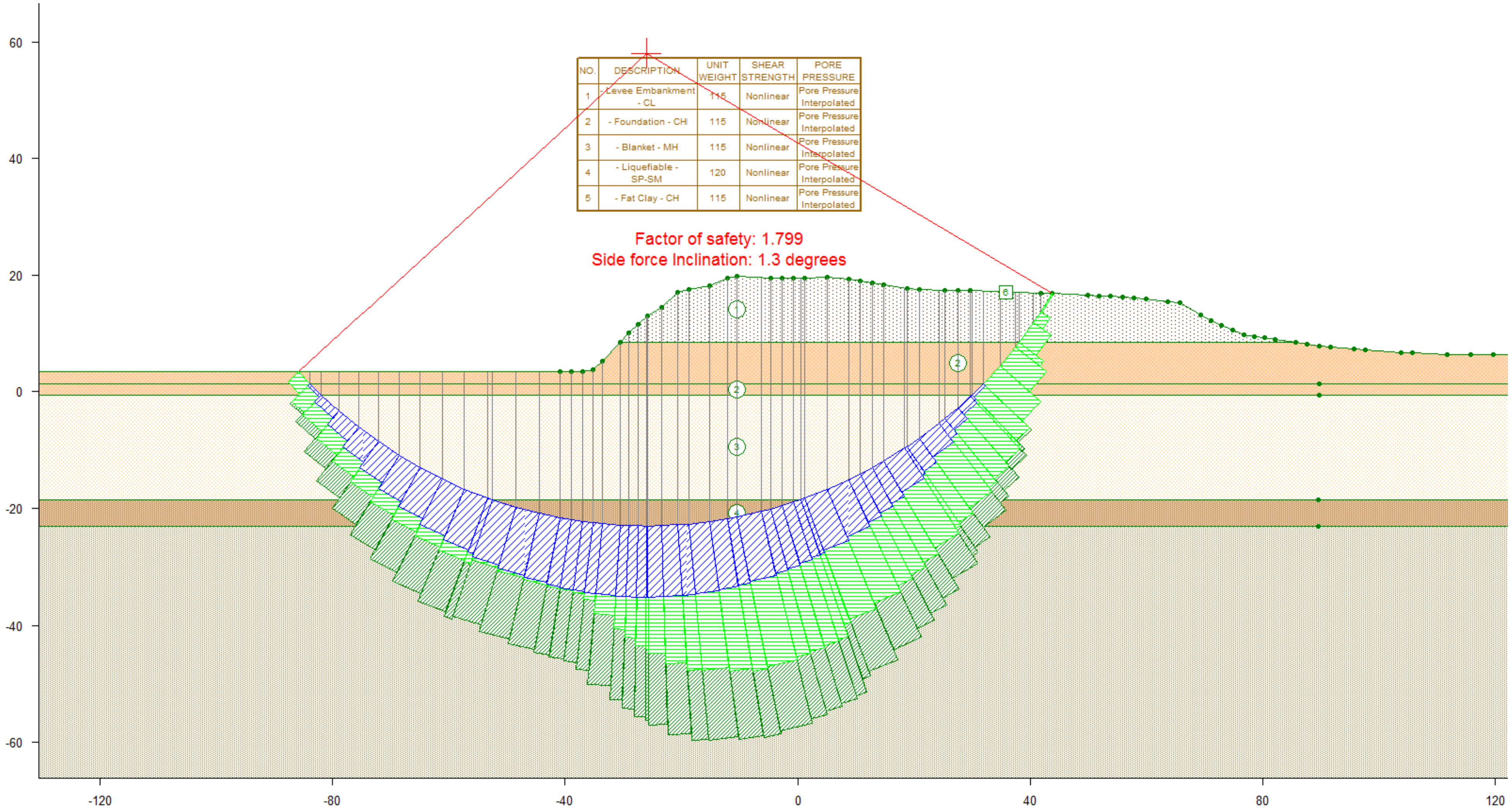
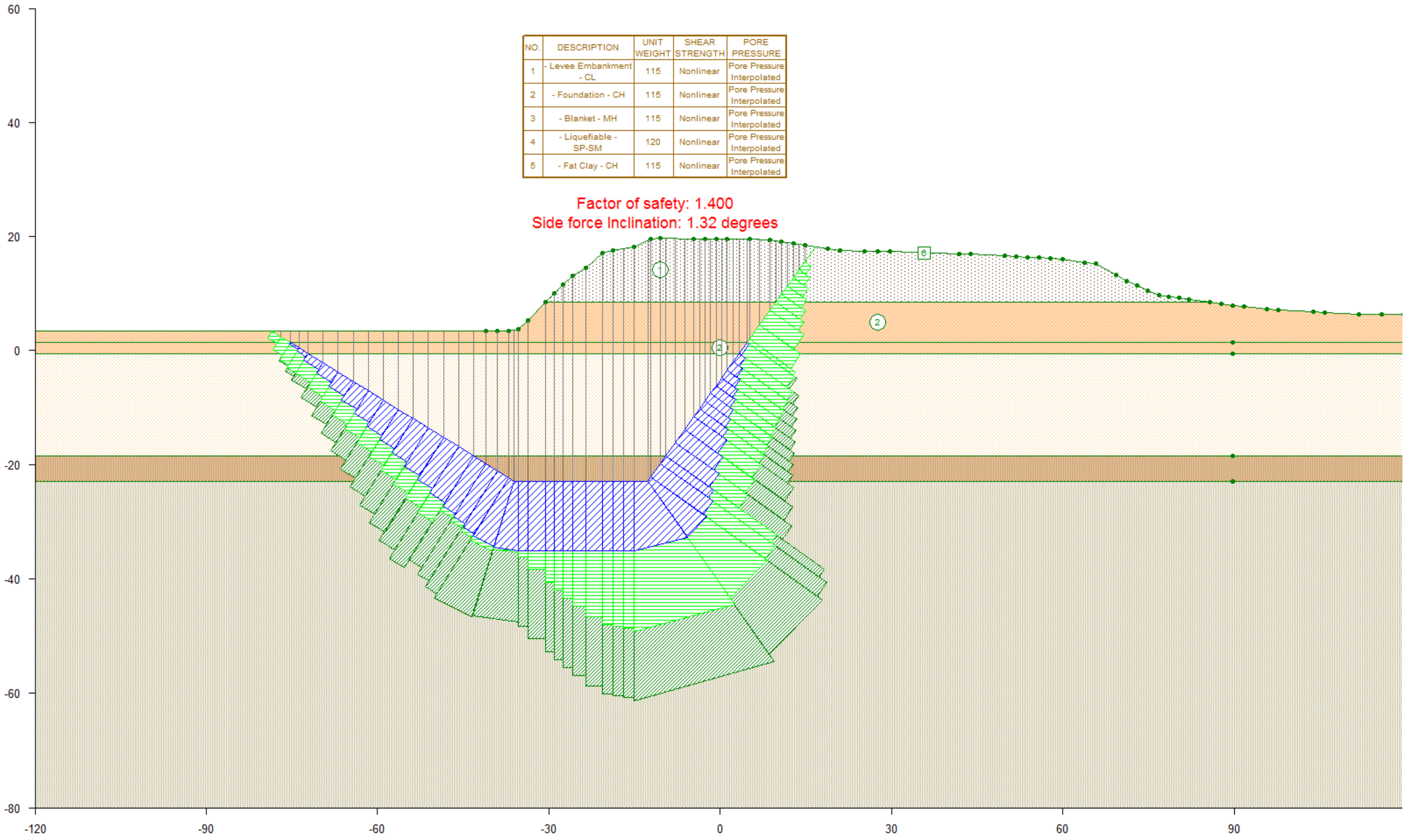


Fig F-21(a). Calaveras River Station 6565+02 – Waterside – Option 1: Circular (Sr = 77 psf in liquefiable material)



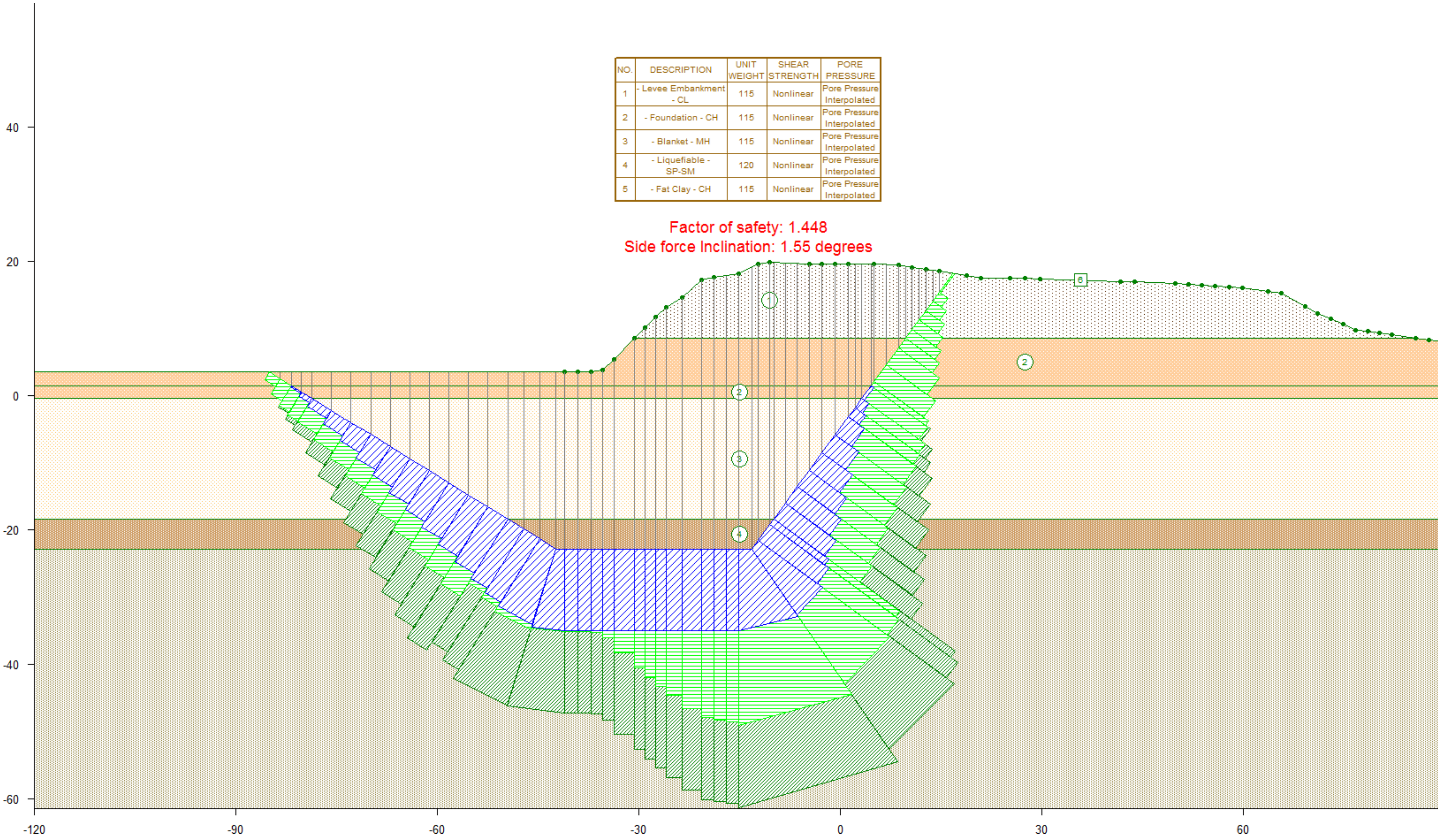
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Foundation - CH	115	Nonlinear	Pore Pressure Interpolated
3	- Blanket - MH	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP-SM	120	Nonlinear	Pore Pressure Interpolated
5	- Fat Clay - CH	115	Nonlinear	Pore Pressure Interpolated

Fig 21(b). Calaveras River Station 6565+02 – Waterside – Option 1: Circular (PHI = 2.6 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Foundation - CH	115	Nonlinear	Pore Pressure Interpolated
3	- Blanket - MH	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP-SM	120	Nonlinear	Pore Pressure Interpolated
5	- Fat Clay - CH	115	Nonlinear	Pore Pressure Interpolated

Fig F-22(a). Calaveras River Station 6565+02 – Waterside – Option 2: Wedges (Sr = 77 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Foundation - CH	115	Nonlinear	Pore Pressure Interpolated
3	- Blanket - MH	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - SP-SM	120	Nonlinear	Pore Pressure Interpolated
5	- Fat Clay - CH	115	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.448
Side force Inclination: 1.55 degrees

Fig 22(b). Calaveras River Station 6565+02 – Waterside – Option 2: Wedges (PHI = 2.6 in liquefiable material)

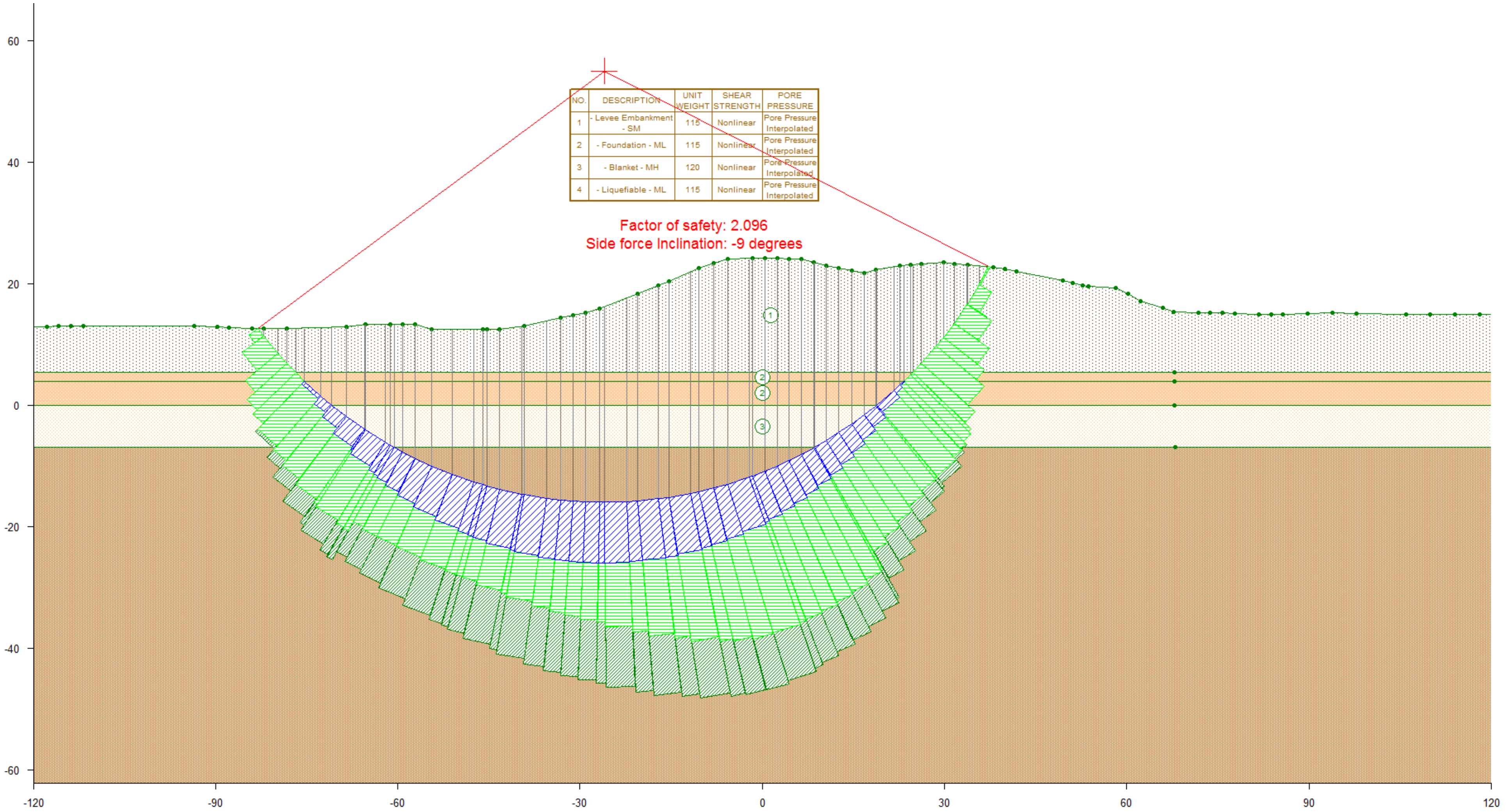
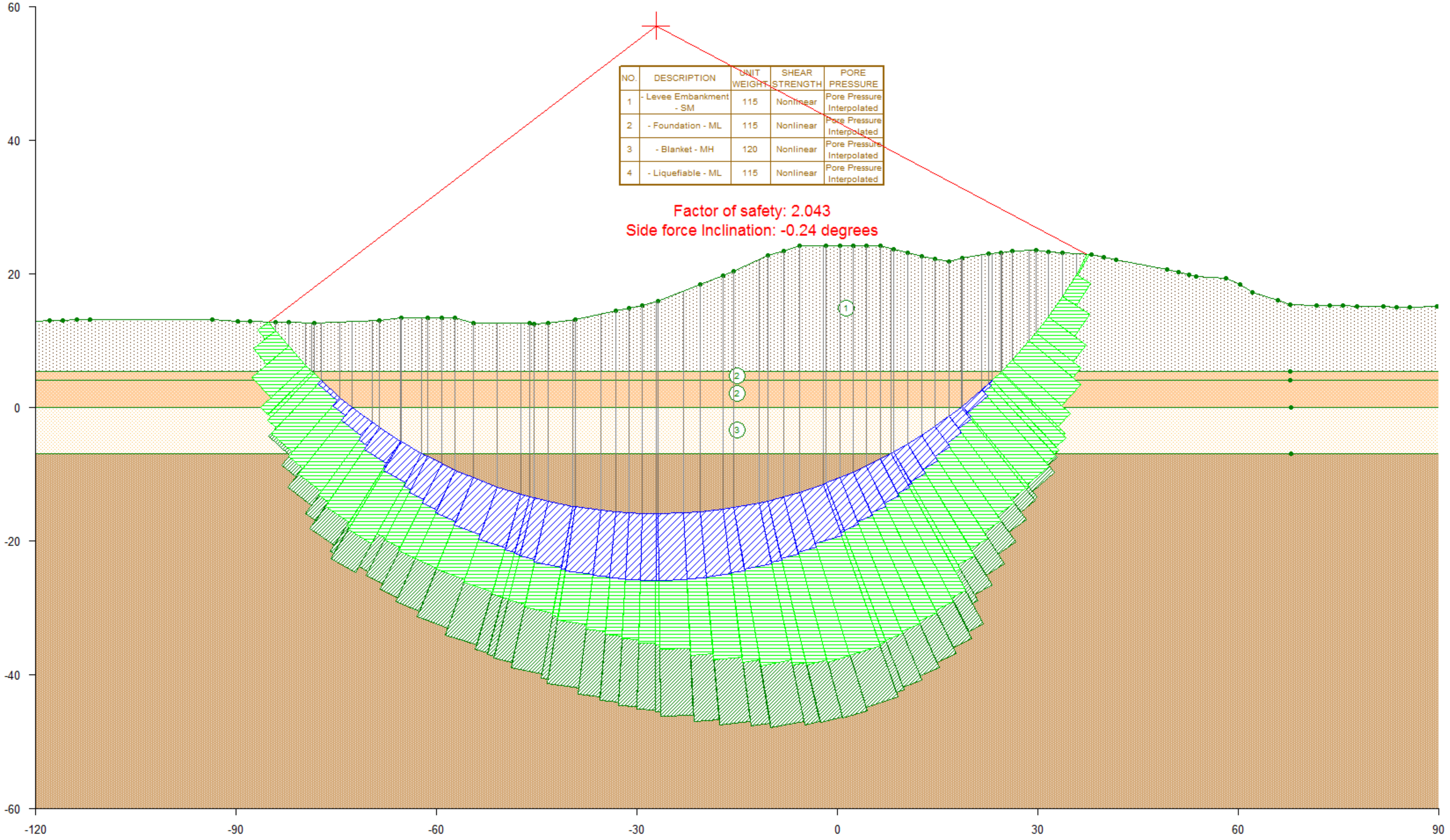
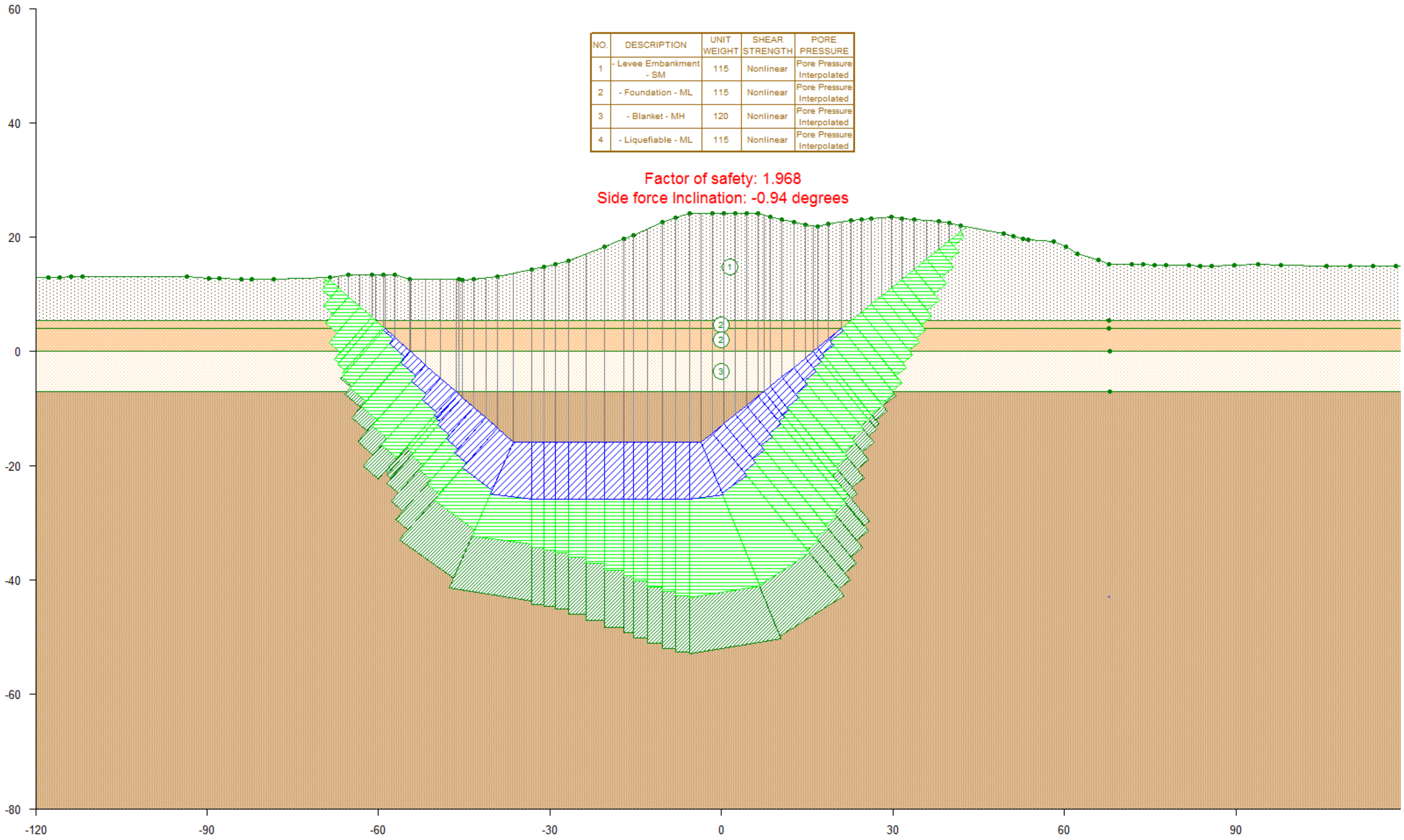


Fig F-23(a). Calaveras River Station 6669+40 – Waterside – Option 1: Circular (Sr = 98 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - SM	115	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	115	Nonlinear	Pore Pressure Interpolated
3	- Blanket - MH	120	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - ML	115	Nonlinear	Pore Pressure Interpolated

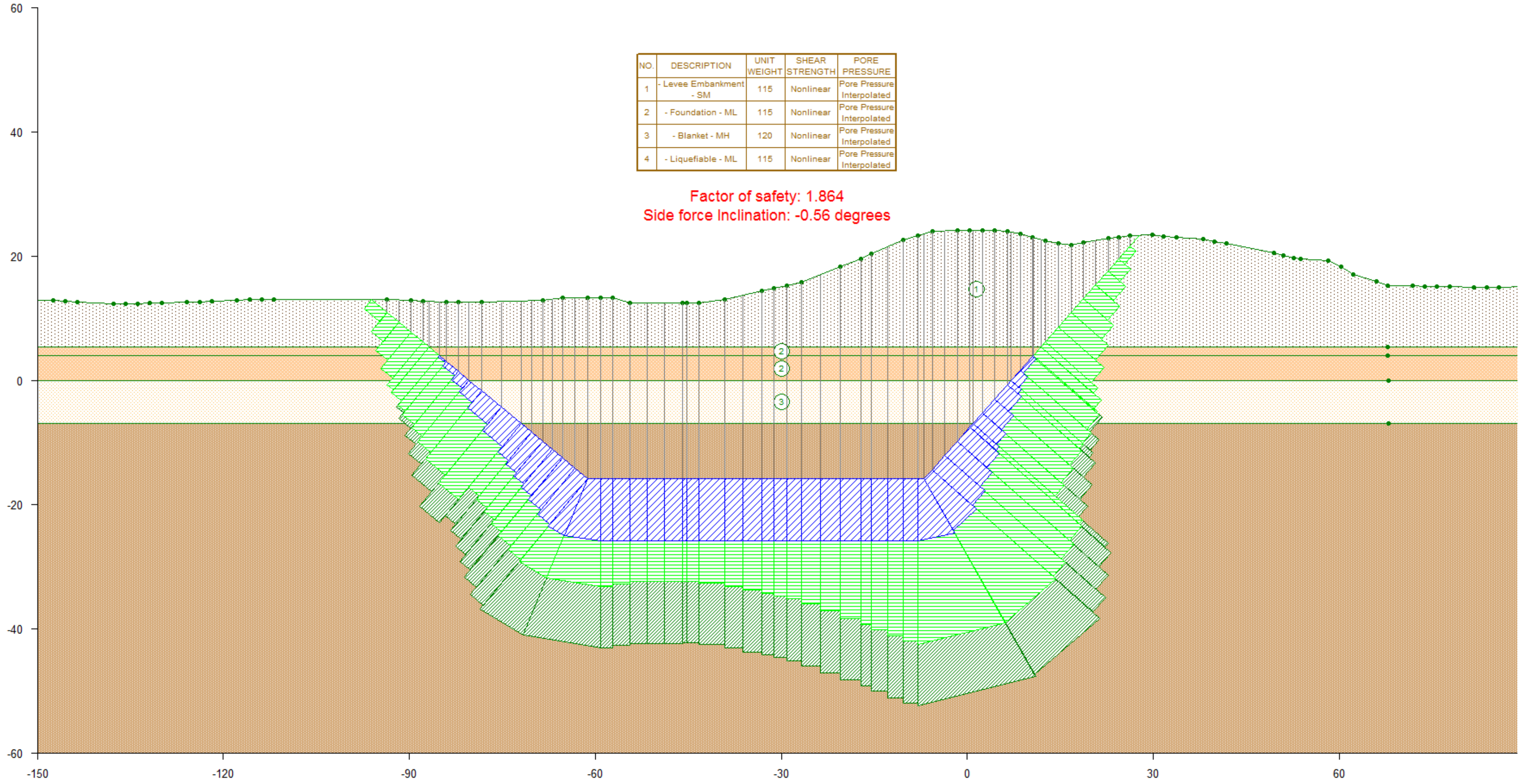
Fig 23(b). Calaveras River Station 6669+40 – Waterside – Option 1: Circular (PHI = 1.7 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - SM	115	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	115	Nonlinear	Pore Pressure Interpolated
3	- Blanket - MH	120	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - ML	115	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.968
Side force Inclination: -0.94 degrees

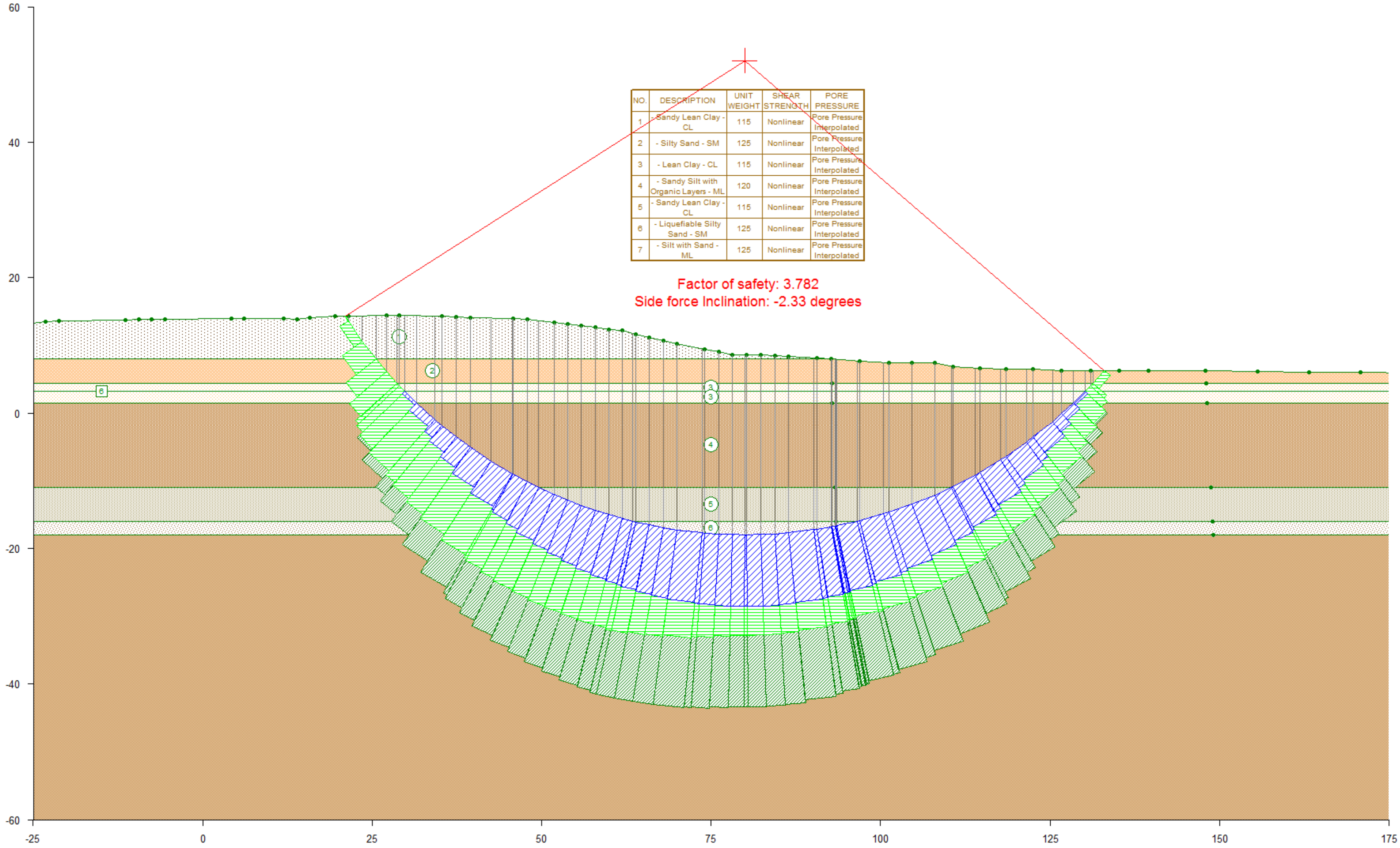
Fig F-24(a). Calaveras River Station 6669+40 – Waterside – Option 2: Wedges (Sr = 98 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - SM	115	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	115	Nonlinear	Pore Pressure Interpolated
3	- Blanket - MH	120	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable - ML	115	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.864
Side force Inclination: -0.56 degrees

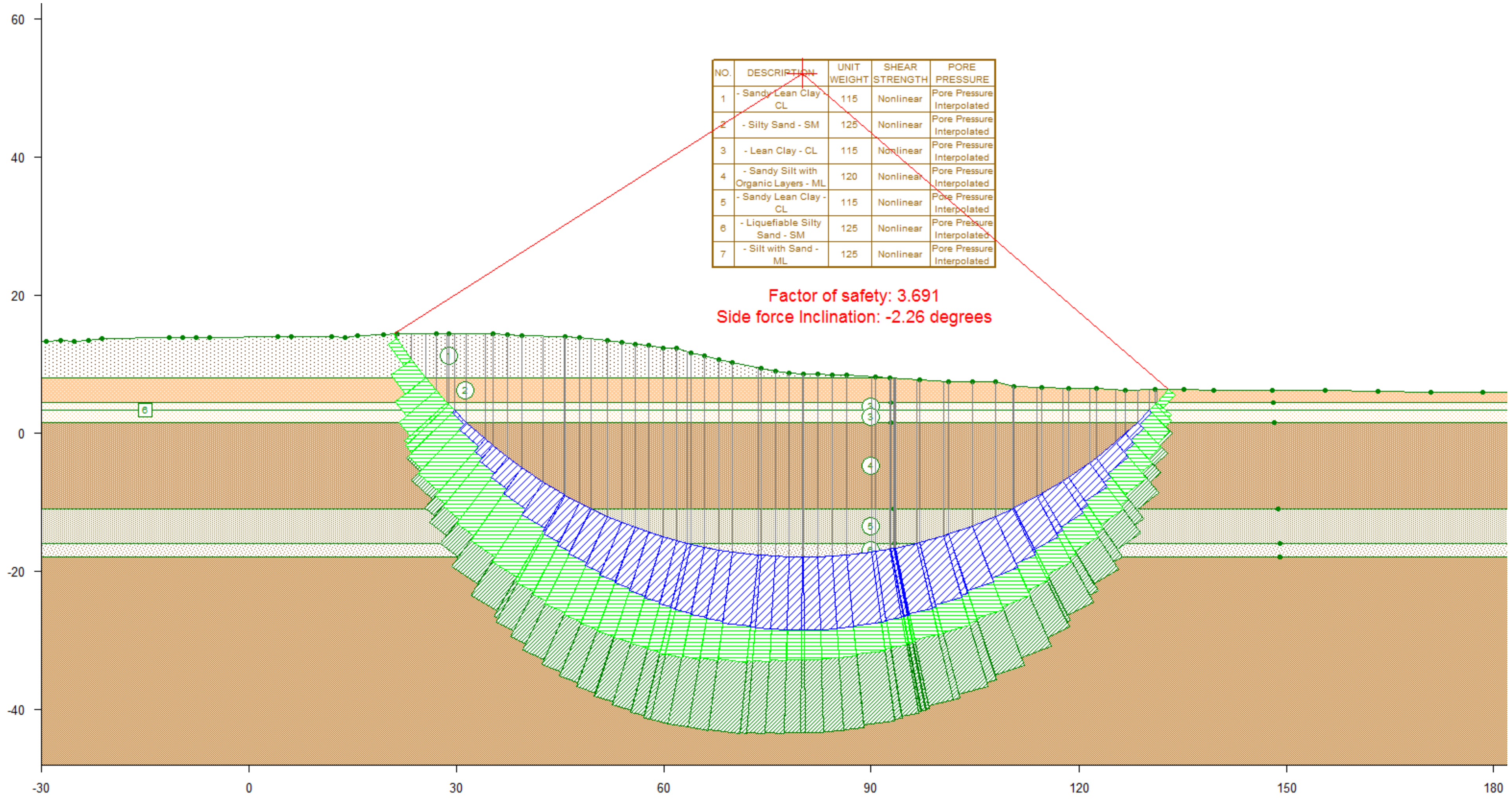
Fig 24(b). Calaveras River Station 6669+40 – Waterside – Option 2: Wedges (PHI = 1.7 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Sandy Silt with Organic Layers - ML	120	Nonlinear	Pore Pressure Interpolated
5	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
7	- Silt with Sand - ML	125	Nonlinear	Pore Pressure Interpolated

Factor of safety: 3.782
Side force Inclination: -2.33 degrees

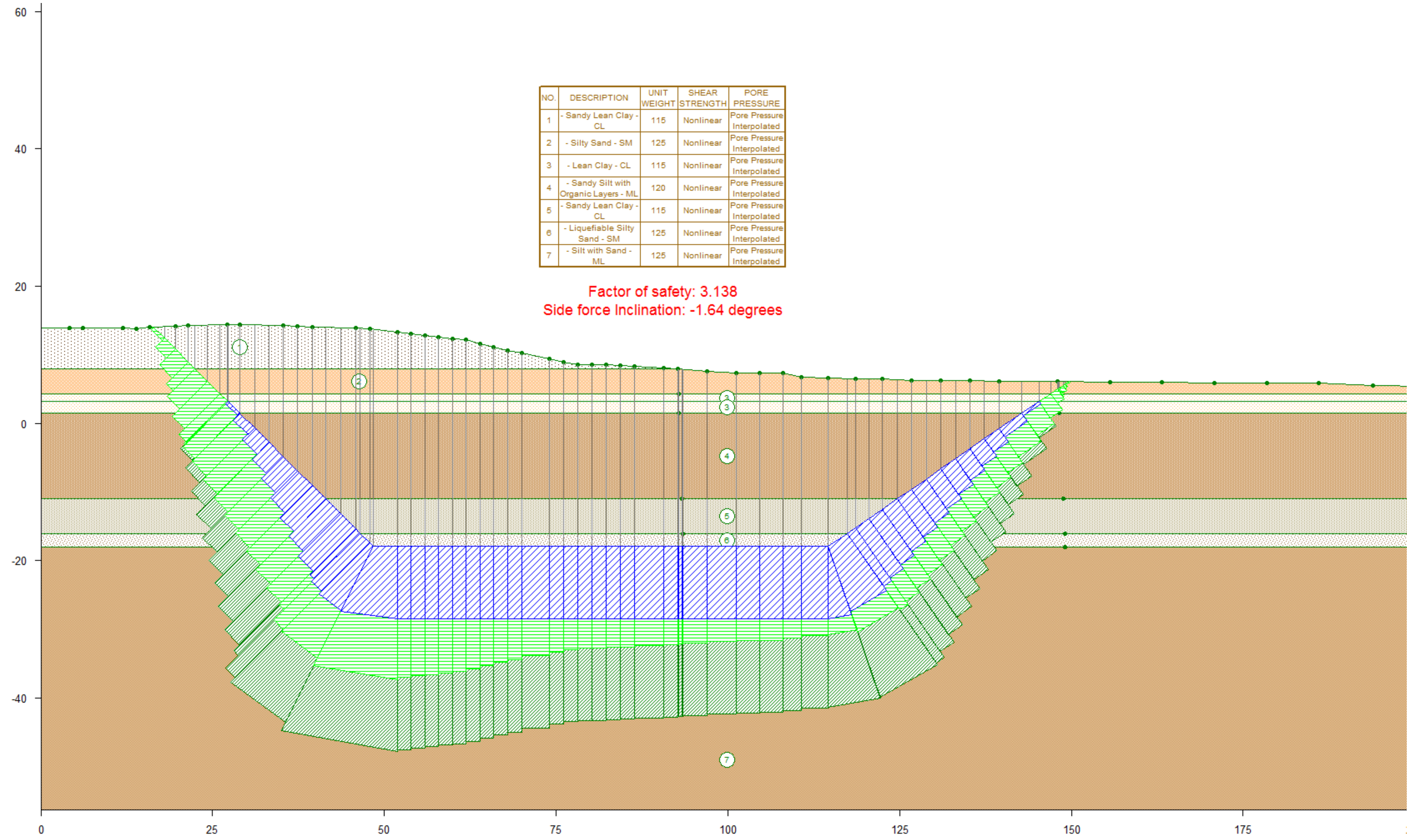
Fig F-25(a). Brookside Station 117+51 – Landside – Option 1: Circular (Sr = 189 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Sandy Silt with Organic Layers - ML	120	Nonlinear	Pore Pressure Interpolated
5	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
7	- Silt with Sand - ML	125	Nonlinear	Pore Pressure Interpolated

Factor of safety: 3.691
Side force Inclination: -2.26 degrees

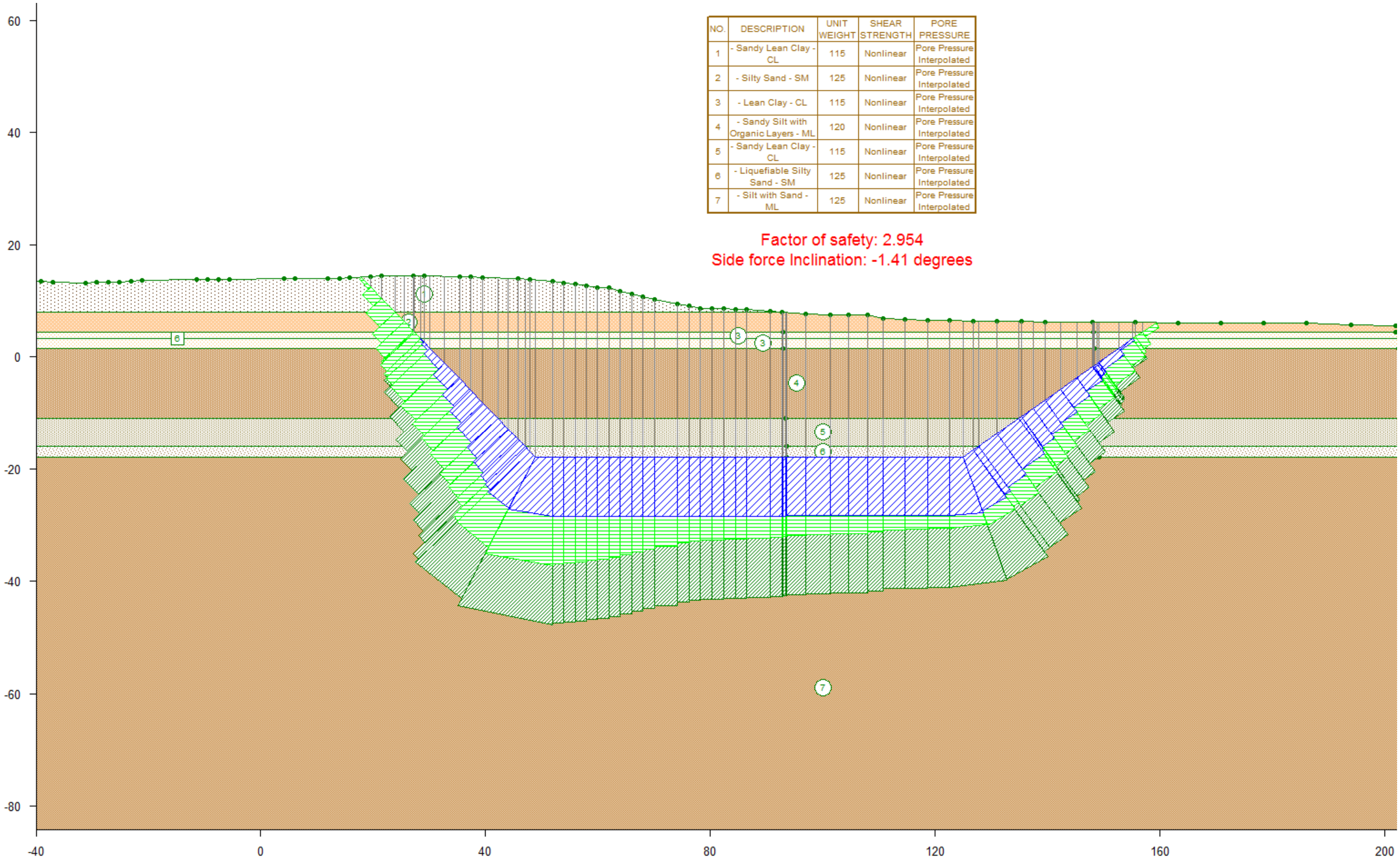
Fig 25(b). Brookside Station 117+51 – Landside – Option 1: Circular (PHI = 4.3 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Sandy Silt with Organic Layers - ML	120	Nonlinear	Pore Pressure Interpolated
5	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
7	- Silt with Sand - ML	125	Nonlinear	Pore Pressure Interpolated

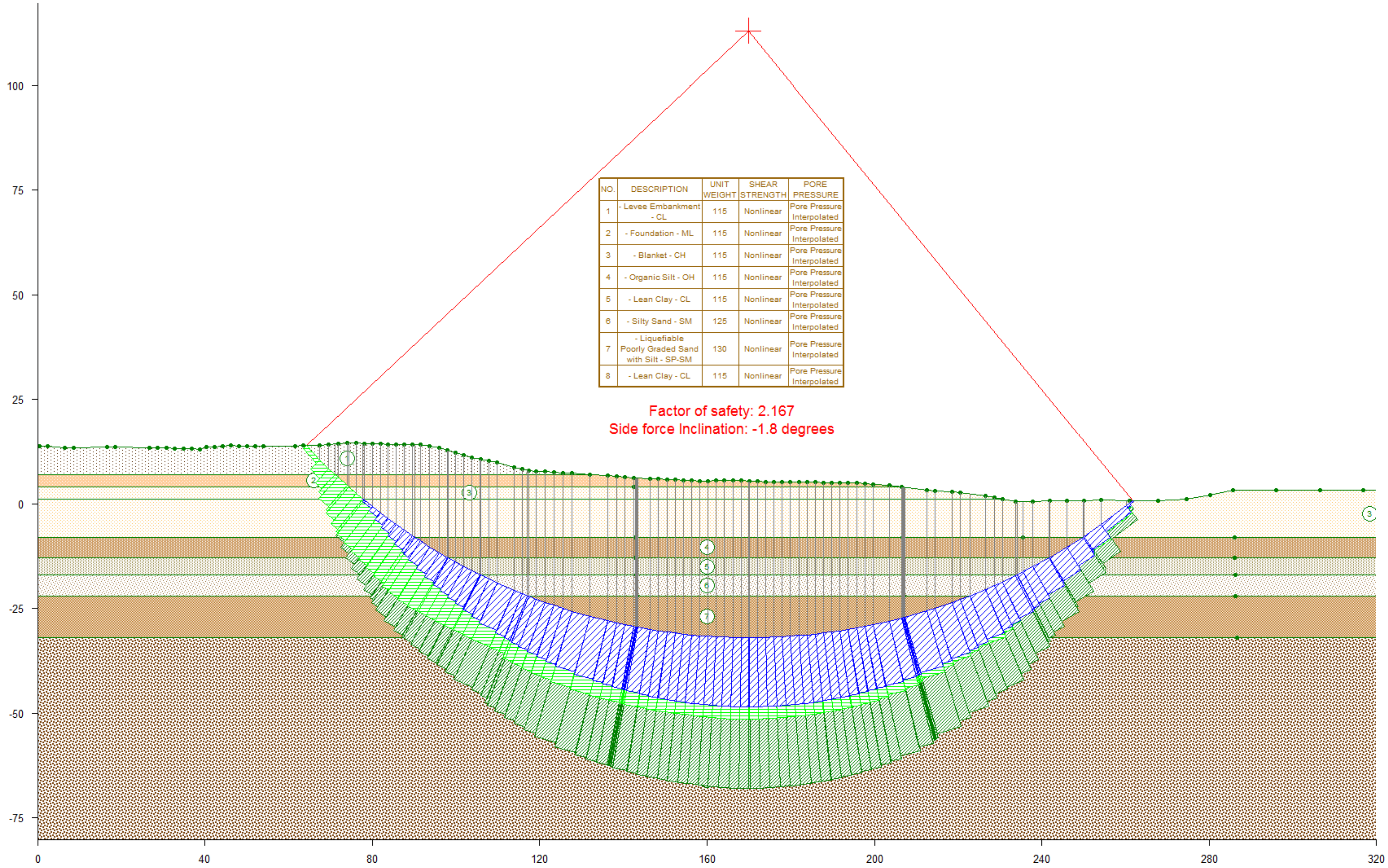
Factor of safety: 3.138
Side force Inclination: -1.64 degrees

Fig F-26(a). Brookside Station 117+51 – Landside – Option 2: Wedges (Sr = 189 psf in liquefiable material)



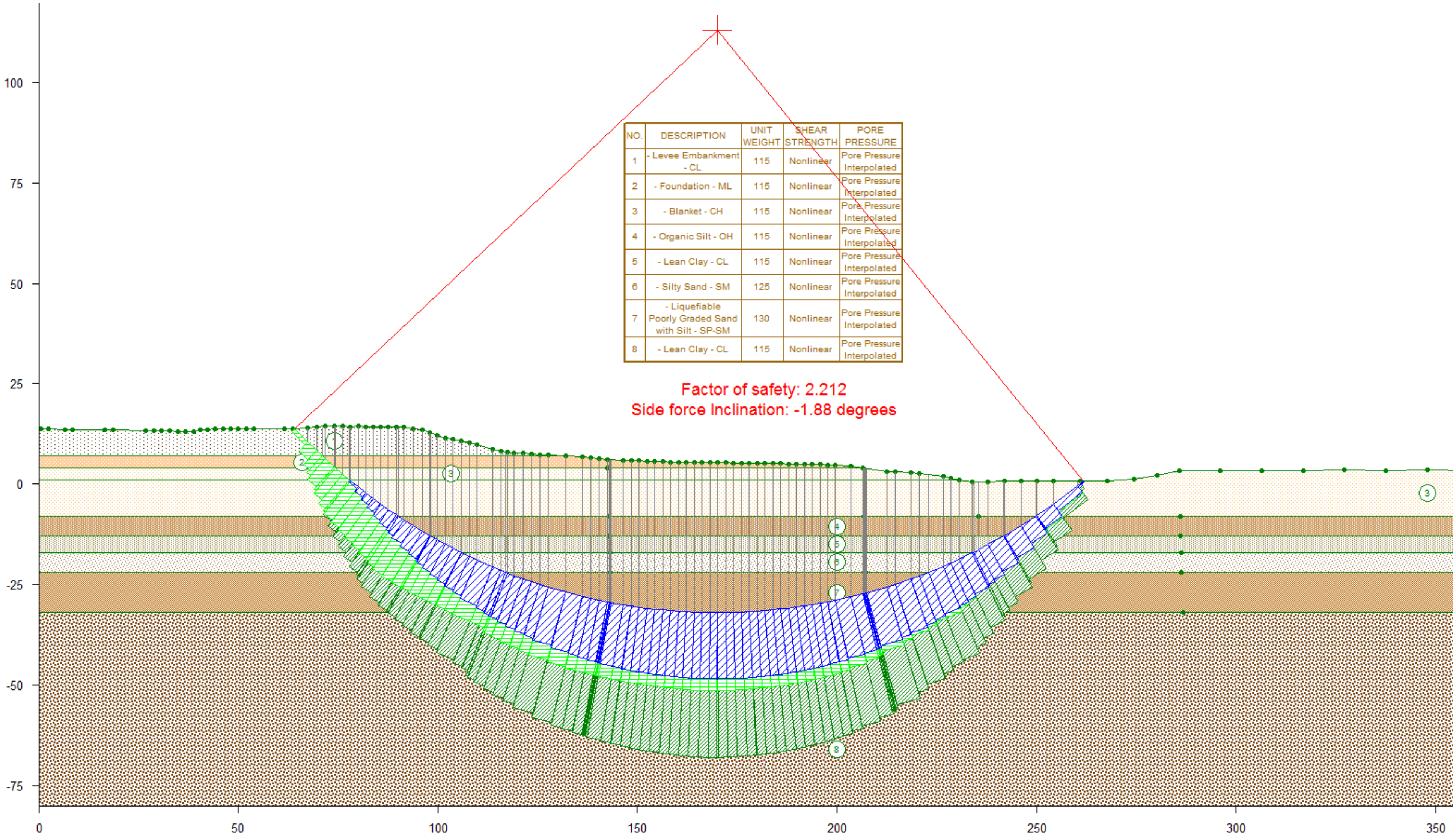
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Sandy Silt with Organic Layers - ML	120	Nonlinear	Pore Pressure Interpolated
5	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
6	- Liquefiable Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
7	- Silt with Sand - ML	125	Nonlinear	Pore Pressure Interpolated

Fig 26(b). Brookside Station 117+51 – Landside – Option 2: Wedges (PHI = 4.3 in liquefiable material)



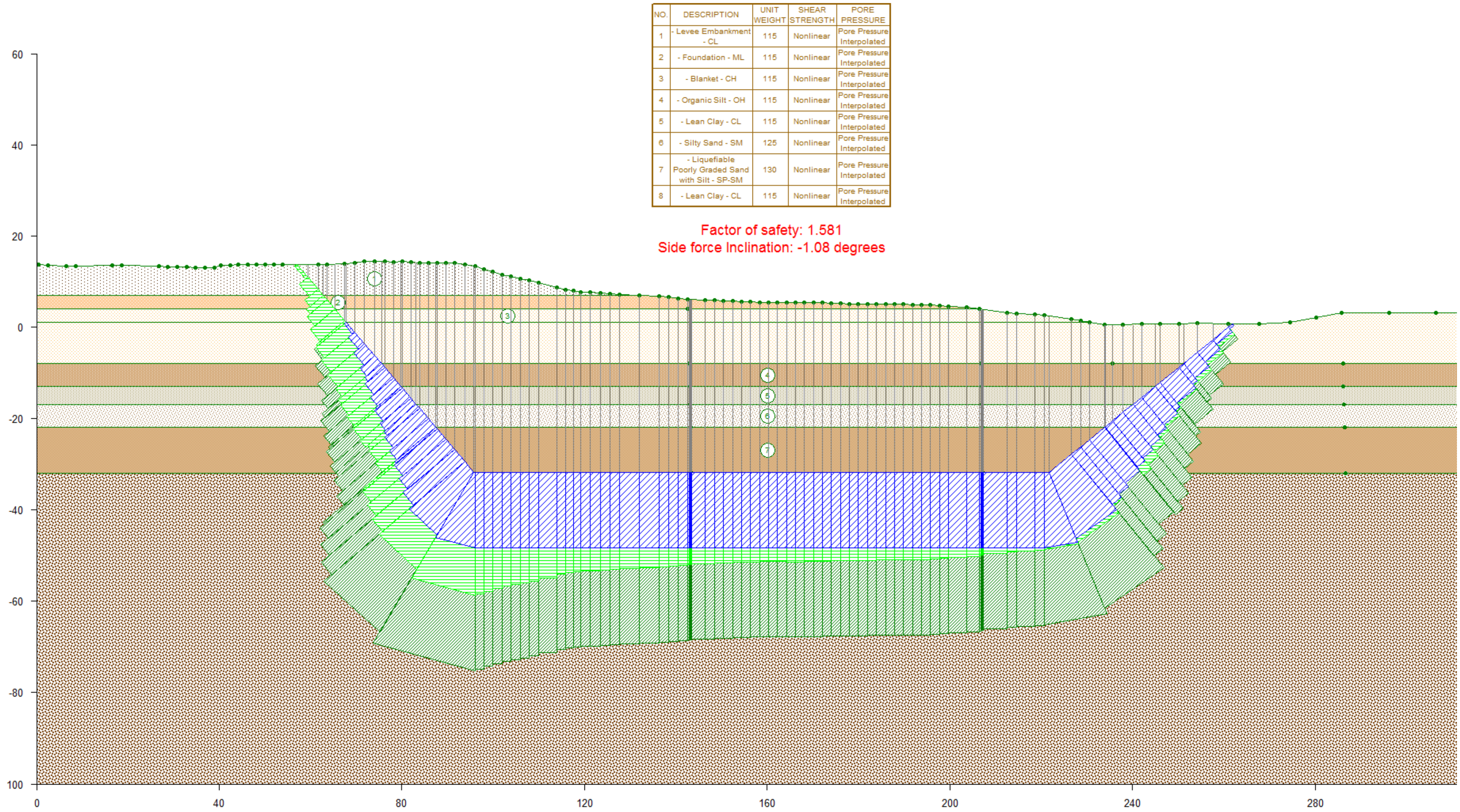
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	115	Nonlinear	Pore Pressure Interpolated
3	- Blanket - CH	115	Nonlinear	Pore Pressure Interpolated
4	- Organic Silt - OH	115	Nonlinear	Pore Pressure Interpolated
5	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
6	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
8	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated

Fig F-27(a). Brookside Station 118+02 – Landside – Option 1: Circular (Sr = 151 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	115	Nonlinear	Pore Pressure Interpolated
3	- Blanket - CH	115	Nonlinear	Pore Pressure Interpolated
4	- Organic Silt - OH	115	Nonlinear	Pore Pressure Interpolated
5	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
6	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
8	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated

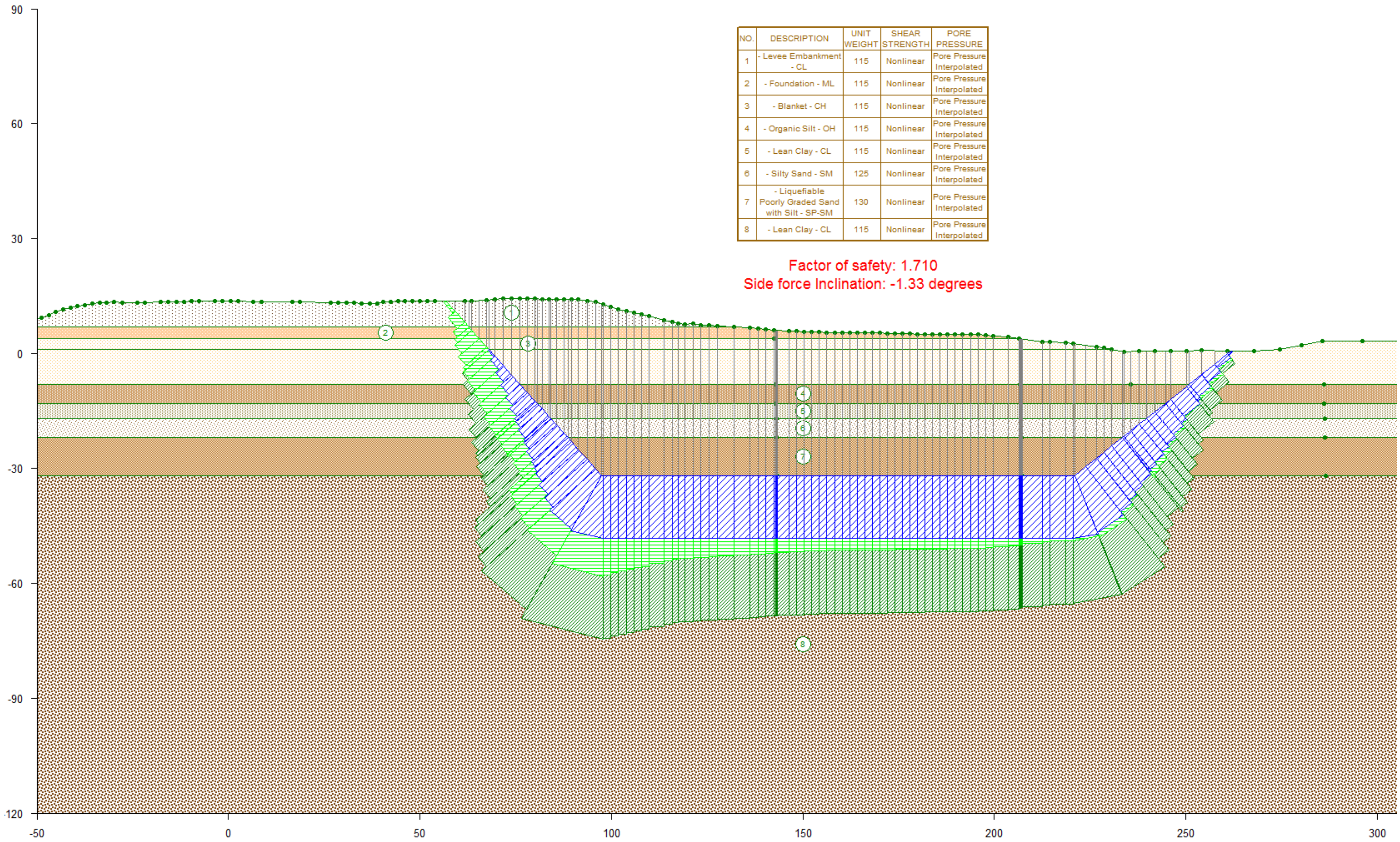
Fig 27(b). Brookside Station 118+02 – Landside – Option 1: Circular (PHI = 4.3 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	115	Nonlinear	Pore Pressure Interpolated
3	- Blanket - CH	115	Nonlinear	Pore Pressure Interpolated
4	- Organic Silt - OH	115	Nonlinear	Pore Pressure Interpolated
5	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
6	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
8	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.581
Side force Inclination: -1.08 degrees

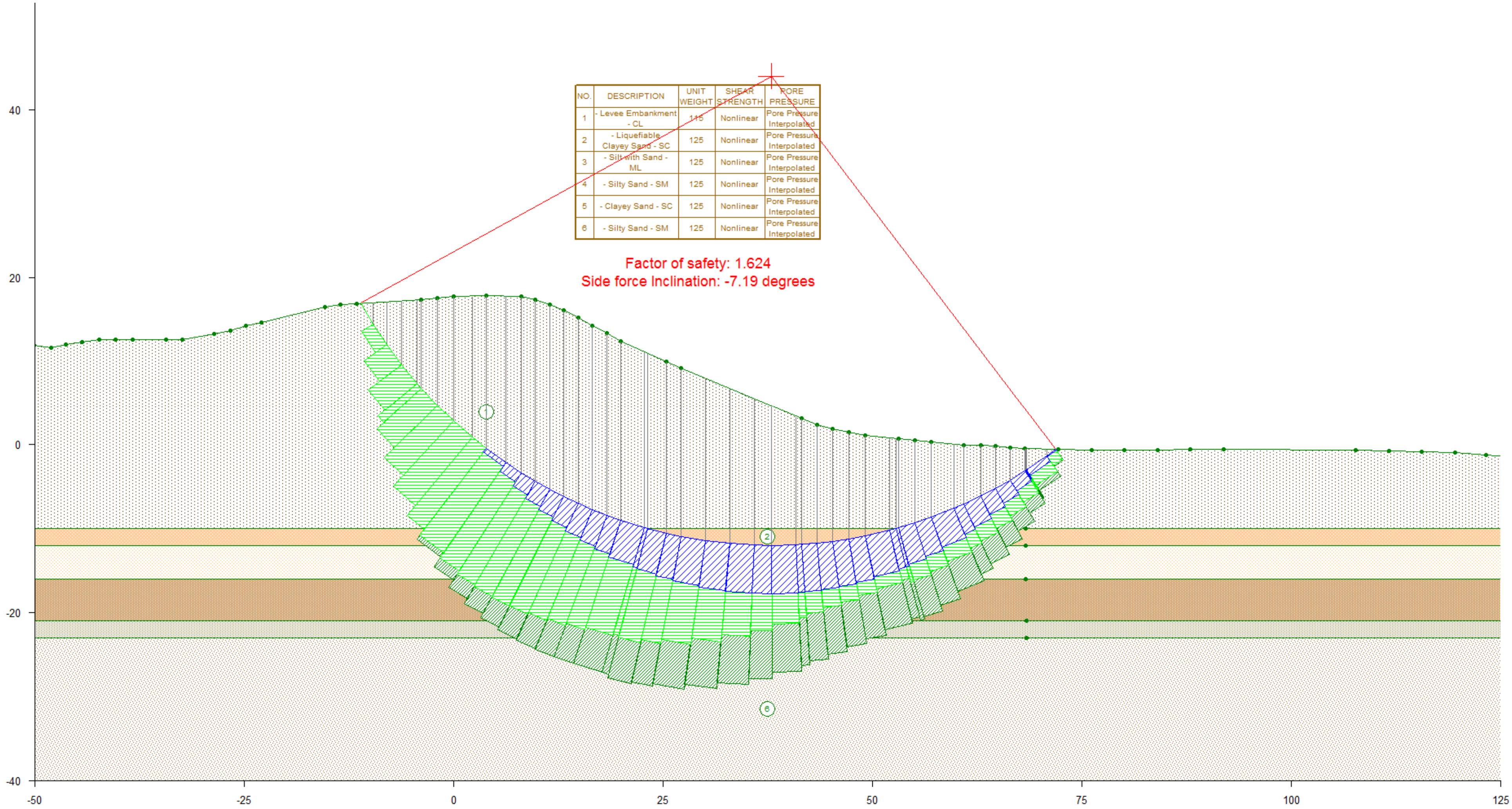
Fig F-28(a). Brookside Station 118+02 – Landside – Option 2: Wedges (Sr = 151 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Foundation - ML	115	Nonlinear	Pore Pressure Interpolated
3	- Blanket - CH	115	Nonlinear	Pore Pressure Interpolated
4	- Organic Silt - OH	115	Nonlinear	Pore Pressure Interpolated
5	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
6	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
8	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.710
Side force Inclination: -1.33 degrees

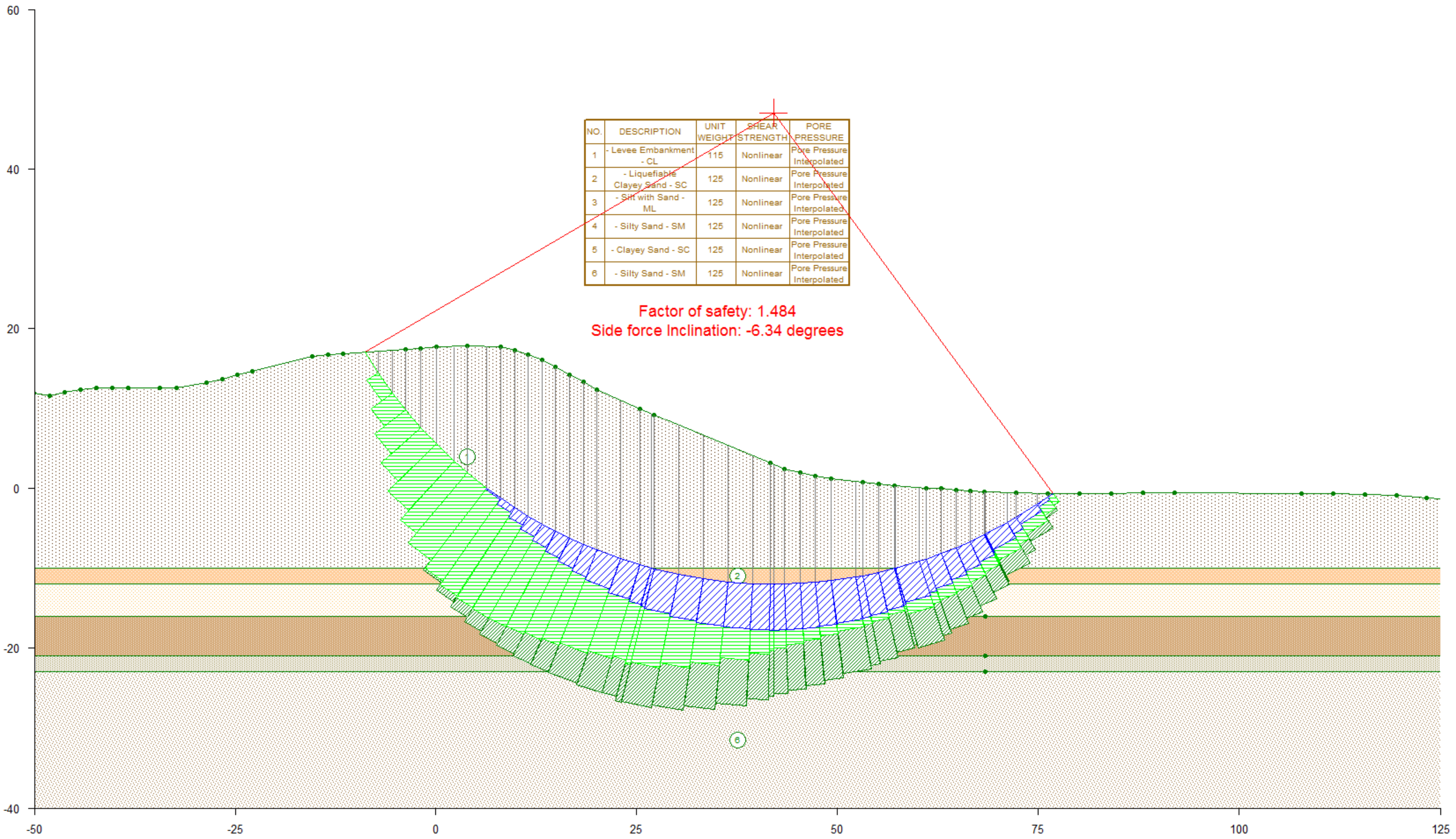
Fig 28(b). Brookside Station 118+02 – Landside – Option 2: Wedges (PHI = 4.3 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Liquefiable Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
3	- Silt with Sand - ML	125	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
6	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.624
Side force Inclination: -7.19 degrees

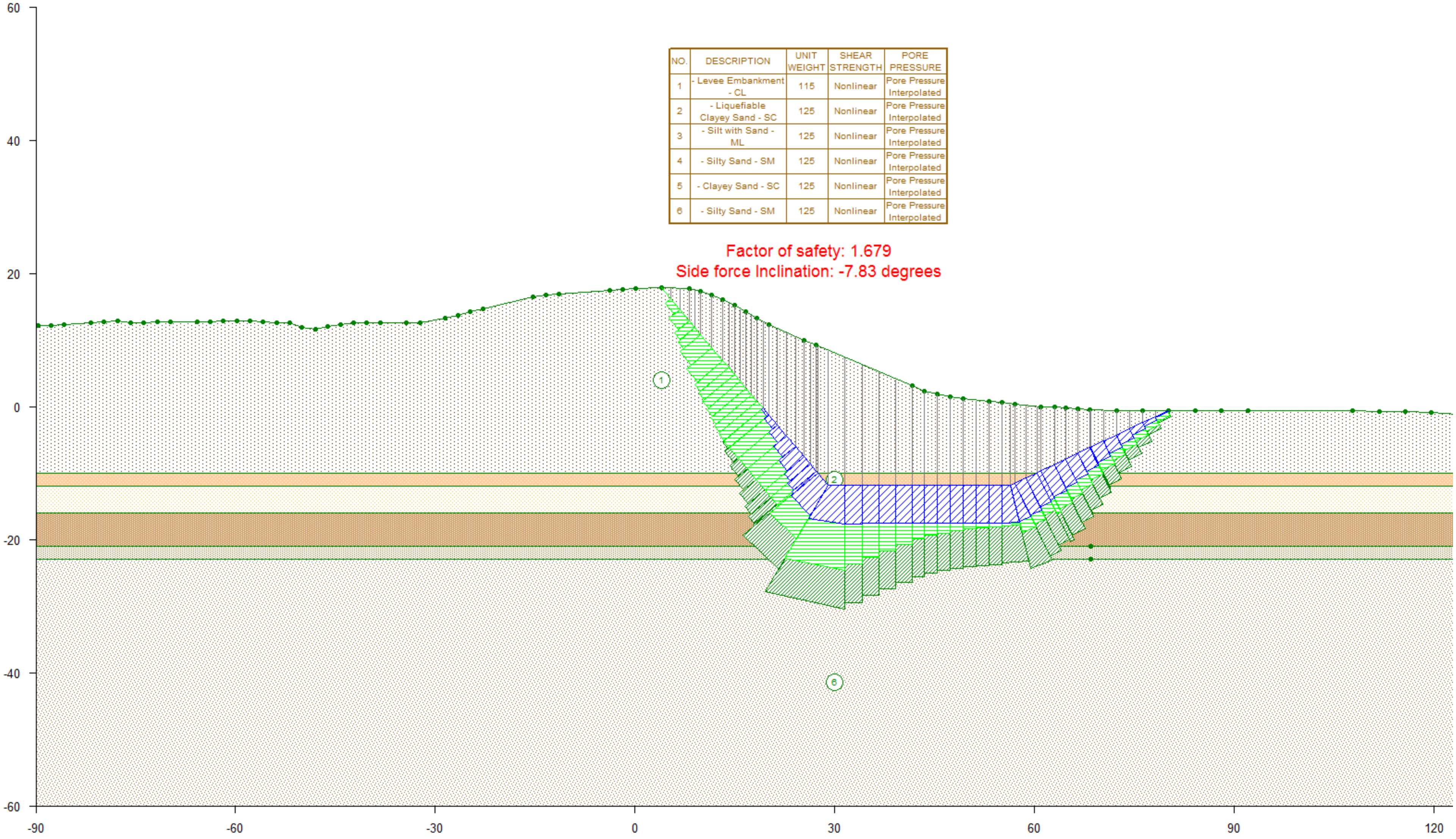
Fig F-29(a). Brookside Station 133+82 – Landside – Option 1: Circular (Sr = 242 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Liquefiable Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
3	- Silt with Sand - ML	125	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
6	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated

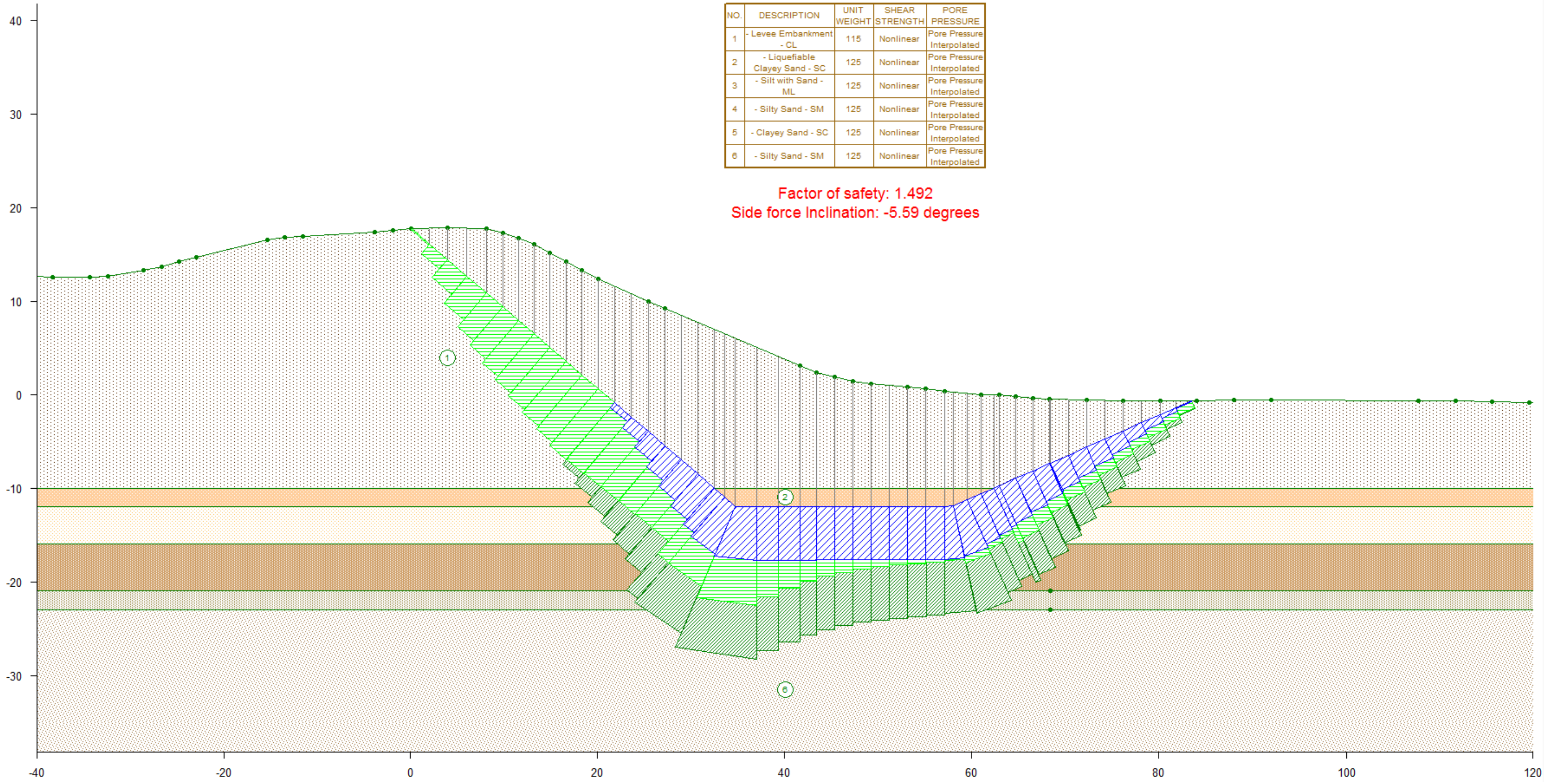
Factor of safety: 1.484
Side force Inclination: -6.34 degrees

Fig 29(b). Brookside Station 133+82 – Landside – Option 1: Circular (PHI = 5.1 in liquefiable material)



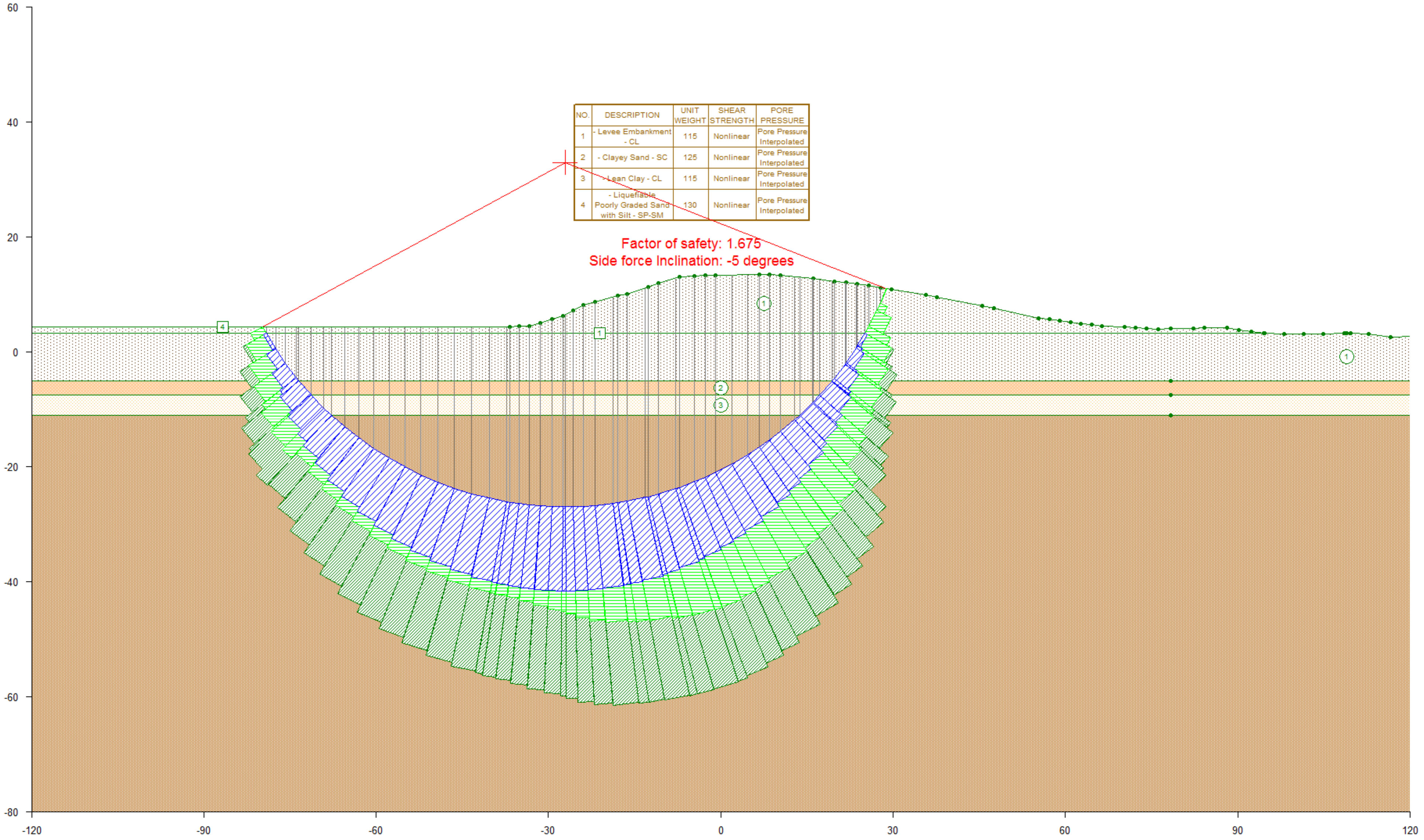
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Liquefiable Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
3	- Silt with Sand - ML	125	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
6	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated

Fig F-30(a). Brookside Station 133+82 – Landside – Option 2: Wedges (Sr = 242 psf in liquefiable material)



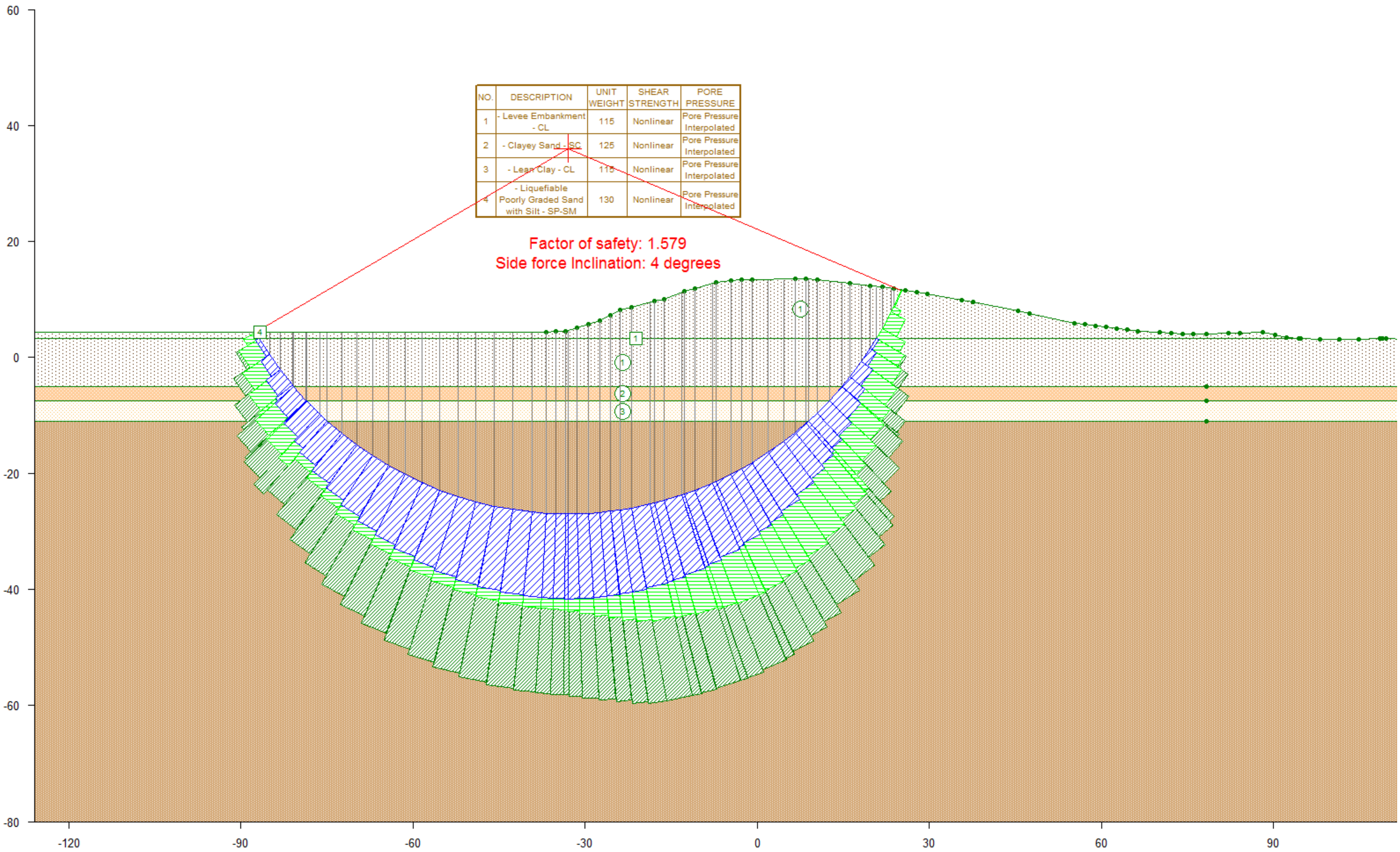
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Liquefiable Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
3	- Silt with Sand - ML	125	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
6	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated

Fig 30(b). Brookside Station 133+82 – Landside – Option 2: Wedges (PHI = 5.1 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
3	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

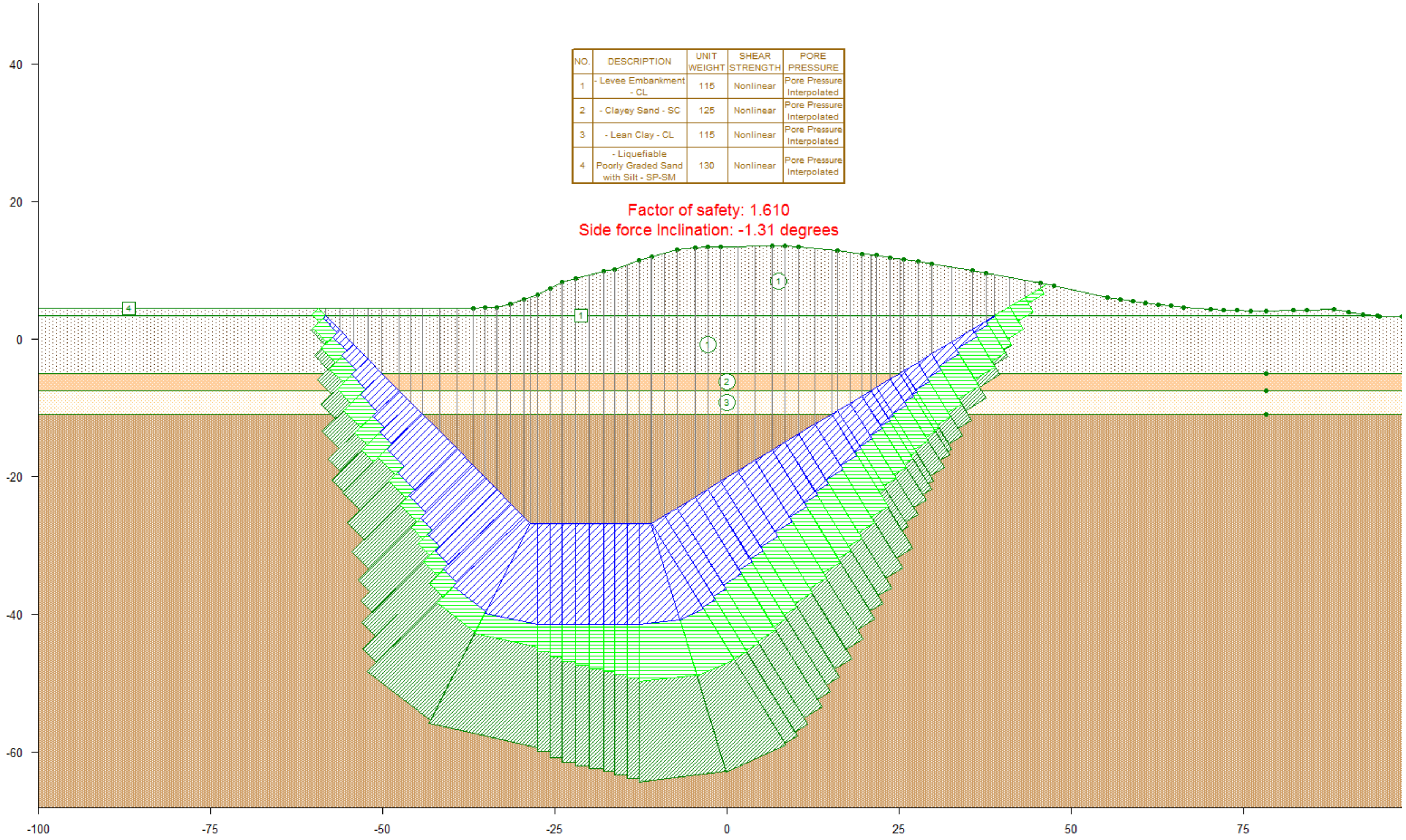
Fig F-31(a). Lincoln Village Station 43+57 – Waterside – Option 1: Circular (Sr = 201 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
3	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.579
Side force Inclination: 4 degrees

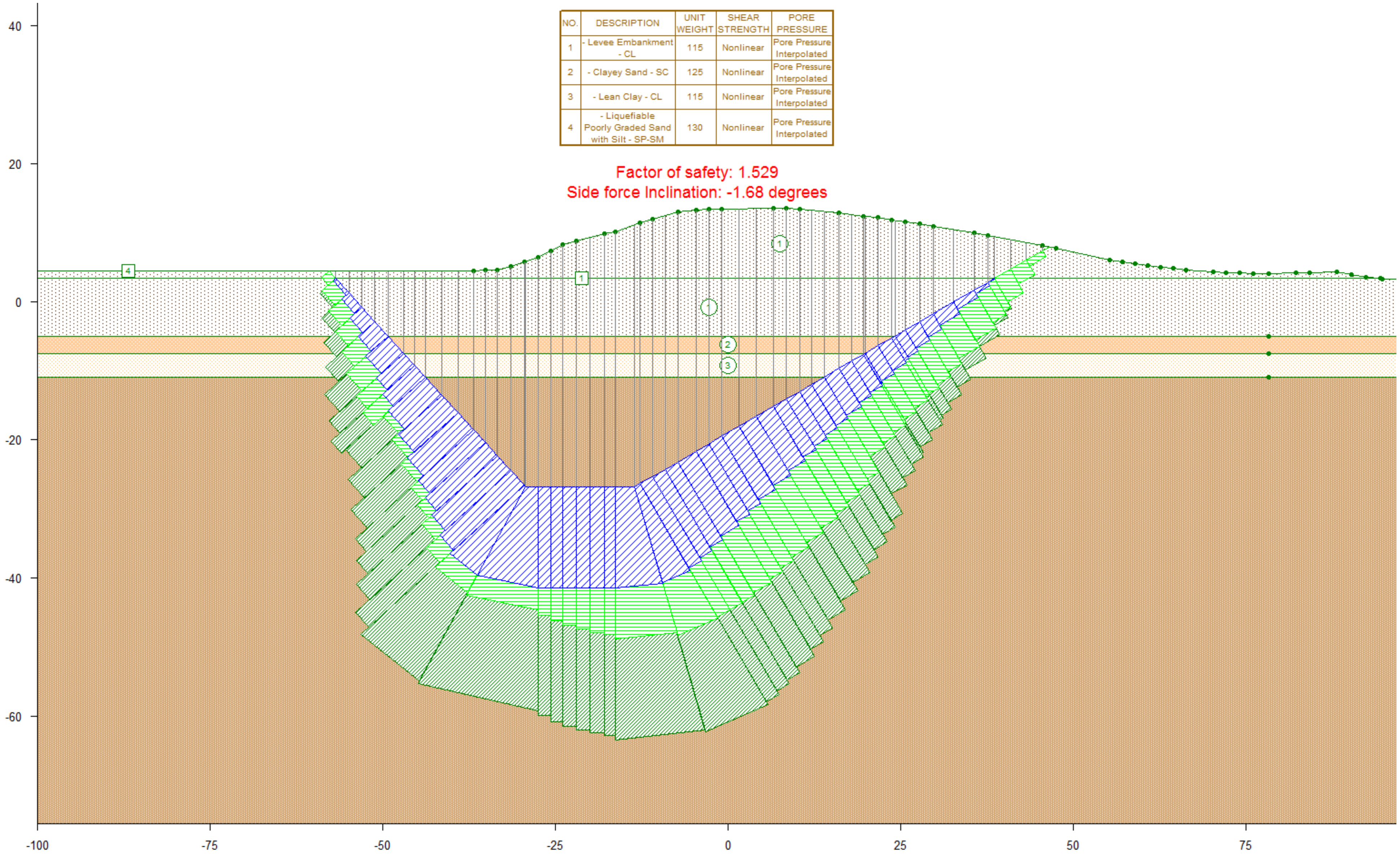
Fig 31(b). Lincoln Village Station 43+57 – Waterside – Option 1: Circular (PHI = 4.7 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
3	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

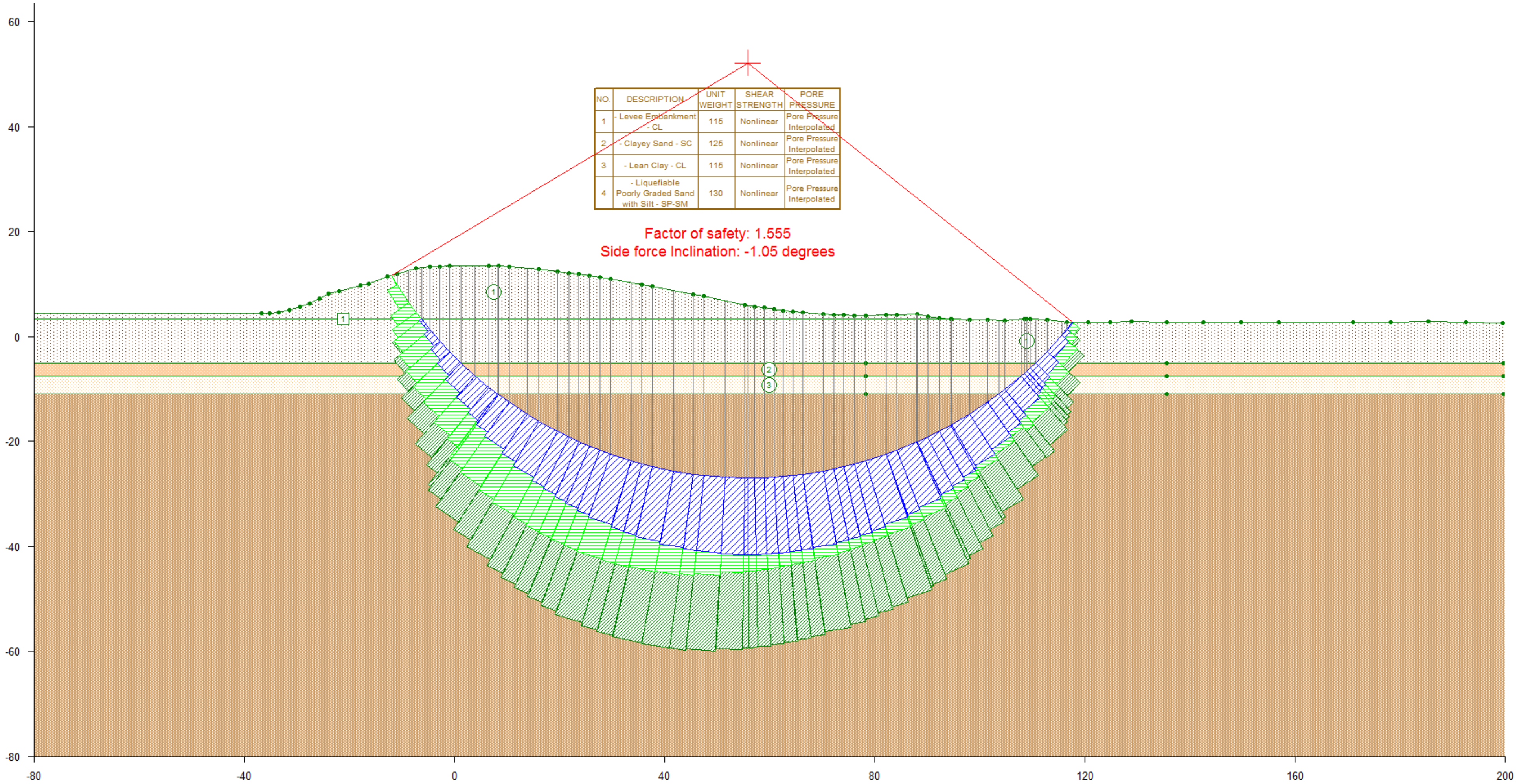
Factor of safety: 1.610
Side force Inclination: -1.31 degrees

Fig F-32(a). Lincoln Village Station 43+57 – Waterside – Option 2: Wedge (Sr = 201 psf in liquefiable material)



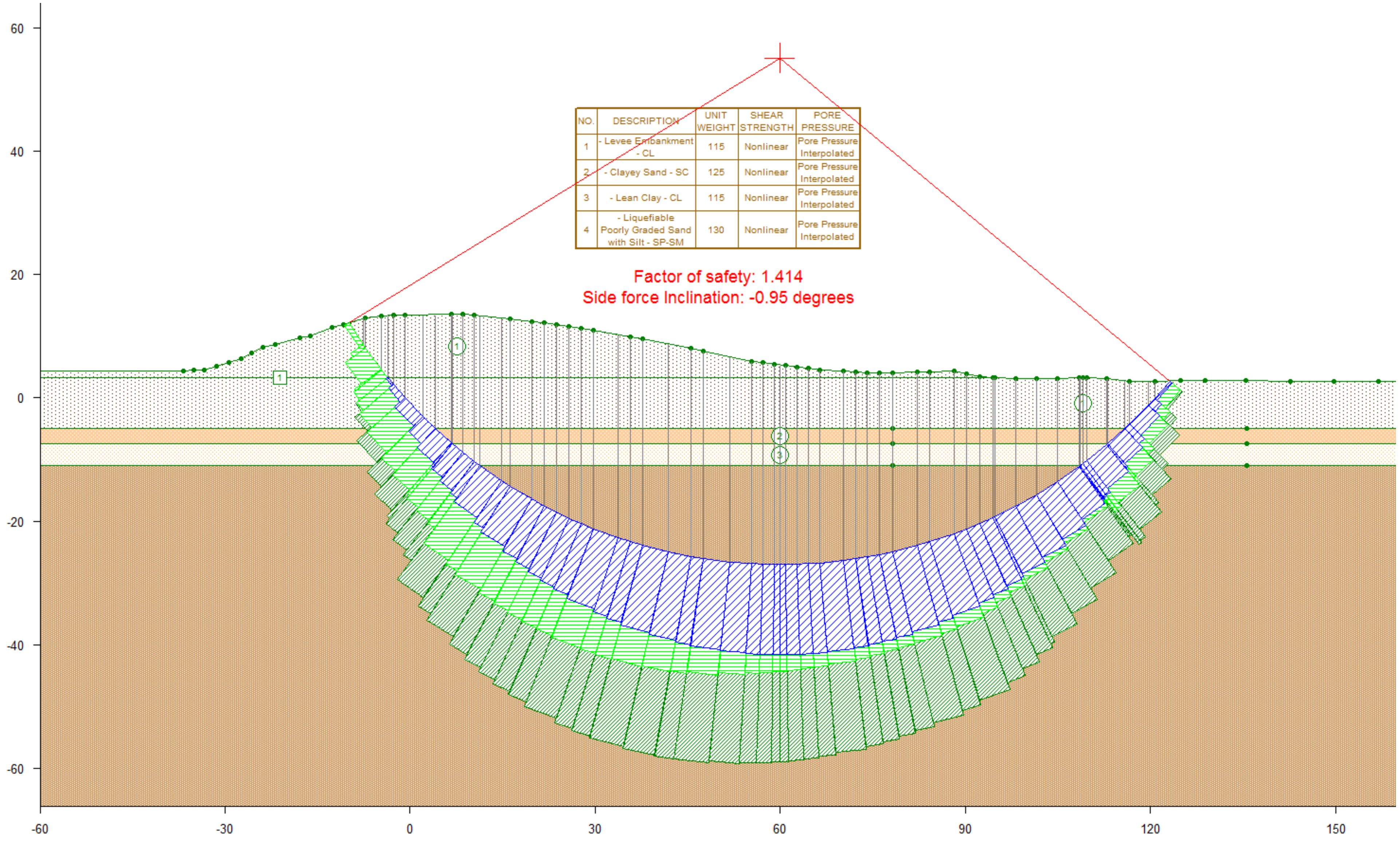
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
3	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

Fig 32(b). Lincoln Village Station 43+57 – Waterside – Option 2: Wedge (PHI = 4.7 in liquefiable material)



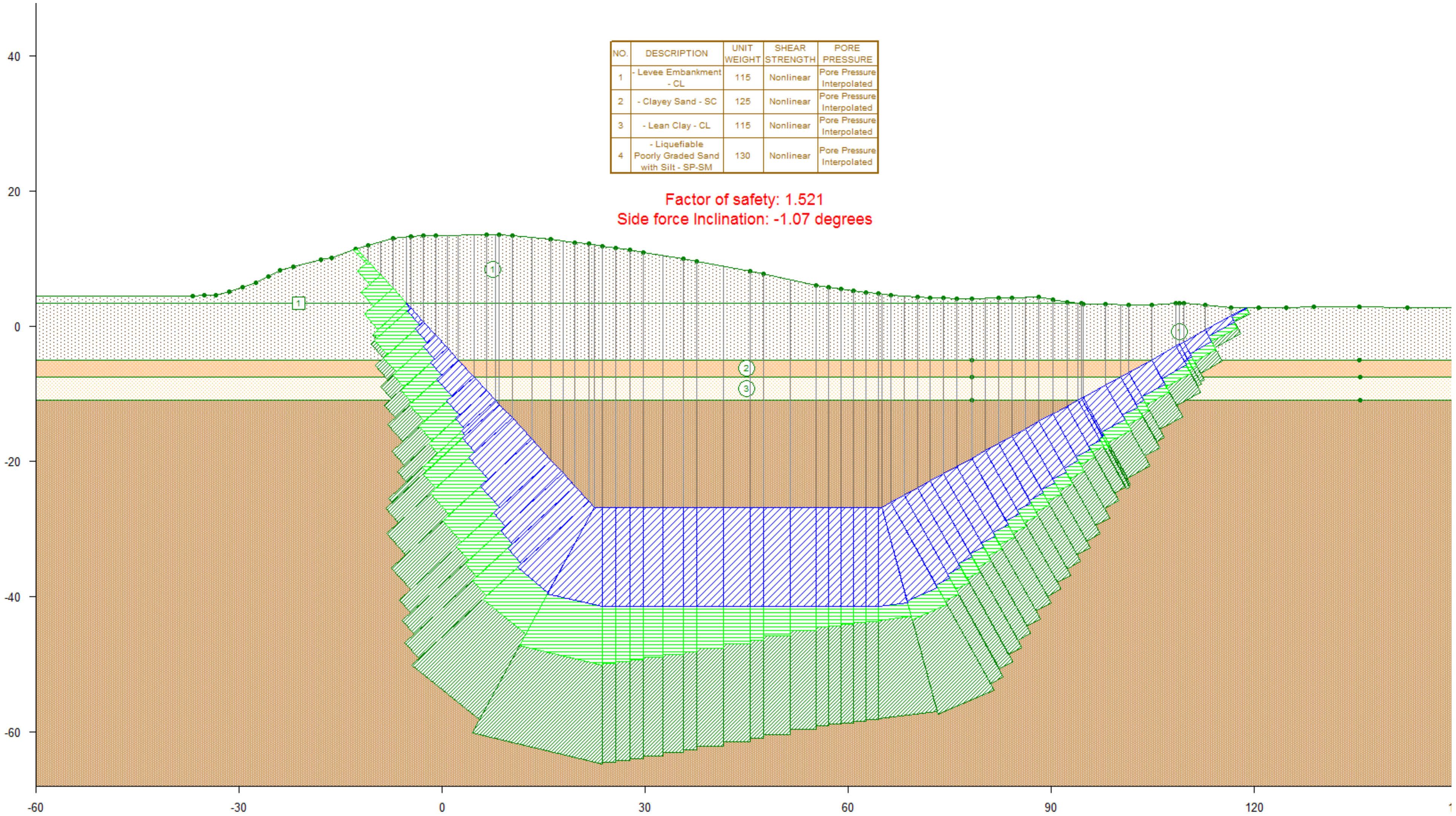
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
3	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

Fig F-33(a). Lincoln Village Station 43+57 – Landside – Option 3: Circular (Sr = 201 psf in liquefiable material)



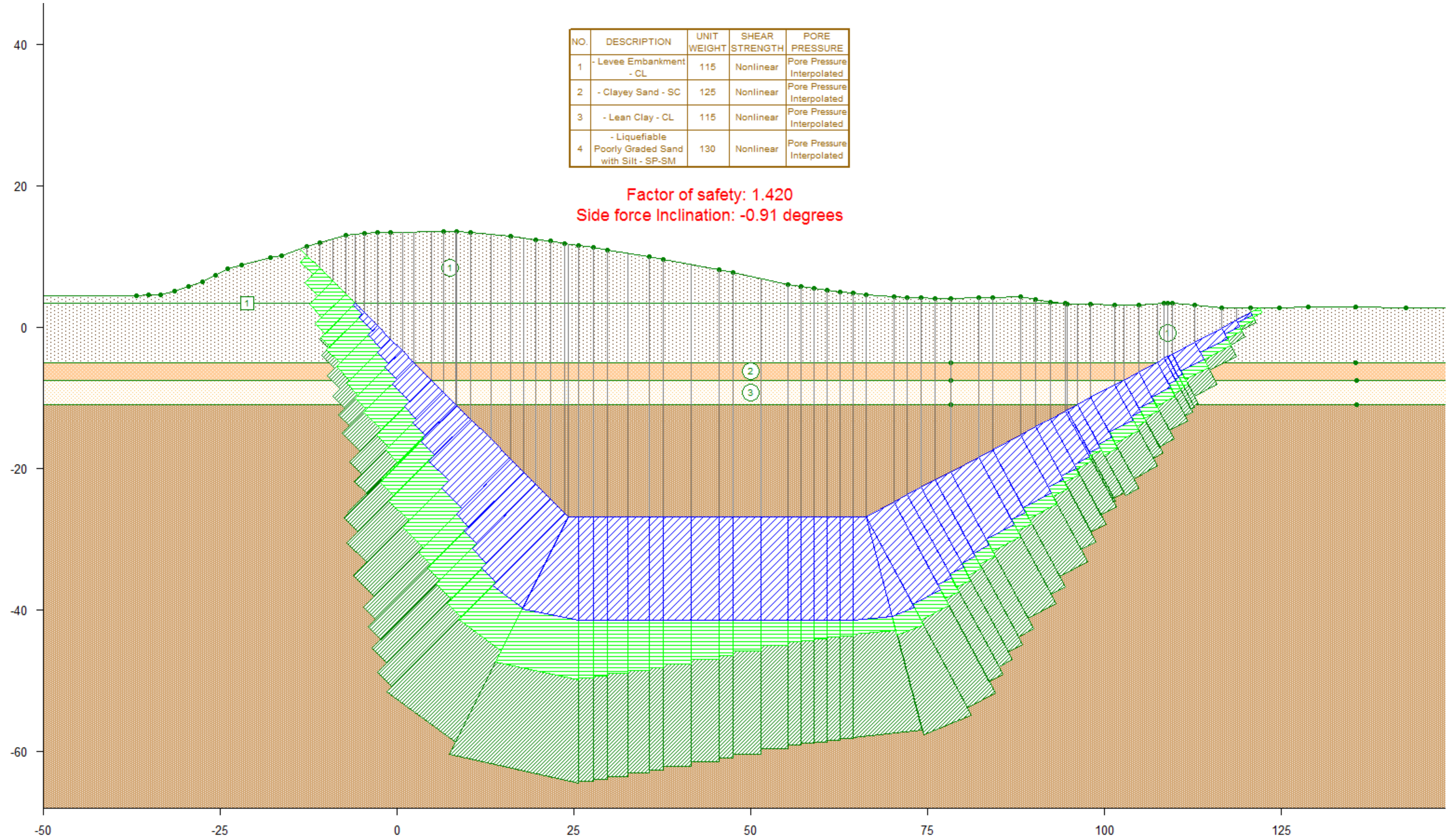
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
3	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

Fig 33(b). Lincoln Village Station 43+57 – Landside – Option 3: Circular (PHI = 4.7 in liquefiable material)



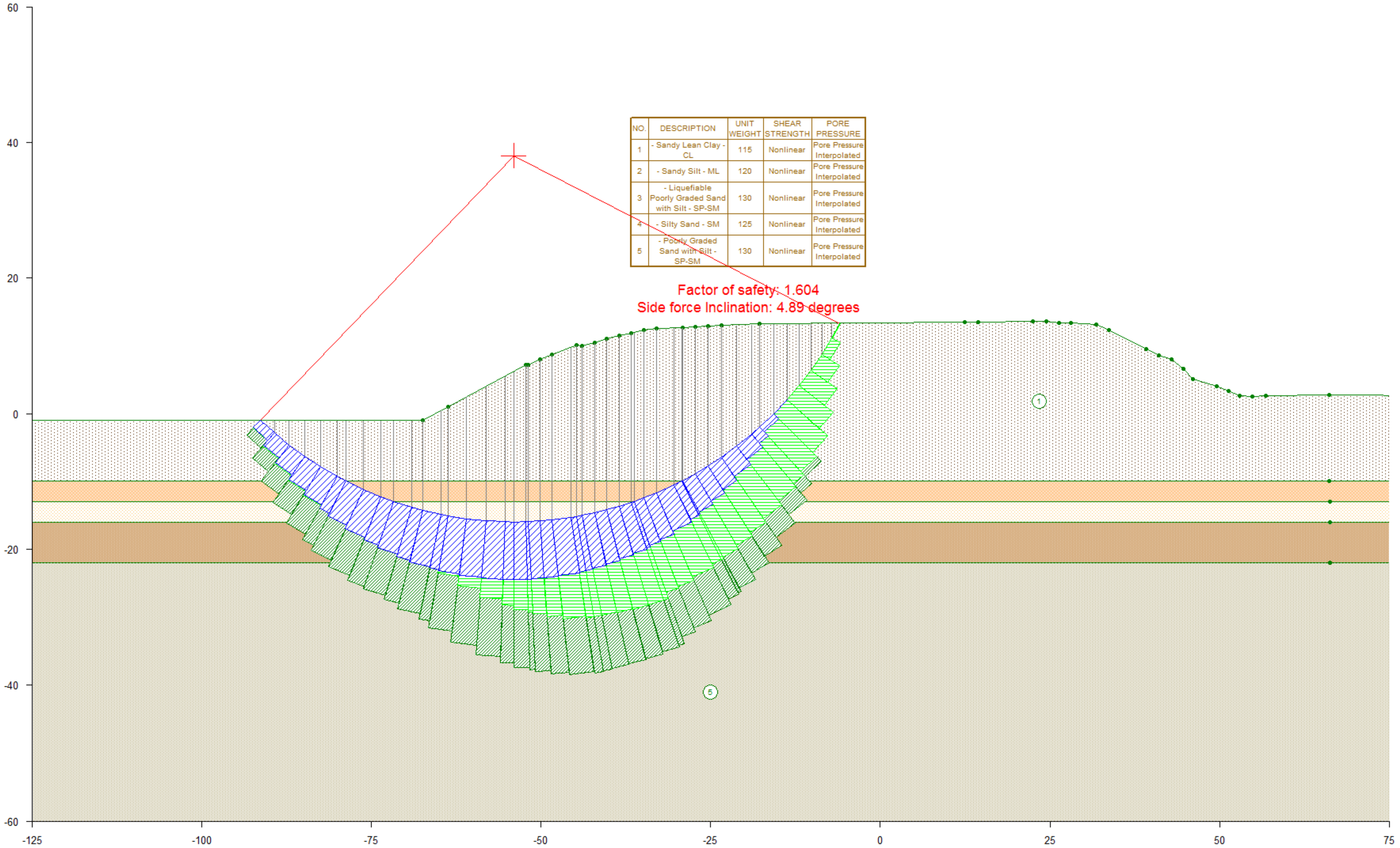
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
3	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

Fig F-34(a). Lincoln Village Station 43+57 – Landside – Option 4: Wedge (Sr = 201 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Clayey Sand - SC	125	Nonlinear	Pore Pressure Interpolated
3	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
4	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

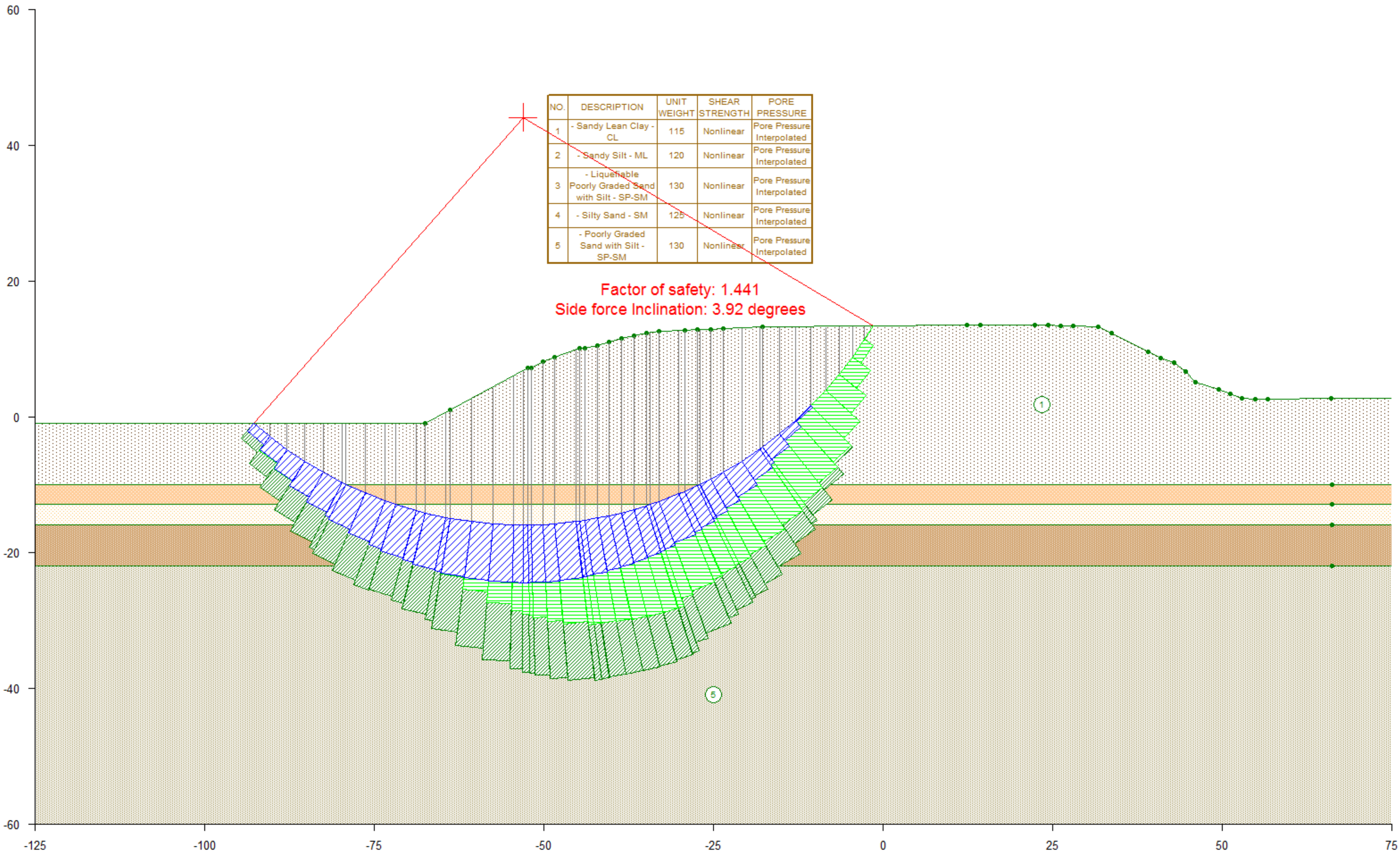
Fig 34(b). Lincoln Village Station 43+57 – Landside – Option 4: Wedge (PHI = 4.7 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Sandy Silt - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.604
Side force Inclination: 4.89 degrees

Fig F-35(a). Lincoln Village Station 109+90 – Waterside – Option 1: Circular (Sr = 282 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Sandy Silt - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.441
Side force Inclination: 3.92 degrees

Fig 35(b). Lincoln Village Station 109+90 – Waterside – Option 1: Circular (PHI = 6.0 in liquefiable material)

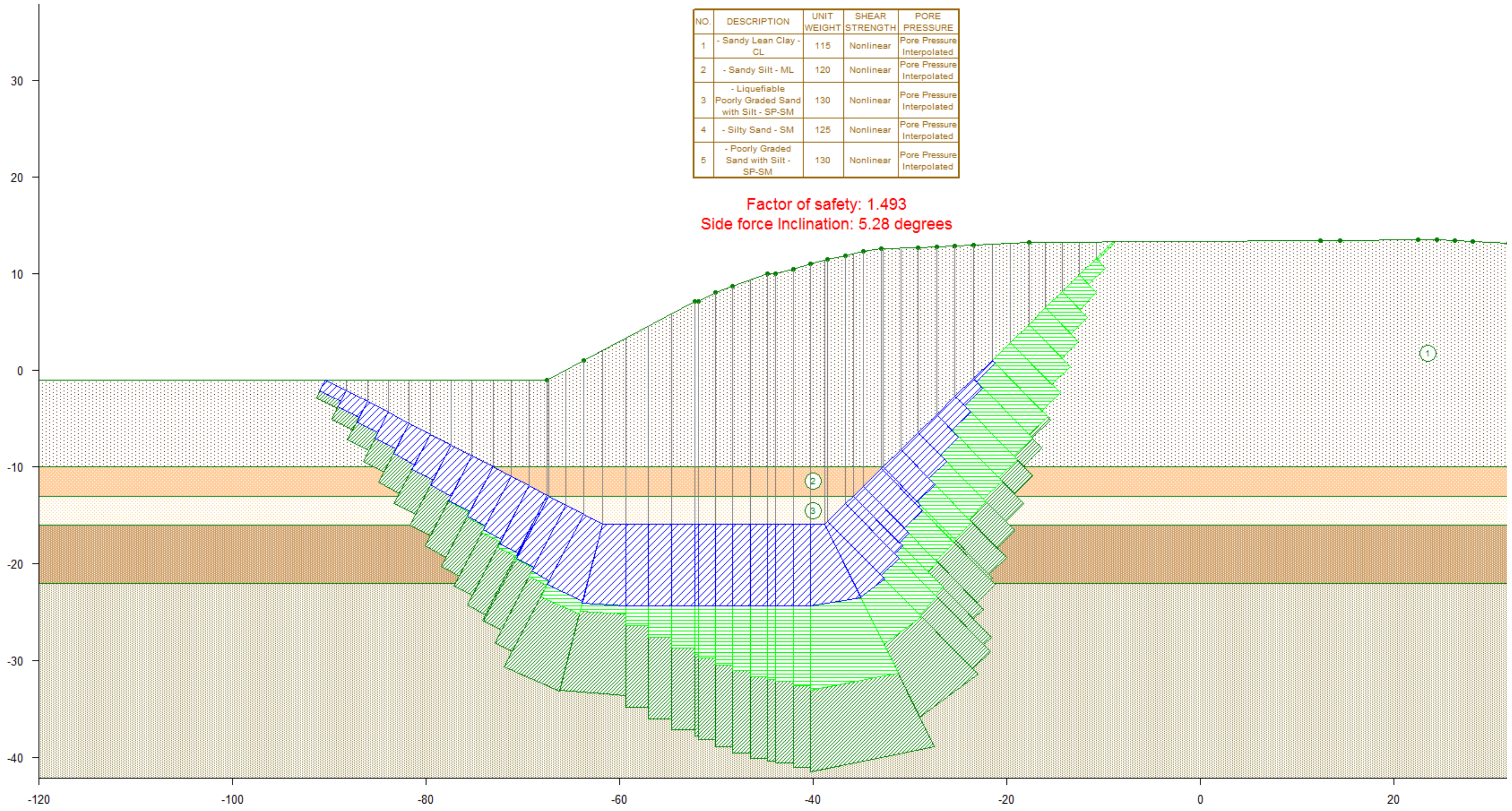
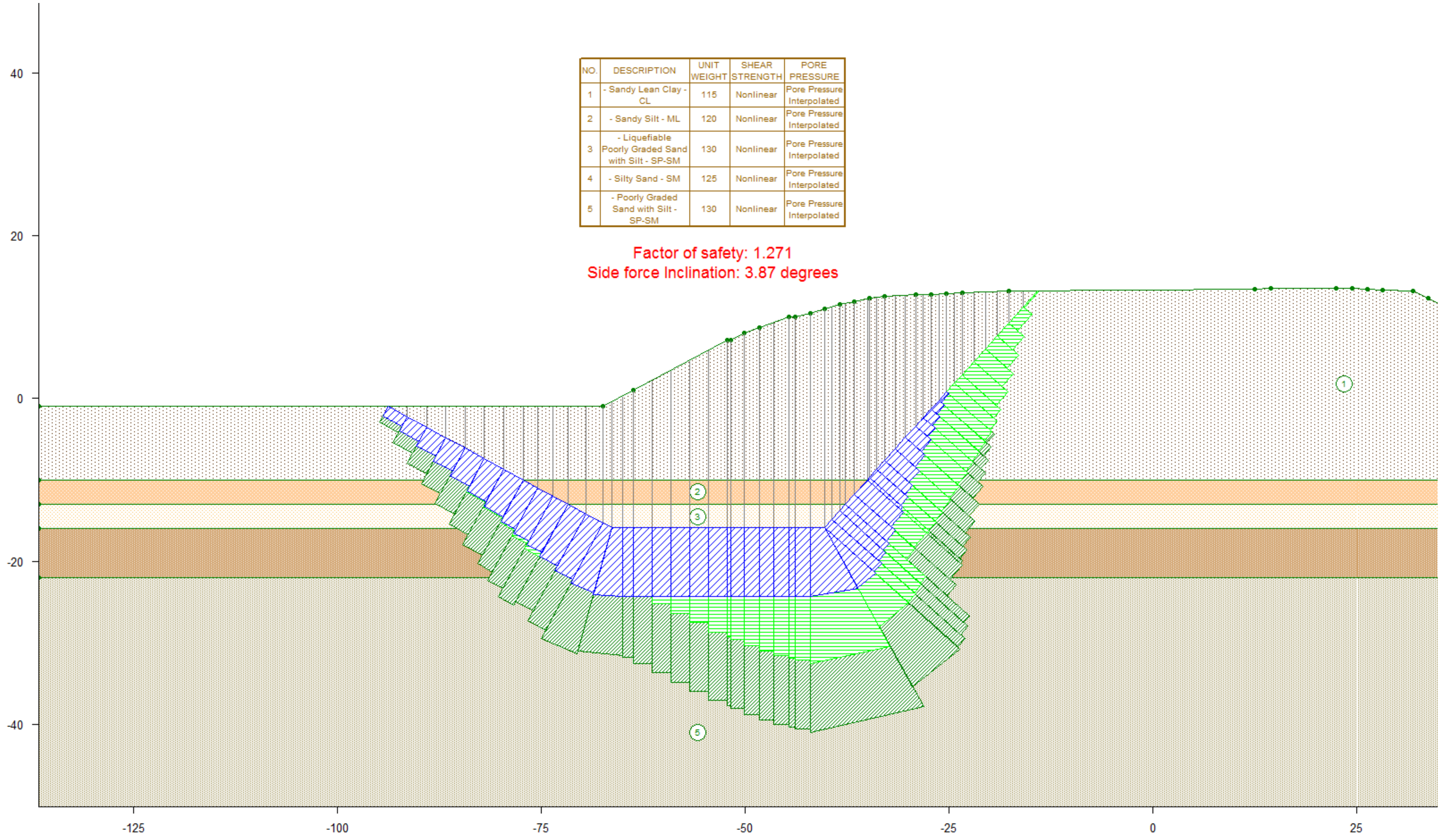


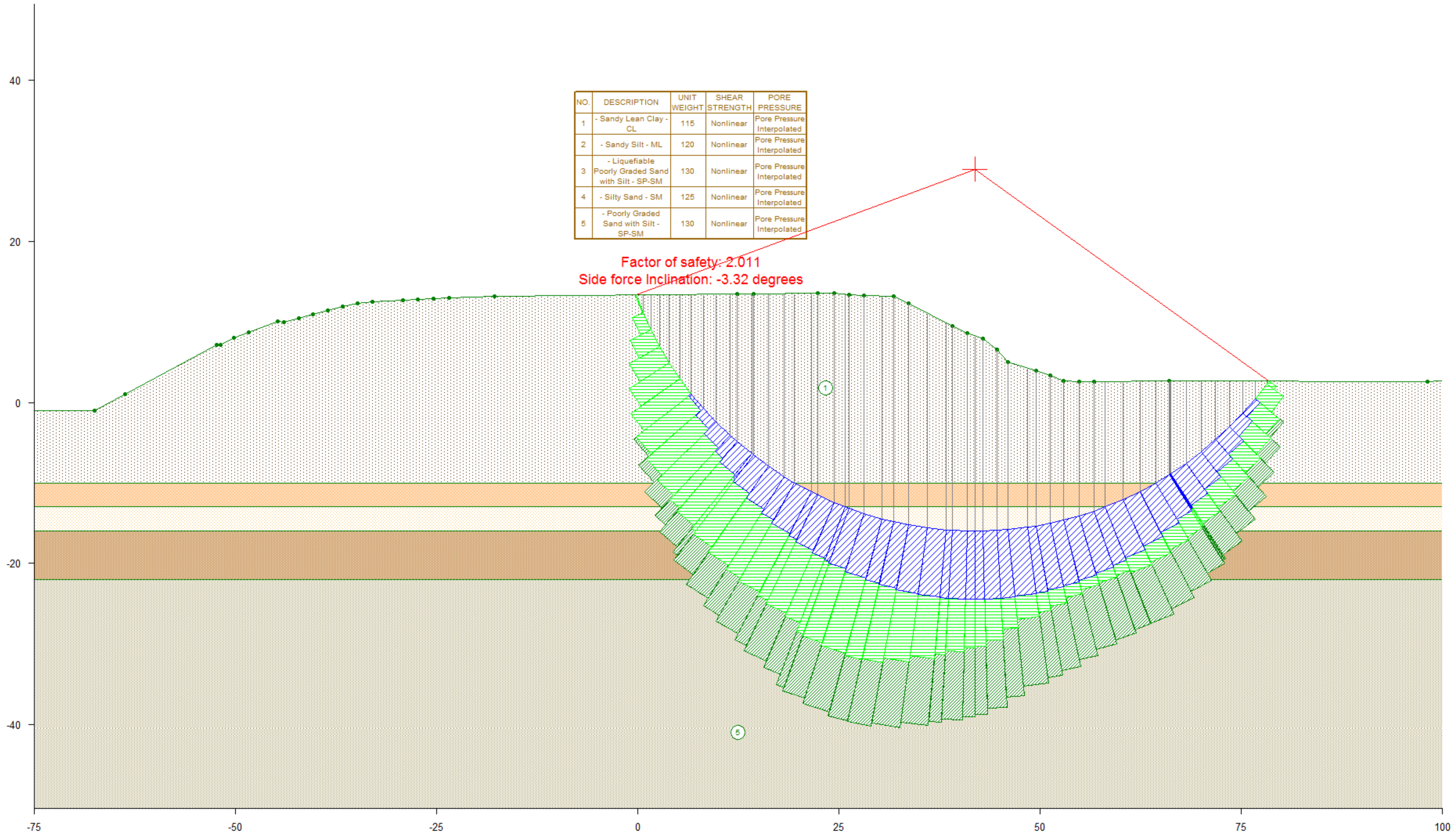
Fig F-36(a). Lincoln Village Station 109+90 – Waterside – Option 2: Wedges (Sr = 282 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Sandy Silt - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

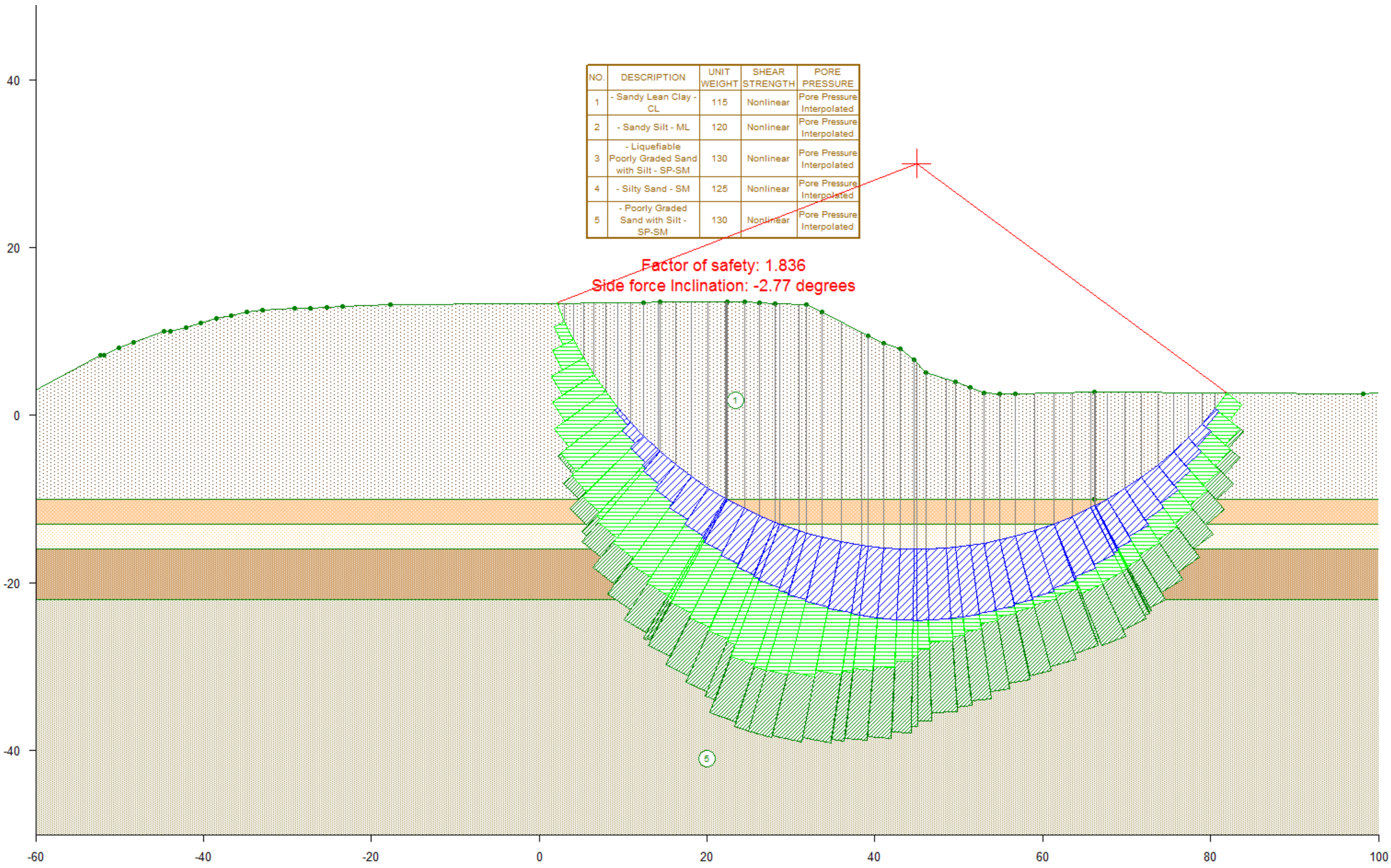
Factor of safety: 1.271
Side force Inclination: 3.87 degrees

Fig 36(b). Lincoln Village Station 109+90 – Waterside – Option 2: Wedges (PHI = 6.0 in liquefiable material)



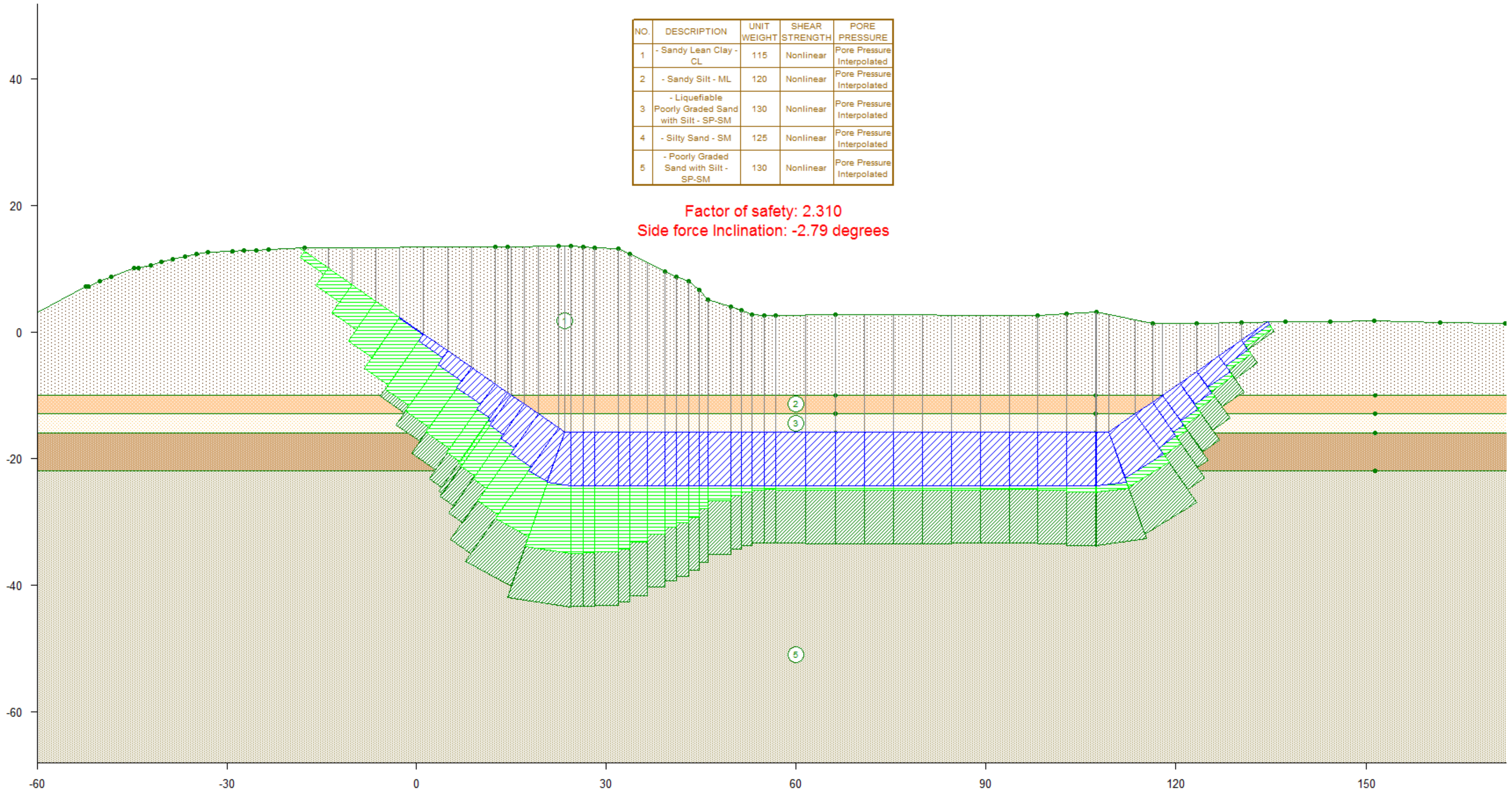
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Sandy Silt - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

Fig F-37(a). Lincoln Village Station 109+90 – Landside – Option 3: Circular (Sr = 282 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Sandy Silt - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

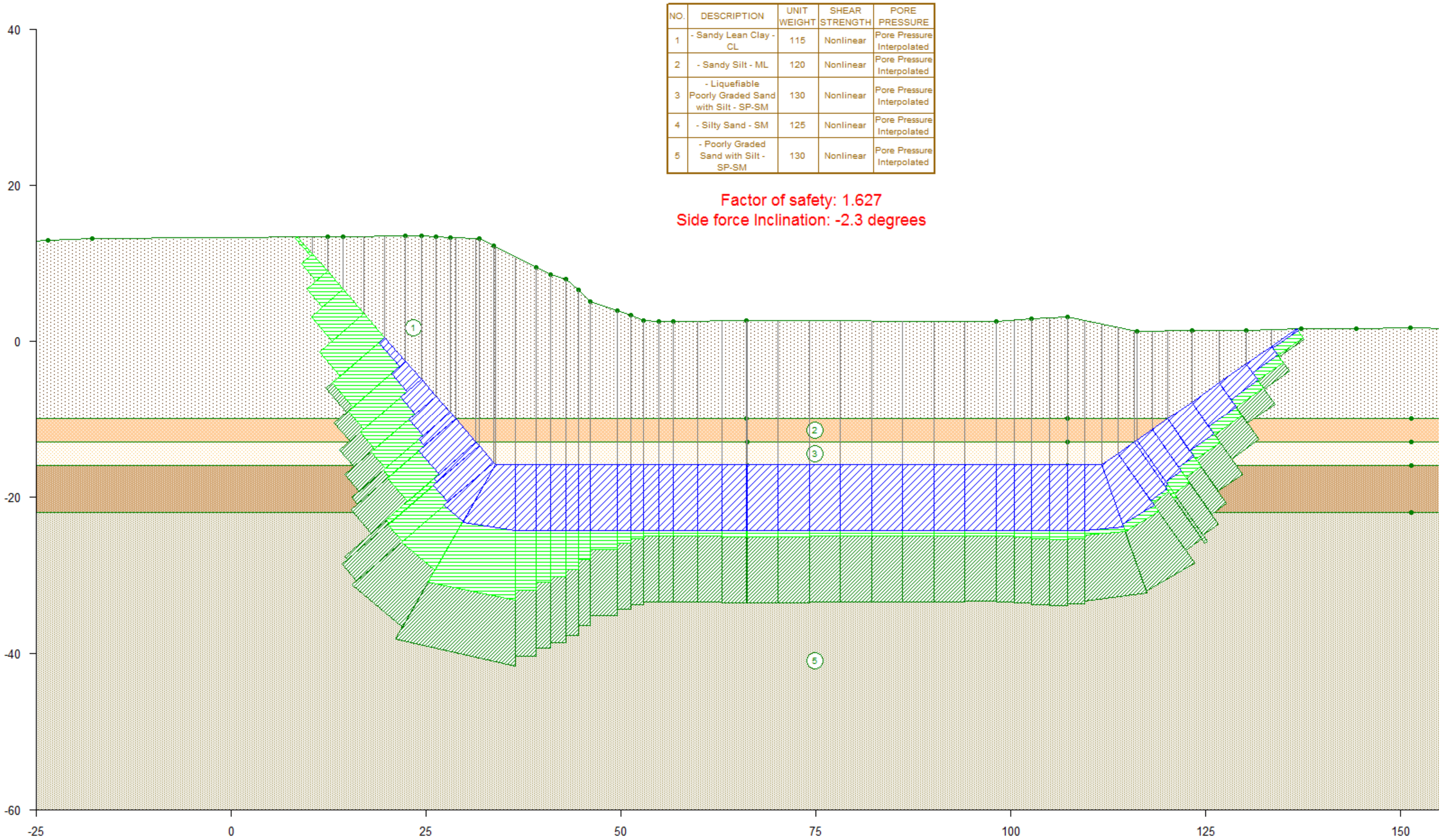
Fig 37(b). Lincoln Village Station 109+90 – Landside – Option 3: Circular (PHI = 6.0 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Sandy Silt - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

Factor of safety: 2.310
Side force Inclination: -2.79 degrees

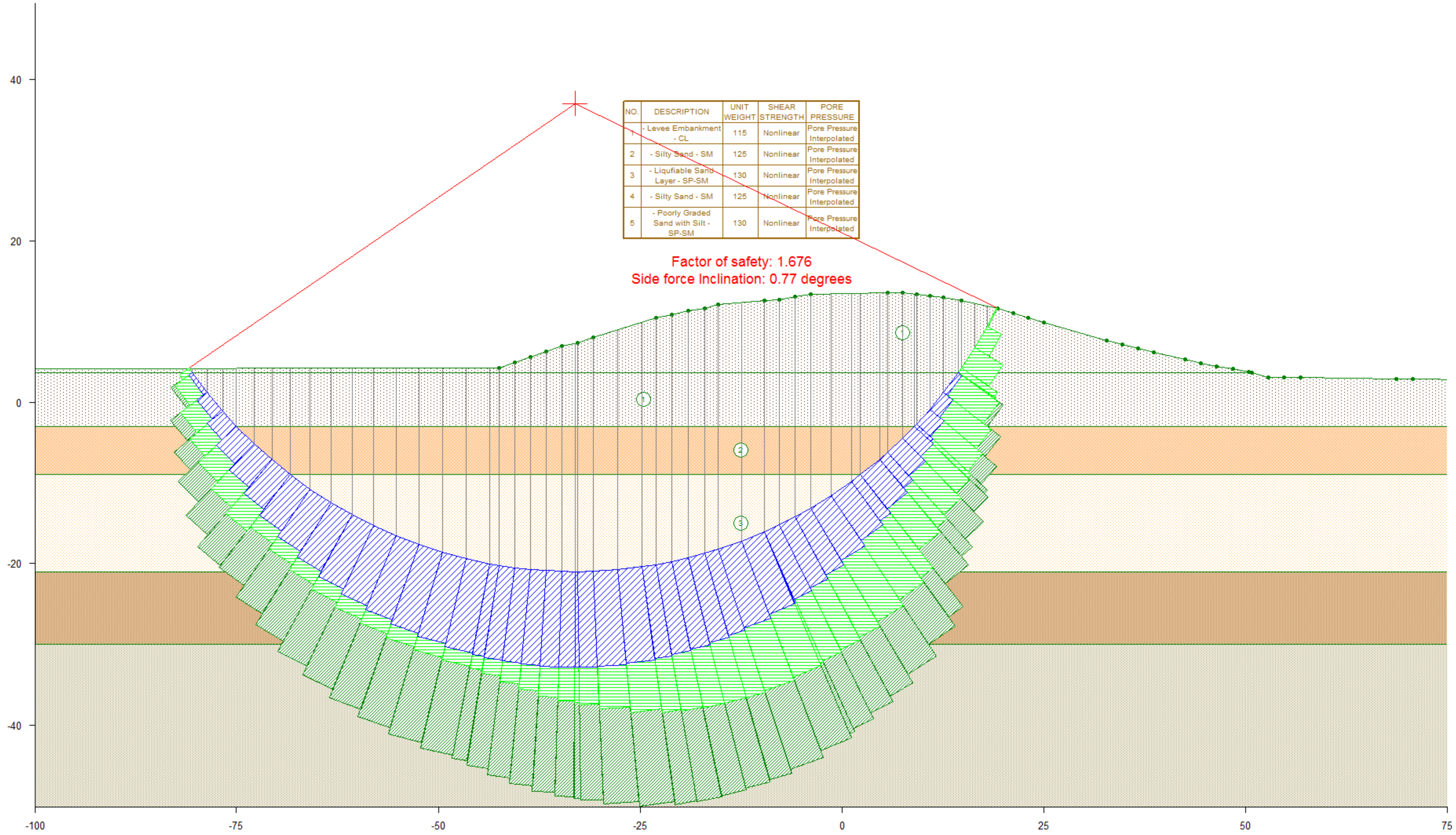
Fig F-38(a). Lincoln Village Station 109+90 – Landside – Option 4: Wedge (Sr = 282 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Sandy Silt - ML	120	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

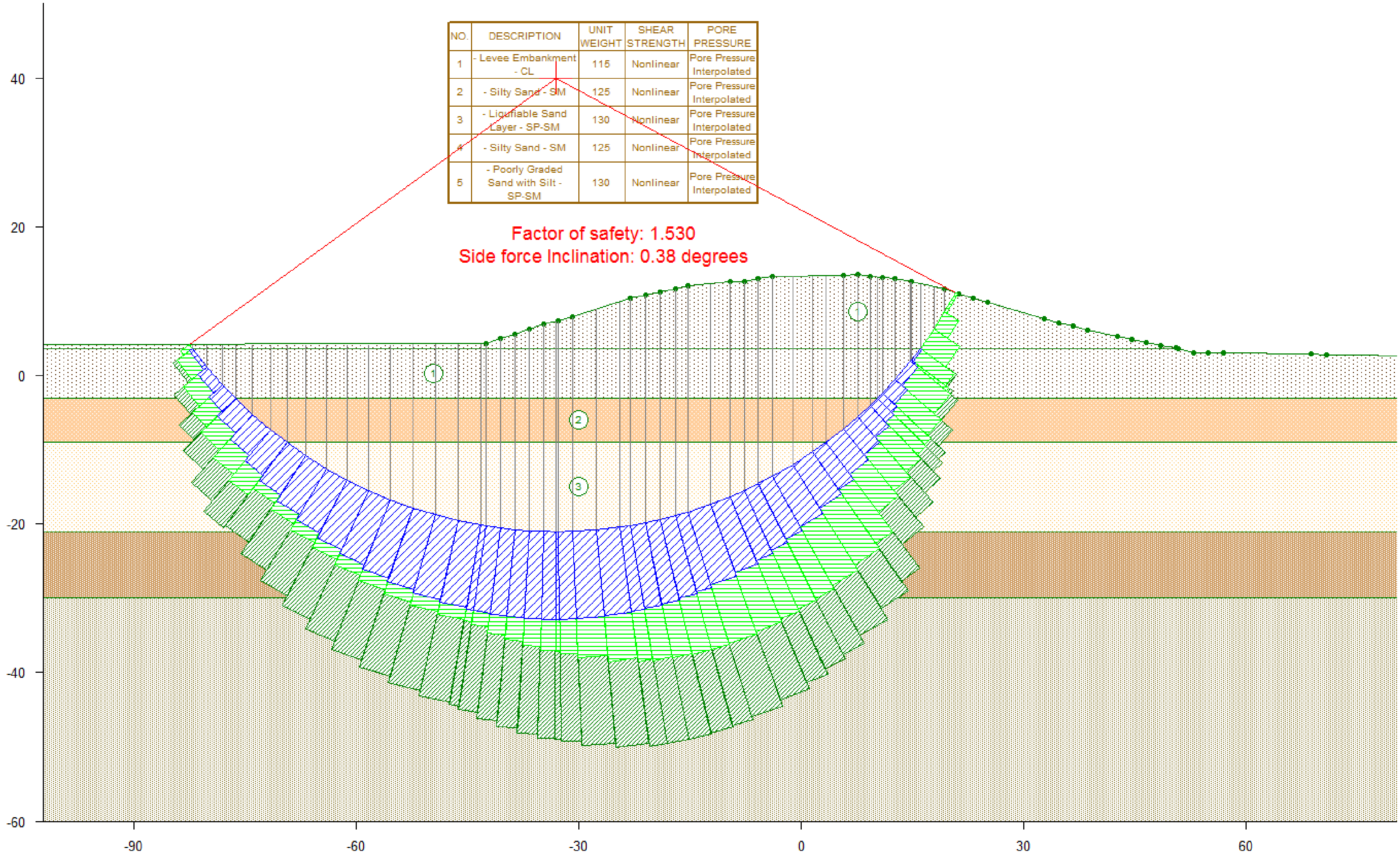
Factor of safety: 1.627
Side force Inclination: -2.3 degrees

Fig 38(b). Lincoln Village Station 109+90 – Landside – Option 4: Wedge (PHI = 6.0 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liqufiable Sand Layer - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

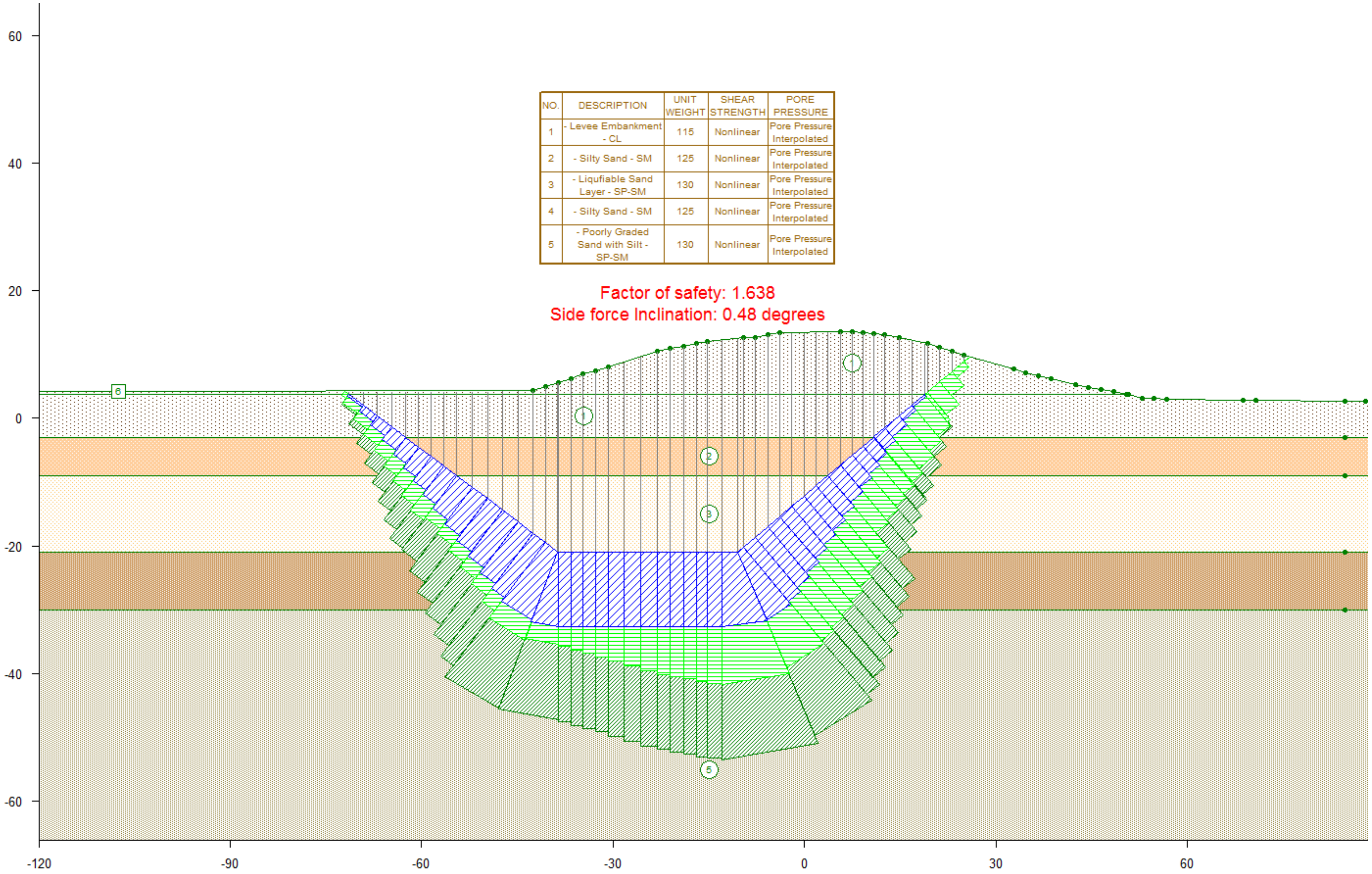
Fig F-39(a). Lincoln Village Station 159+48 – Waterside – Option 1: Circular (Sr = 207 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liquifiable Sand Layer - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.530
Side force Inclination: 0.38 degrees

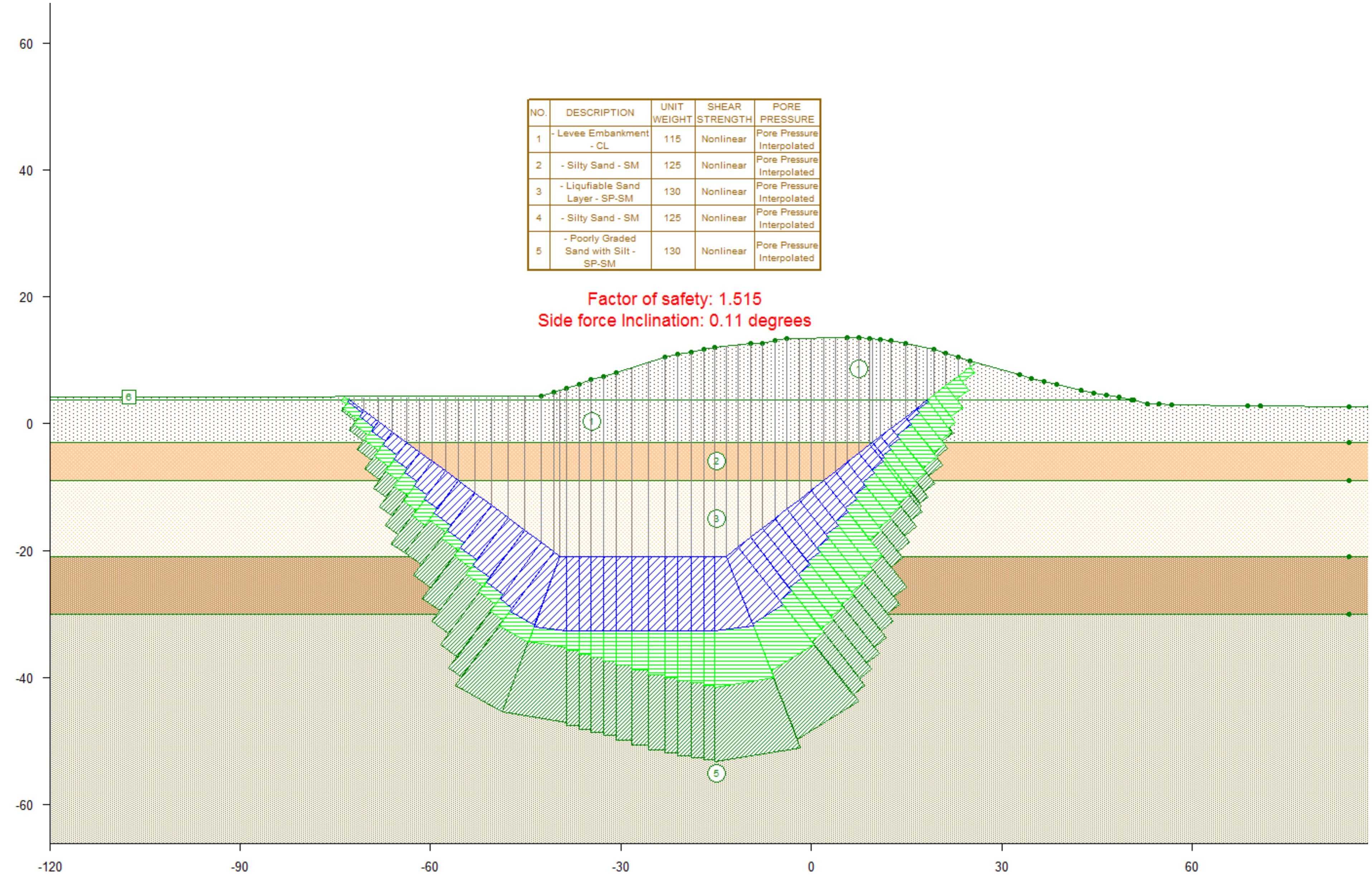
Fig 39b. Lincoln Village Station 159+48 – Waterside – Option 1: Circular (PHI = 5.1 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liqufiable Sand Layer - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.638
Side force Inclination: 0.48 degrees

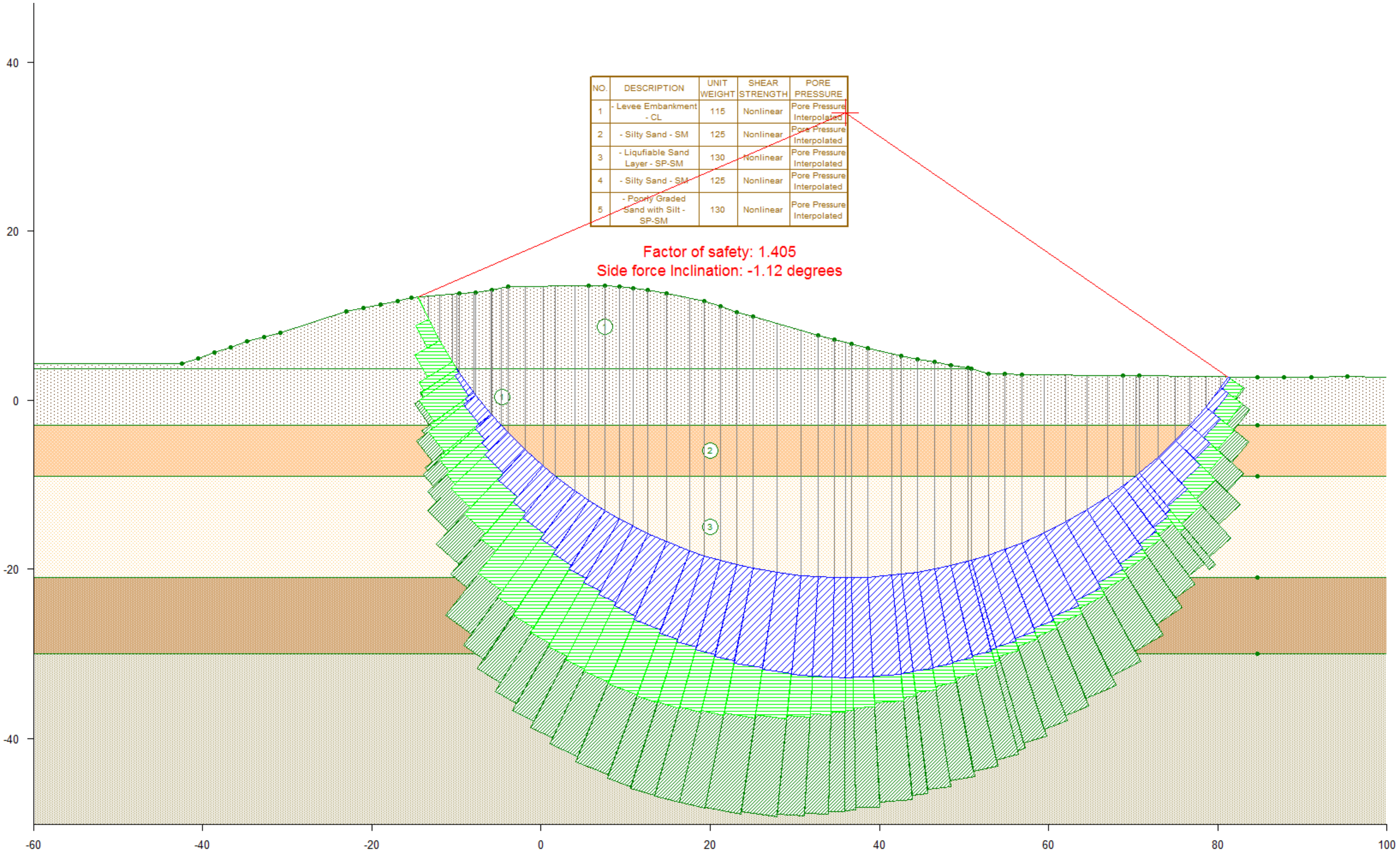
Fig F-40(a). Lincoln Village Station 159+48 – Waterside – Option 2: Wedges (Sr = 207 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liqufiable Sand Layer - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

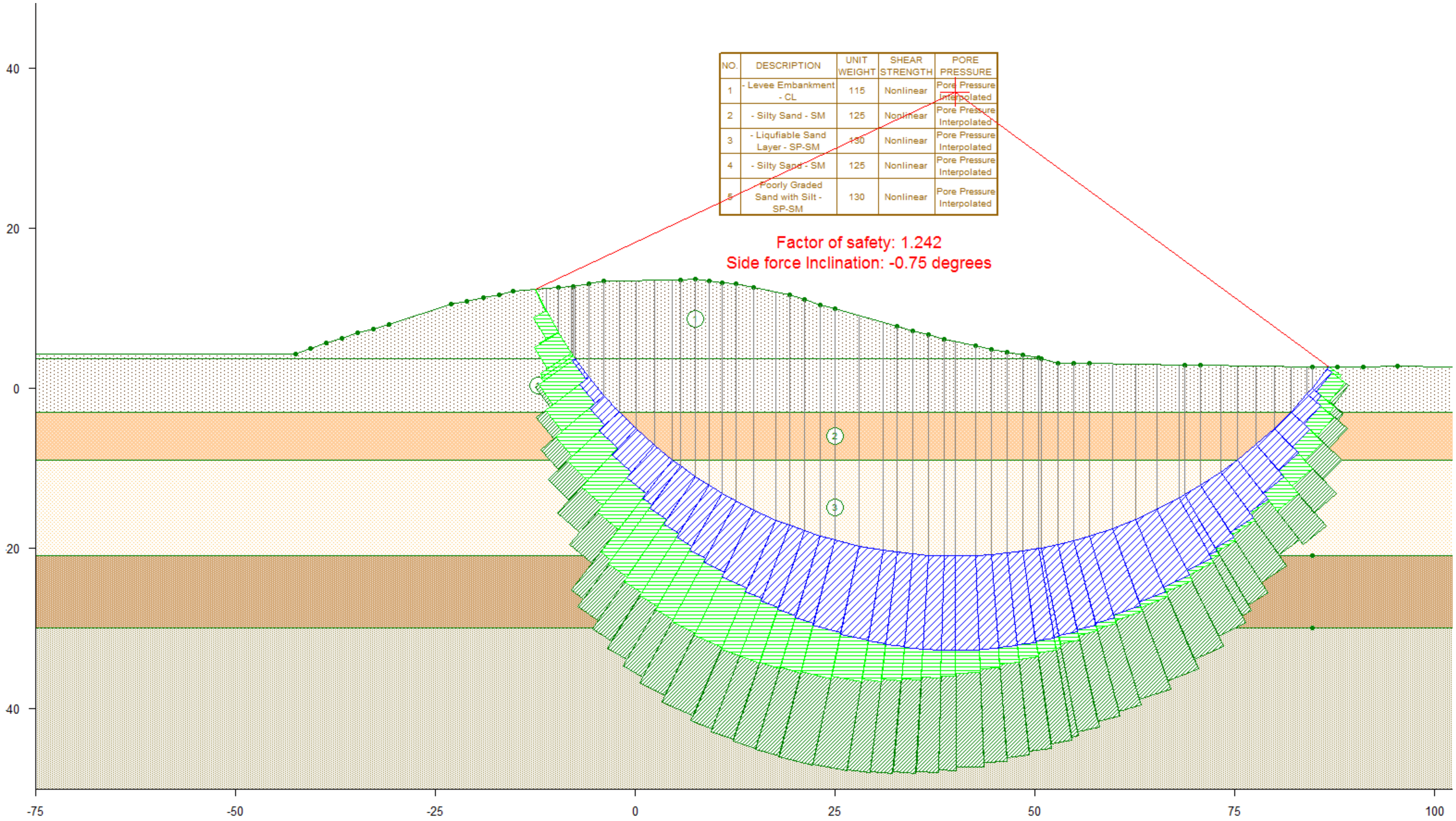
Factor of safety: 1.515
Side force Inclination: 0.11 degrees

Fig 40(b). Lincoln Village Station 159+48 – Waterside – Option 2: Wedges (PHI = 5.1 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liqufiable Sand Layer - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

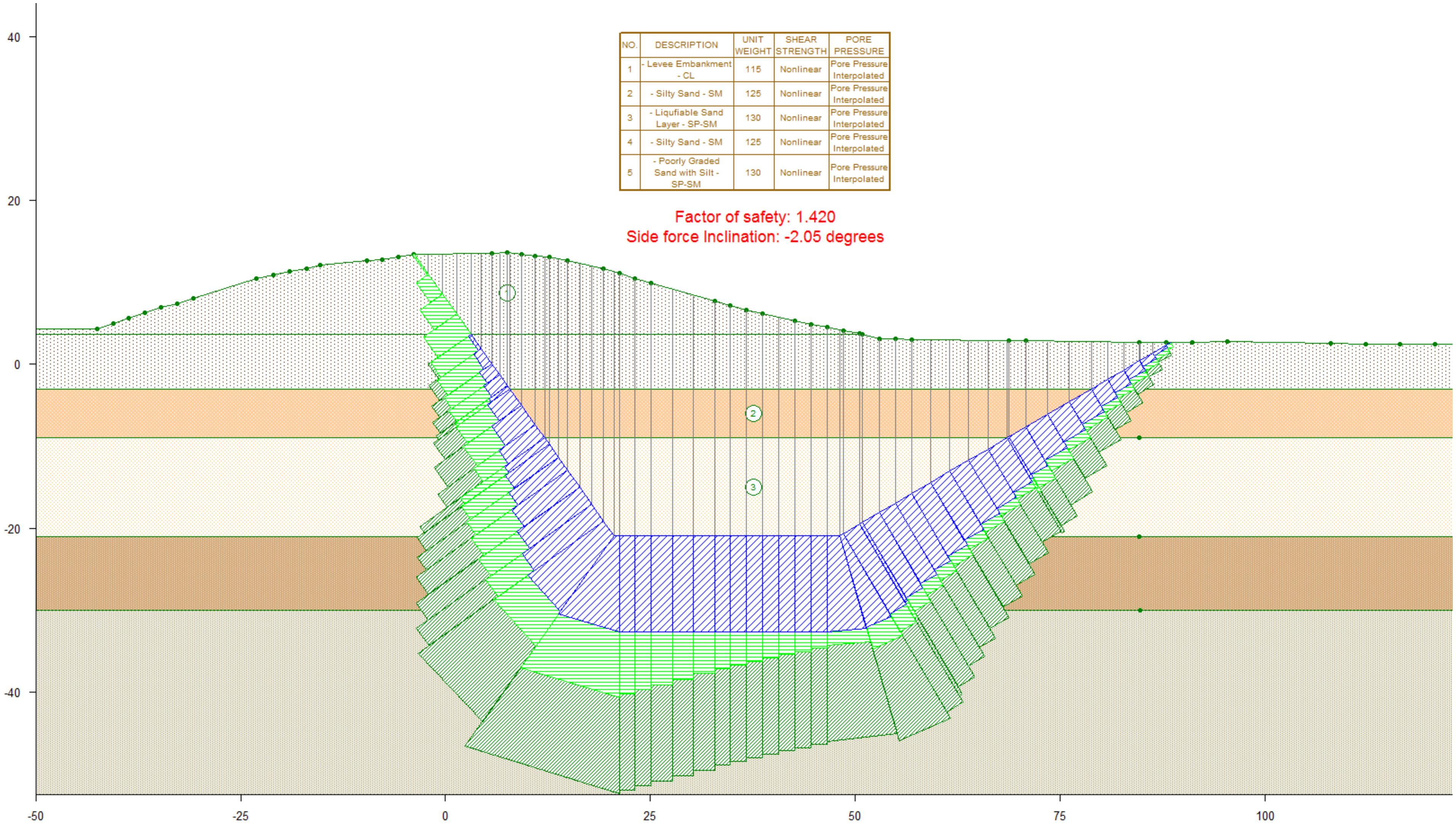
Fig F-41(a). Lincoln Village Station 159+48– Landside – Option 3: Circular (Sr = 207 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liqufiable Sand Layer - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.242
Side force Inclination: -0.75 degrees

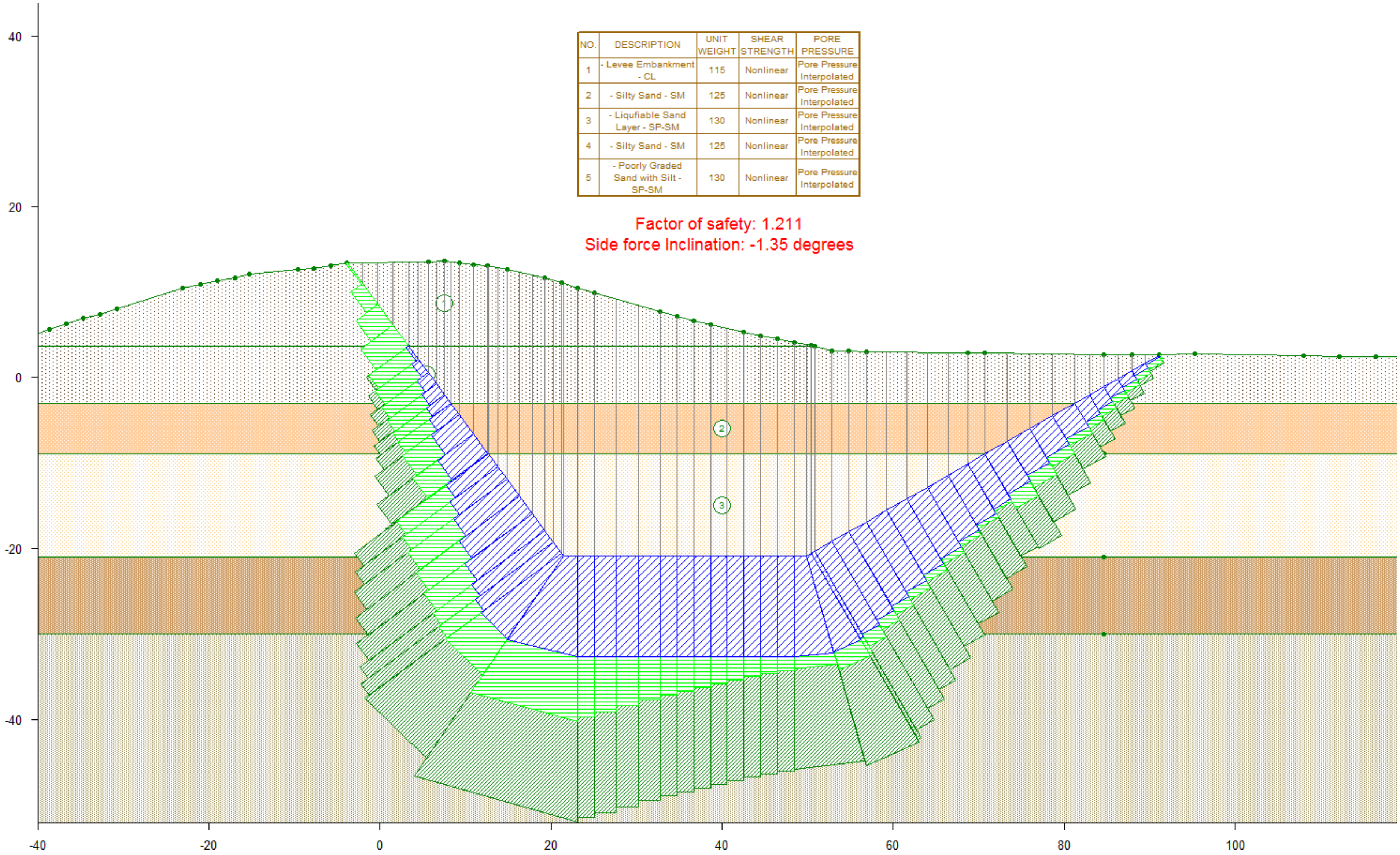
Fig 41(b). Lincoln Village Station 159+48– Landside – Option 3: Circular (PHI = 5.1 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liqufiable Sand Layer - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

Factor of safety: 1.420
Side force Inclination: -2.05 degrees

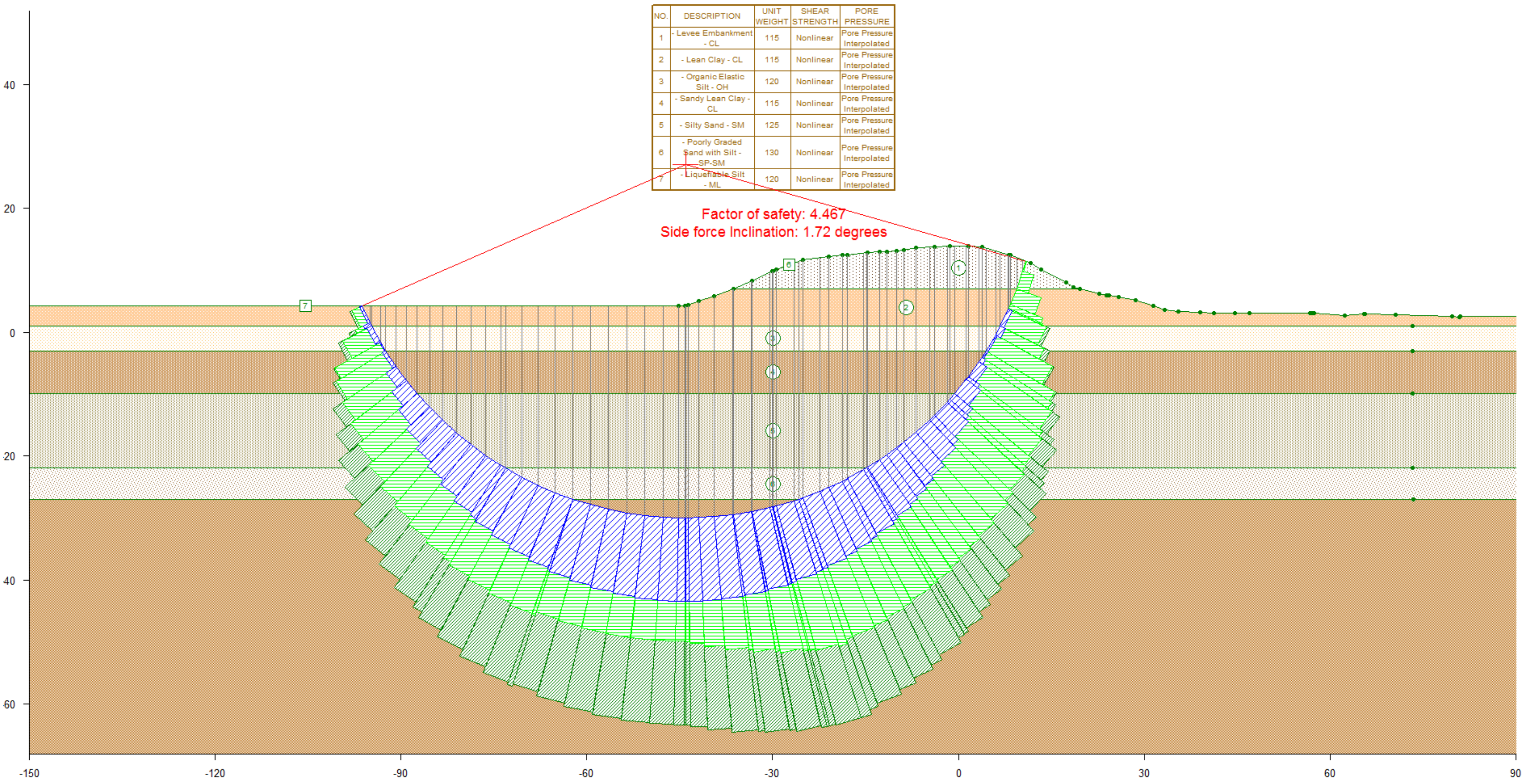
Fig F-42(a). Lincoln Village Station 159+48 – Landside – Option 4: Wedge (Sr = 207 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liqufiable Sand Layer - SP-SM	130	Nonlinear	Pore Pressure Interpolated
4	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
5	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated

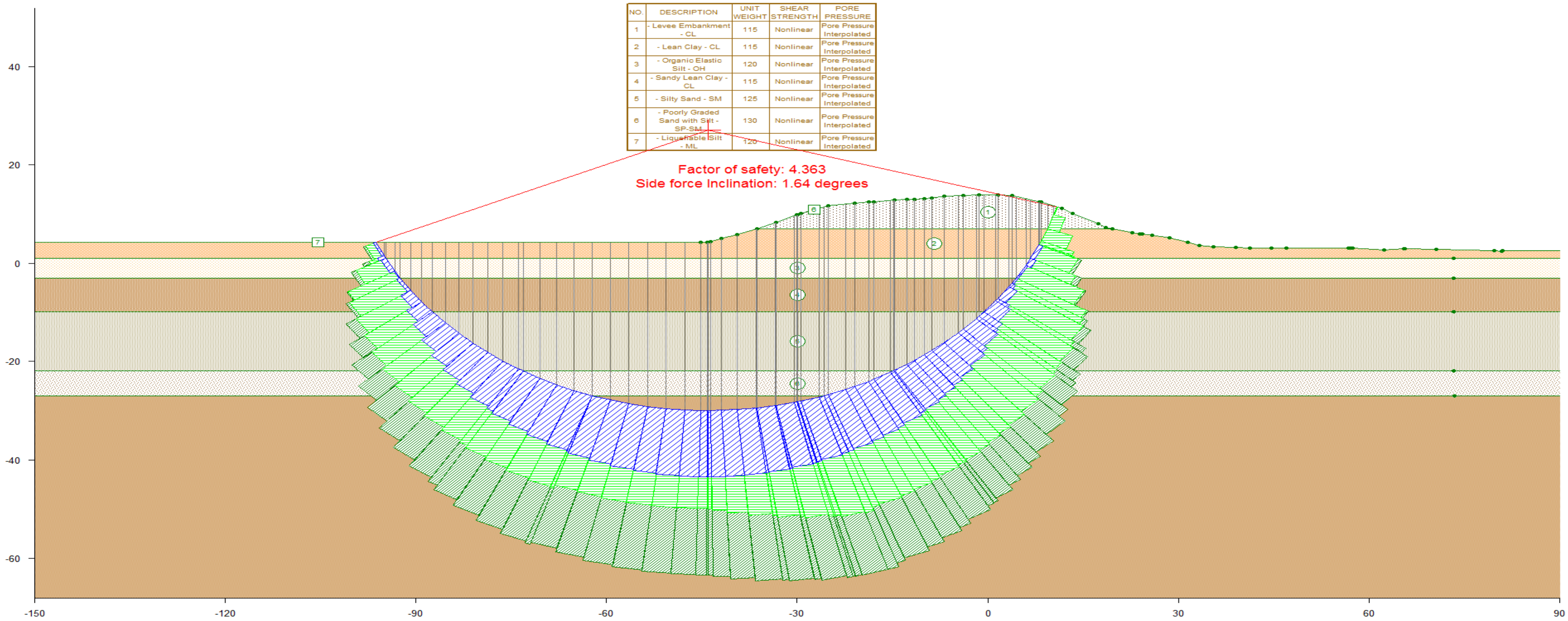
Factor of safety: 1.211
Side force Inclination: -1.35 degrees

Fig 42(b). Lincoln Village Station 159+48 – Landside – Option 4: Wedge (PHI = 5.1 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
3	- Organic Elastic Silt - OH	120	Nonlinear	Pore Pressure Interpolated
4	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
6	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable Silt - ML	120	Nonlinear	Pore Pressure Interpolated

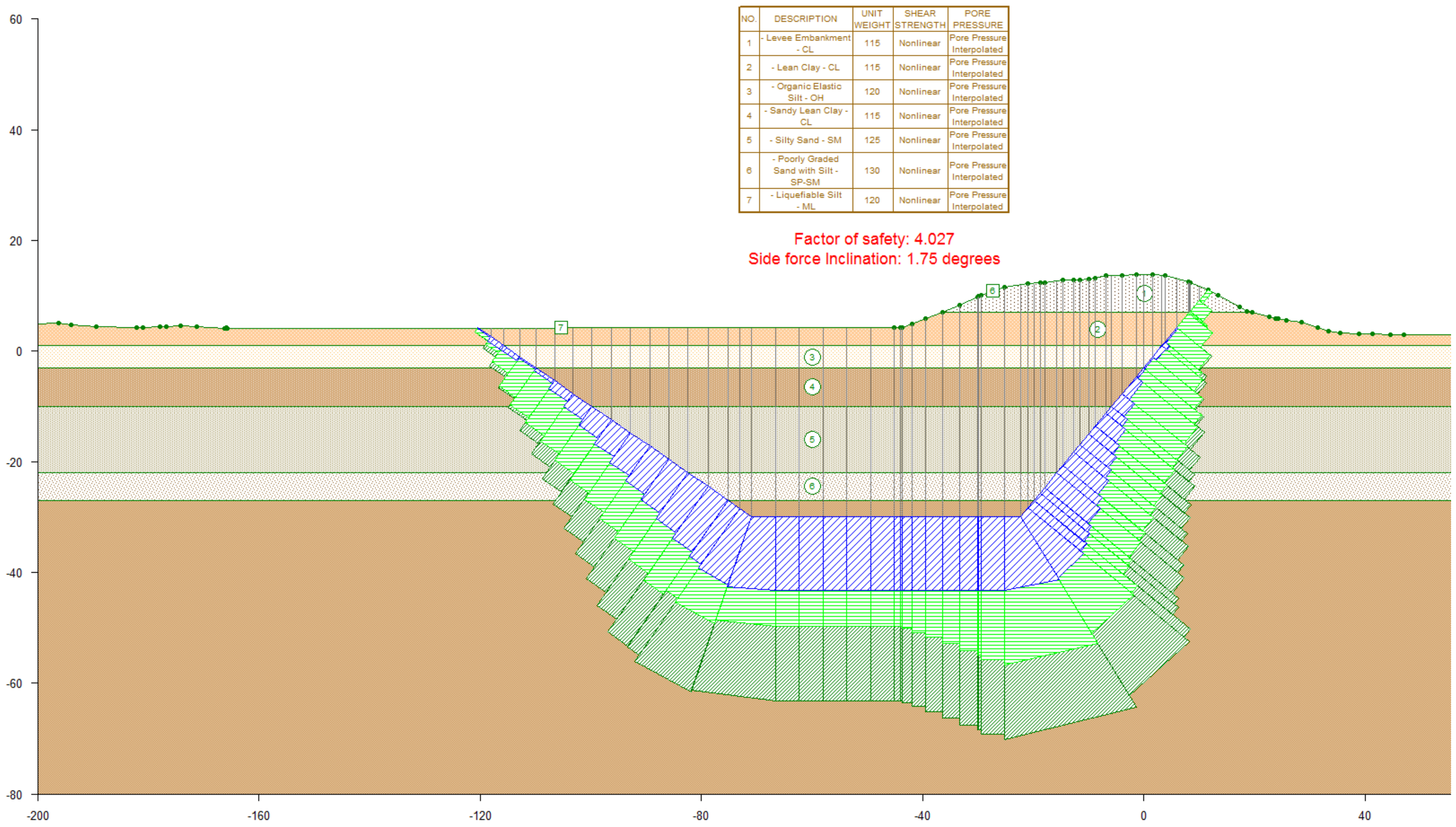
Fig F-43(a). Lincoln Village Station 164+99 – Waterside – Option 1: Circular (Sr = 224 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
3	- Organic Elastic Silt - OH	120	Nonlinear	Pore Pressure Interpolated
4	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
6	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable Silt - ML	120	Nonlinear	Pore Pressure Interpolated

Factor of safety: 4.363
Side force Inclination: 1.64 degrees

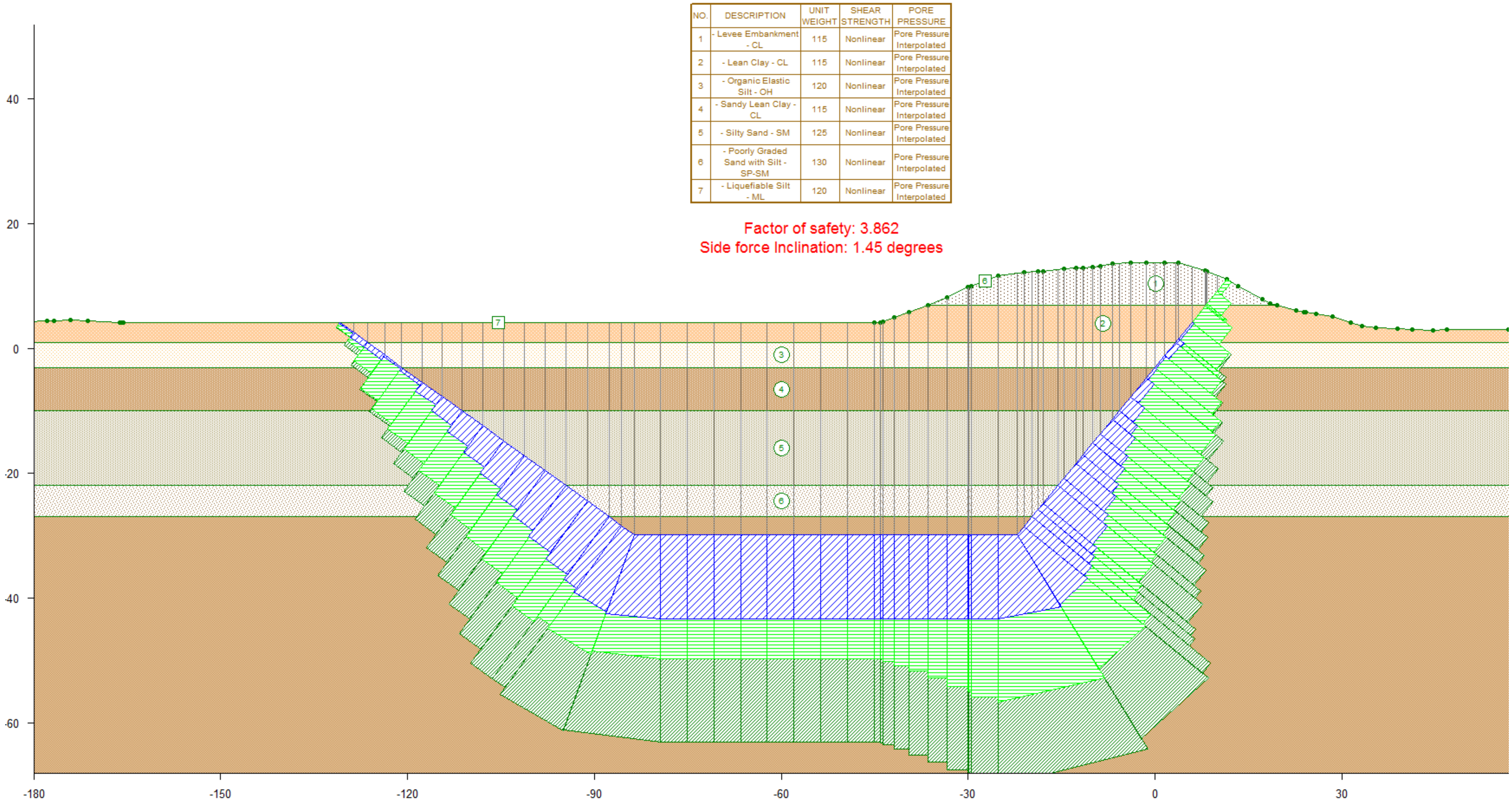
Fig 43(b). Lincoln Village Station 164+99 – Waterside – Option 1: Circular (PHI = 3.4 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
3	- Organic Elastic Silt - OH	120	Nonlinear	Pore Pressure Interpolated
4	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
6	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable Silt - ML	120	Nonlinear	Pore Pressure Interpolated

Factor of safety: 4.027
Side force Inclination: 1.75 degrees

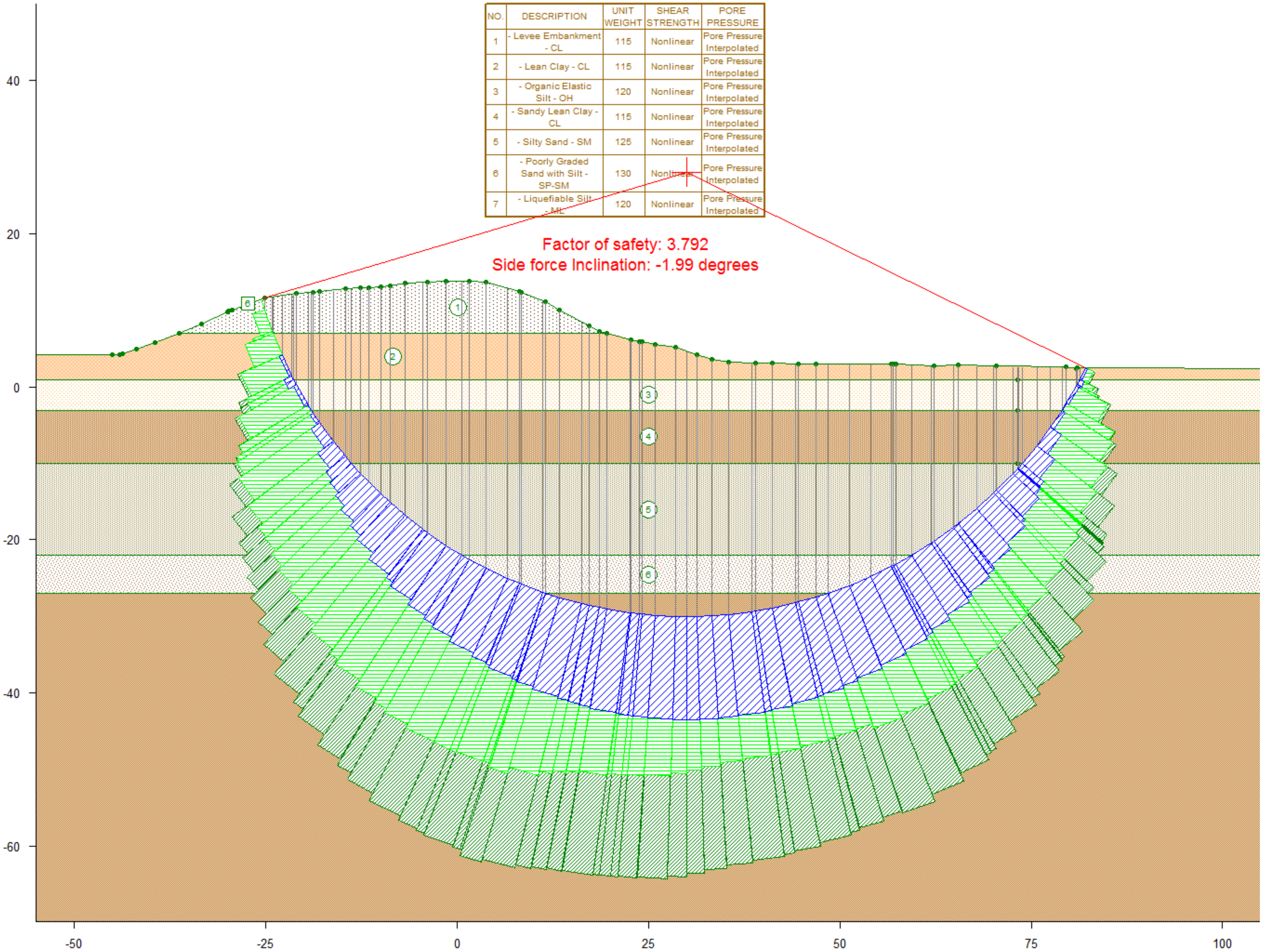
Fig F-44(a). Lincoln Village Station 164+99 – Waterside – Option 2: Wedges (Sr = 224 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
3	- Organic Elastic Silt - OH	120	Nonlinear	Pore Pressure Interpolated
4	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
6	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable Silt - ML	120	Nonlinear	Pore Pressure Interpolated

Factor of safety: 3.862
Side force Inclination: 1.45 degrees

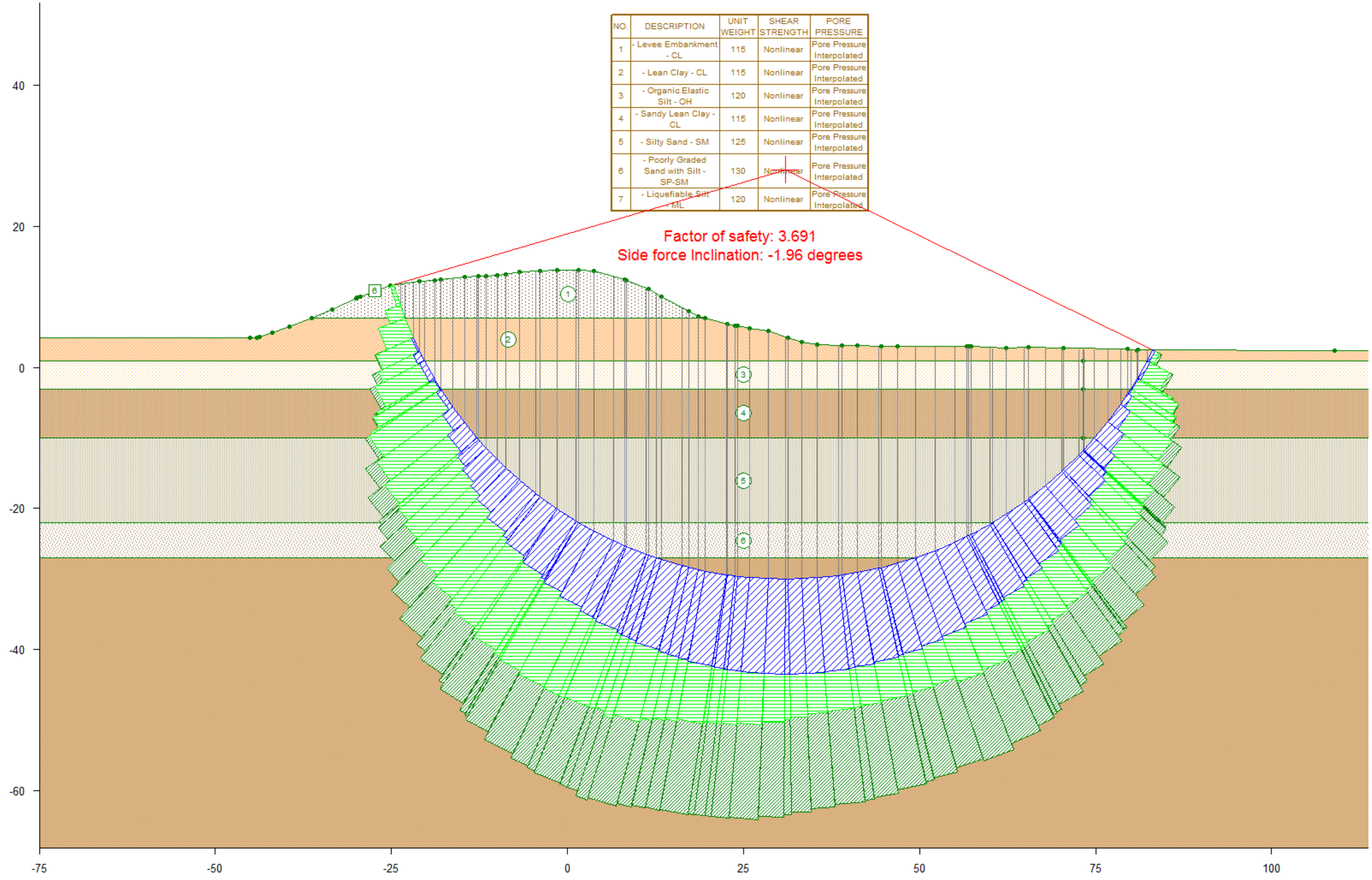
Fig 44(b). Lincoln Village Station 164+99 – Waterside – Option 2: Wedges (PHI = 3.4 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
3	- Organic Elastic Silt - OH	120	Nonlinear	Pore Pressure Interpolated
4	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
6	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable Silt - ML	120	Nonlinear	Pore Pressure Interpolated

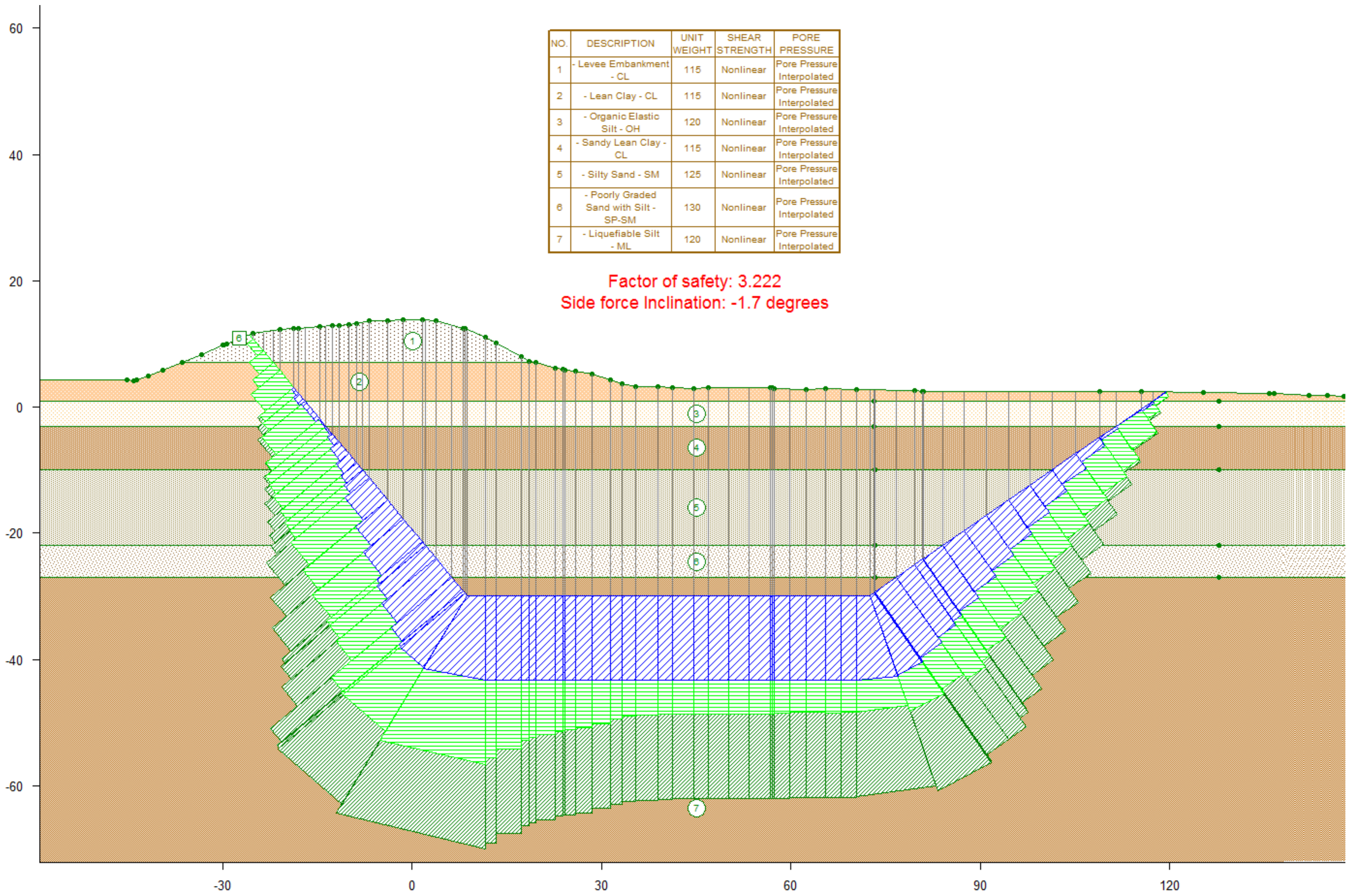
Factor of safety: 3.792
Side force Inclination: -1.99 degrees

Fig F-45(a). Lincoln Village Station 164+99– Landside – Option 3: Circular (Sr = 224 psf in liquefiable material)



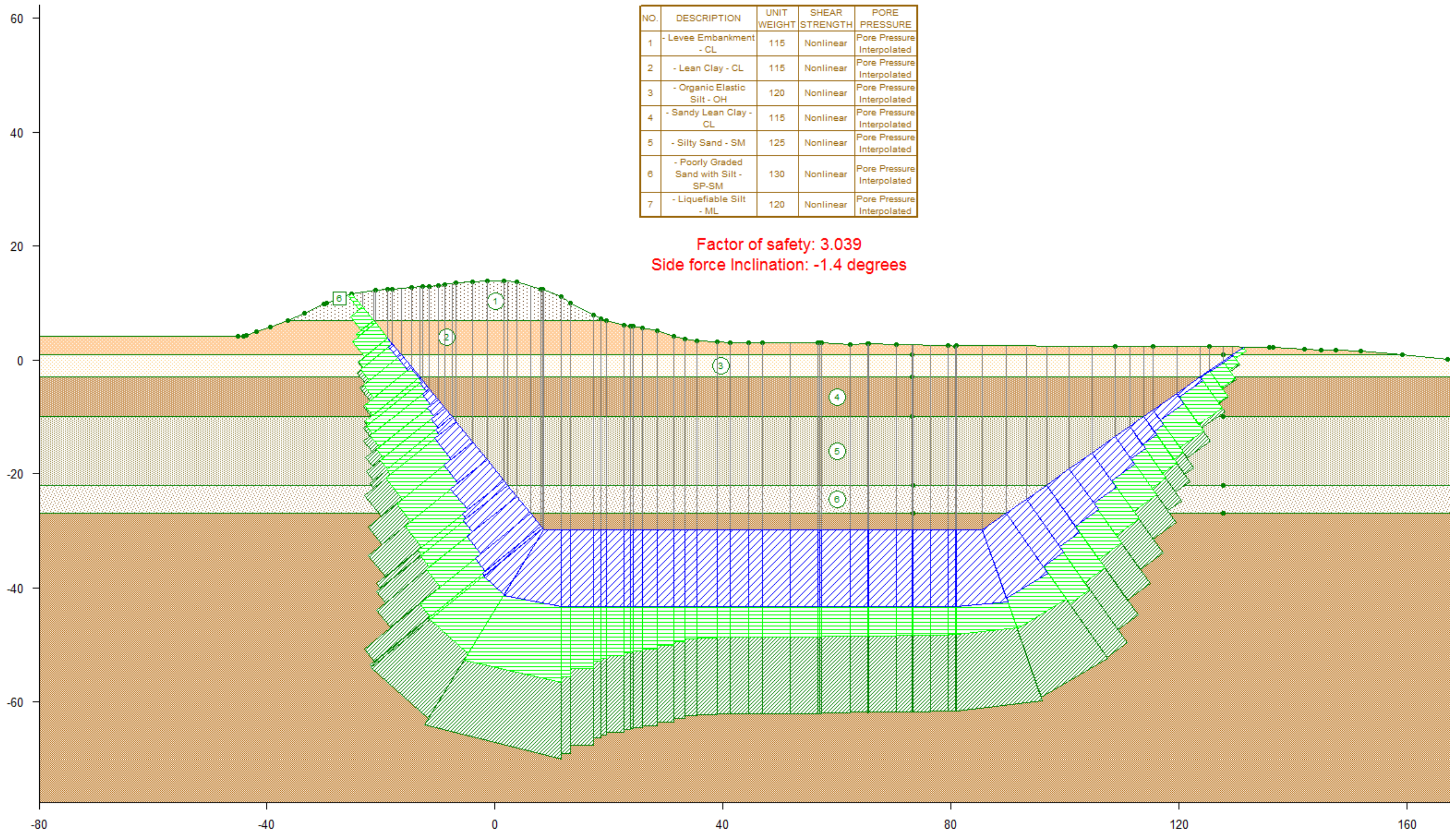
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
3	- Organic Elastic Silt - OH	120	Nonlinear	Pore Pressure Interpolated
4	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
6	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable Silt - ML	120	Nonlinear	Pore Pressure Interpolated

Fig F-45(b). Lincoln Village Station 164+99– Landside – Option 3: Circular (PHI = 3.4 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
3	- Organic Elastic Silt - OH	120	Nonlinear	Pore Pressure Interpolated
4	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
6	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable Silt - ML	120	Nonlinear	Pore Pressure Interpolated

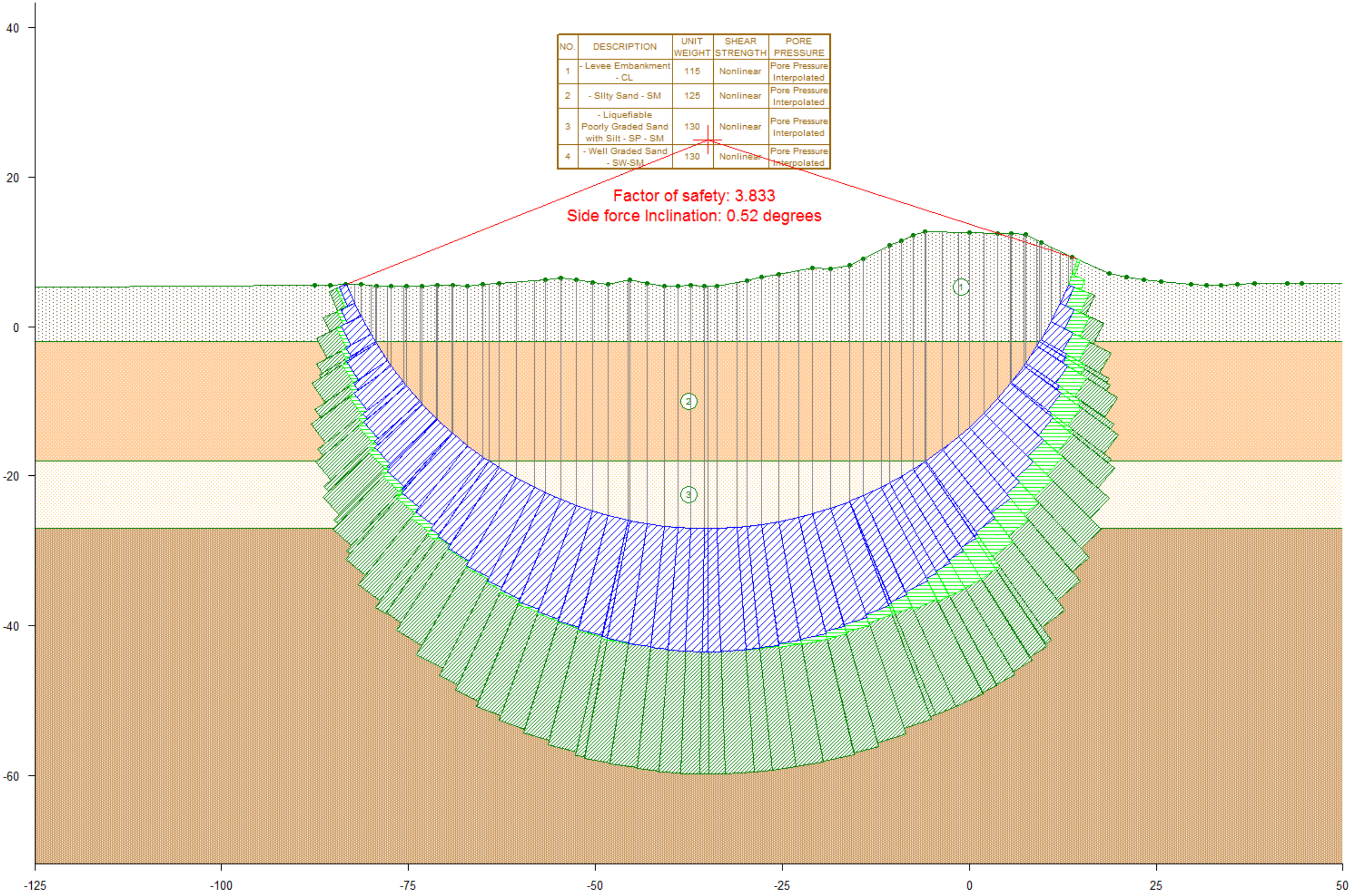
Fig F-46(a). Lincoln Village Station 164+99 – Landside – Option 4: Wedge (Sr = 224 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
3	- Organic Elastic Silt - OH	120	Nonlinear	Pore Pressure Interpolated
4	- Sandy Lean Clay - CL	115	Nonlinear	Pore Pressure Interpolated
5	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
6	- Poorly Graded Sand with Silt - SP-SM	130	Nonlinear	Pore Pressure Interpolated
7	- Liquefiable Silt - ML	120	Nonlinear	Pore Pressure Interpolated

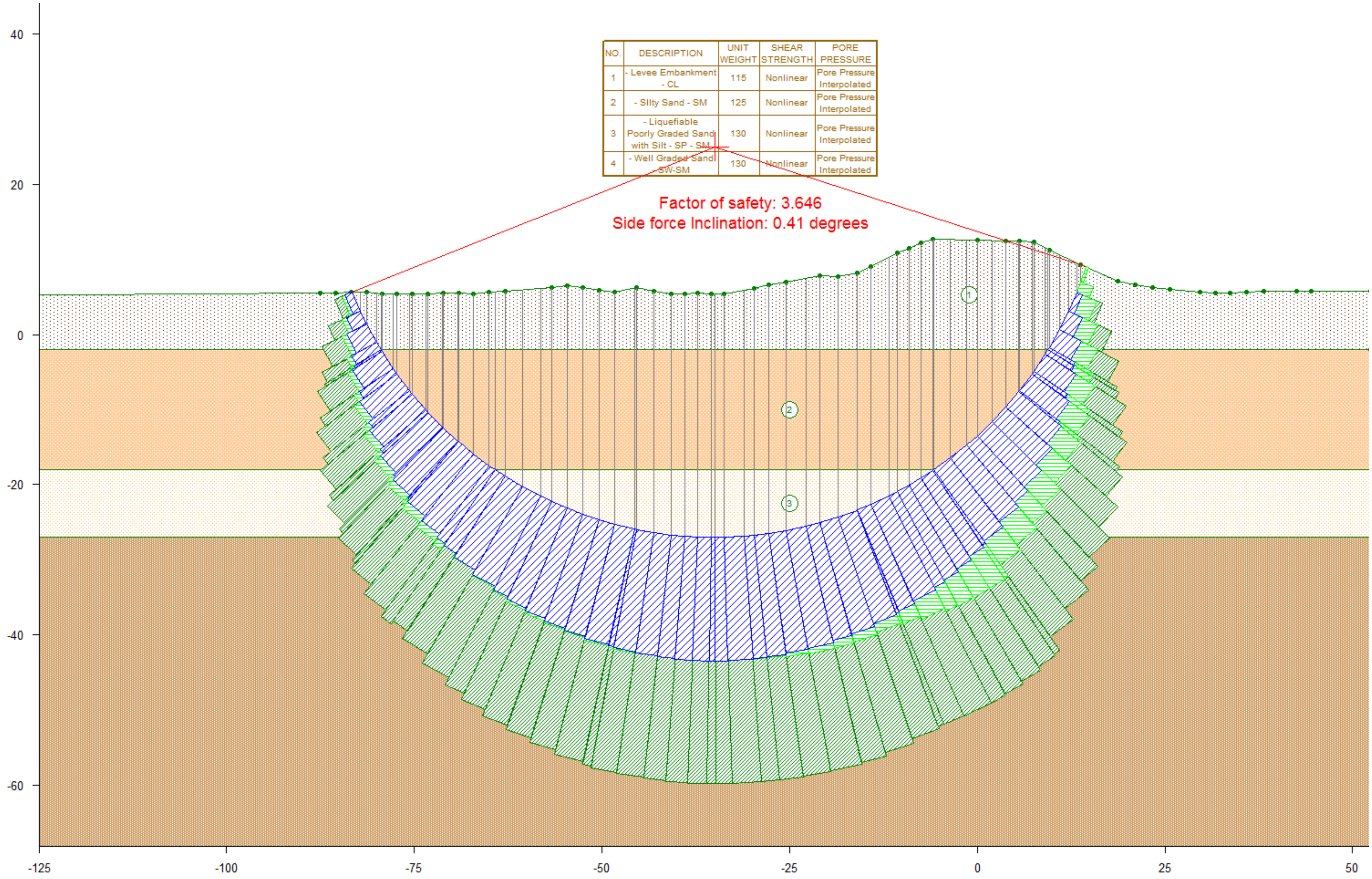
Factor of safety: 3.039
Side force Inclination: -1.4 degrees

Fig 46(b). Lincoln Village Station 164+99 – Landside – Option 4: Wedge (PHI = 3.4 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- SIlty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP - SM	130	Nonlinear	Pore Pressure Interpolated
4	- Well Graded Sand - SW-SM	130	Nonlinear	Pore Pressure Interpolated

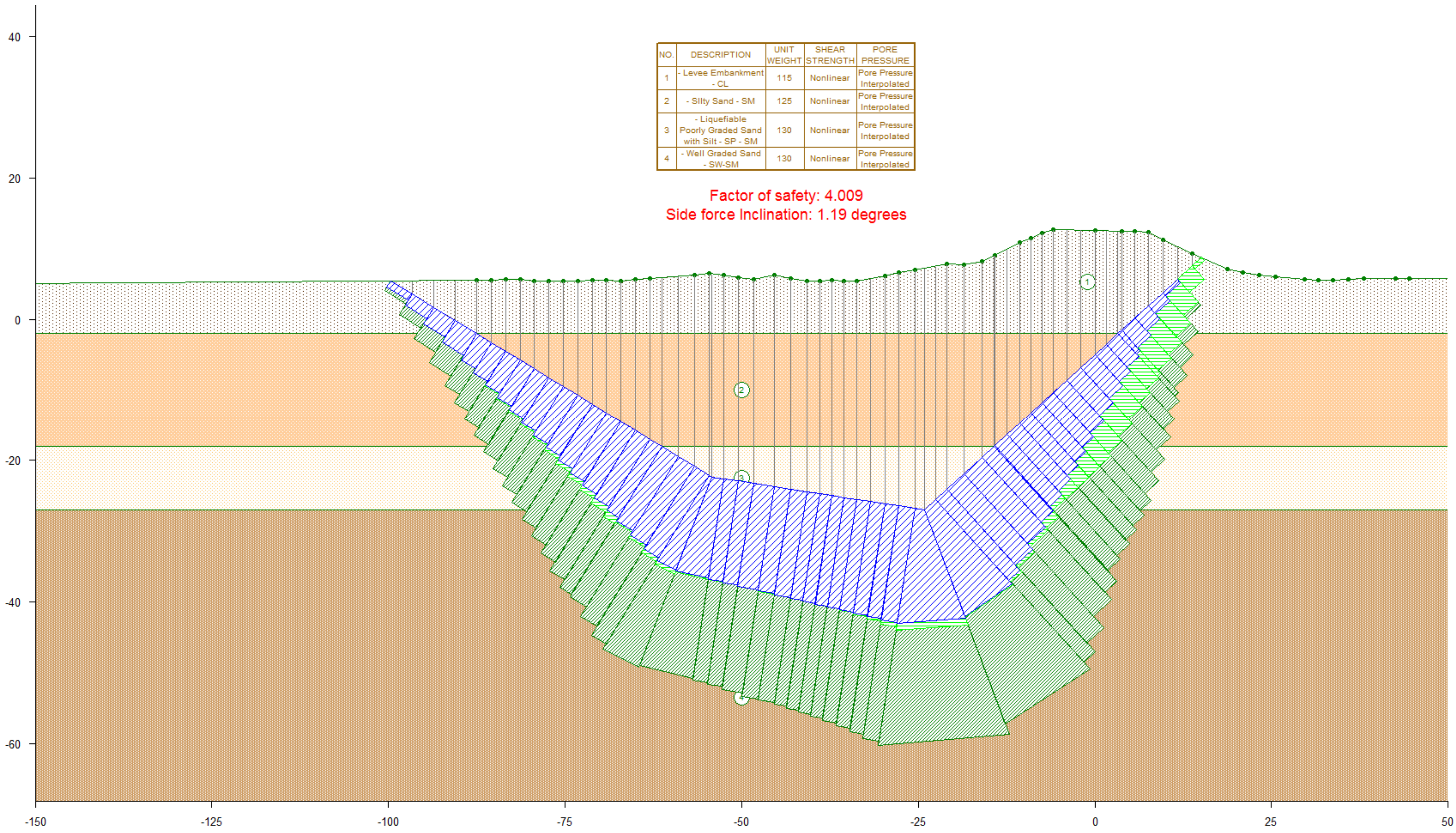
Fig F-47(a). Lincoln Village Station 201+51– Waterside – Option 1: Circular (Sr = 201 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP - SM	130	Nonlinear	Pore Pressure Interpolated
4	- Well Graded Sand - SW-SM	130	Nonlinear	Pore Pressure Interpolated

Factor of safety: 3.646
Side force Inclination: 0.41 degrees

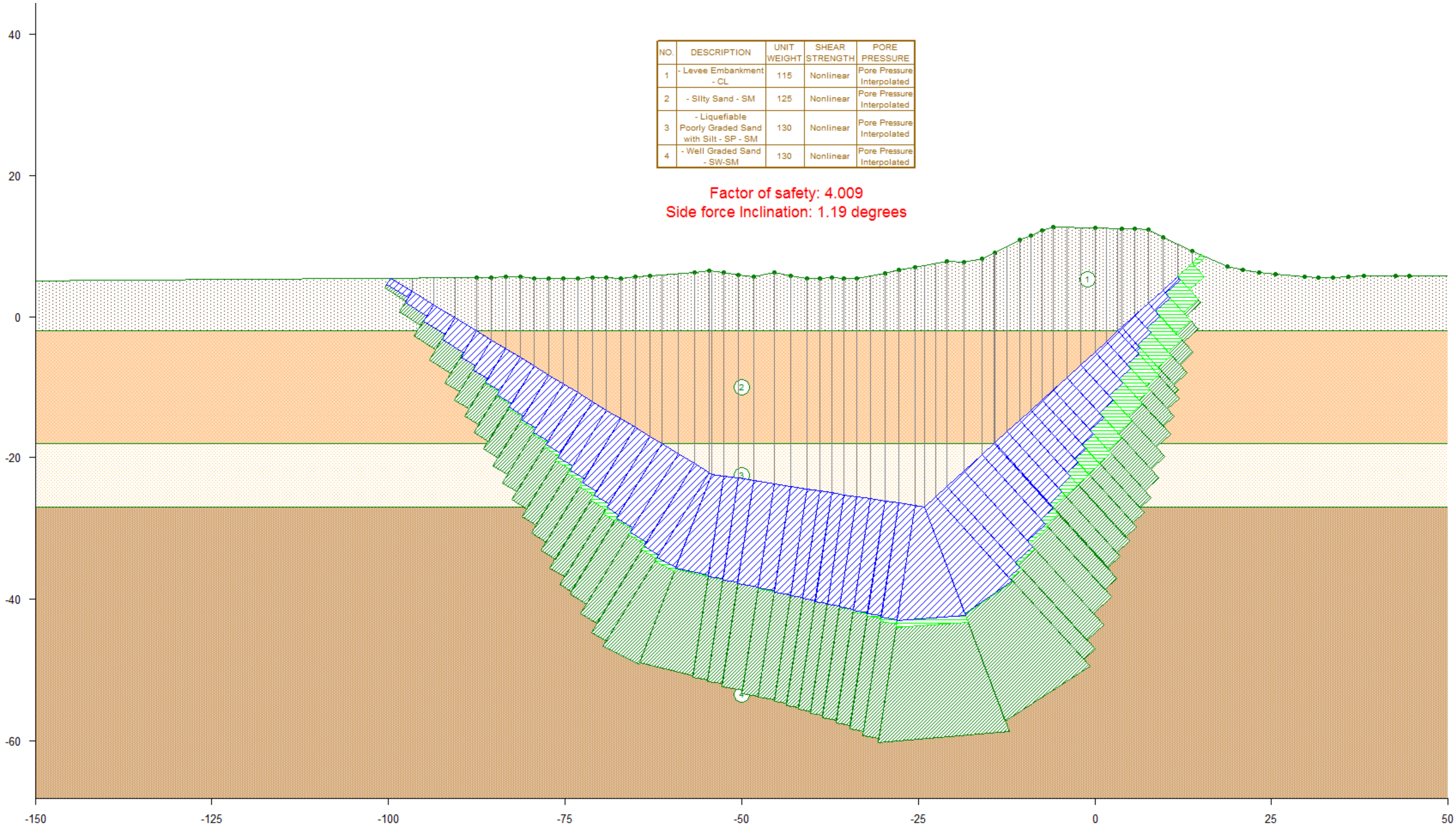
Fig 47(b). Lincoln Village Station 201+51– Waterside – Option 1: Circular (PHI = 4.7 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP - SM	130	Nonlinear	Pore Pressure Interpolated
4	- Well Graded Sand - SW-SM	130	Nonlinear	Pore Pressure Interpolated

Factor of safety: 4.009
Side force Inclination: 1.19 degrees

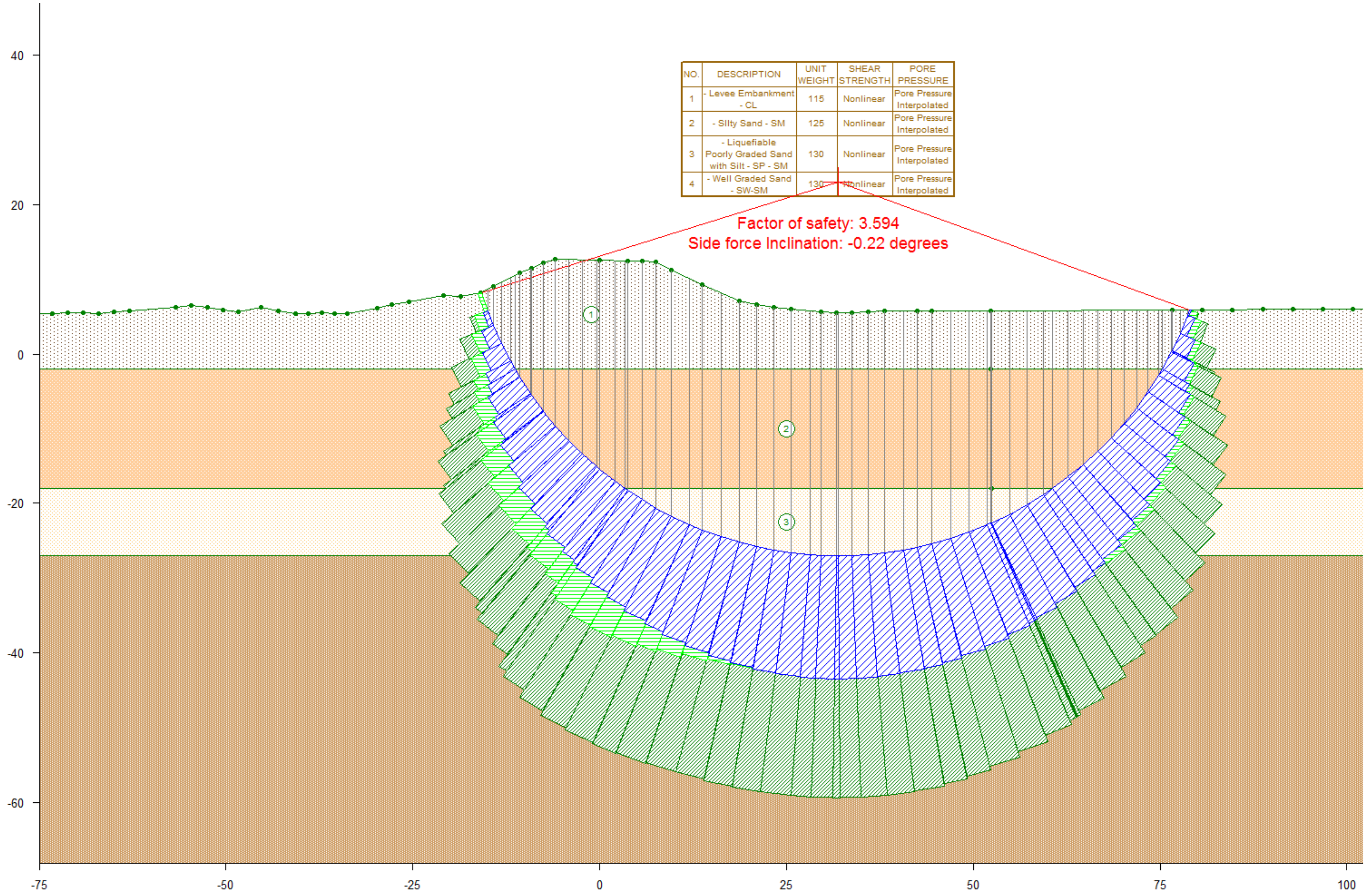
Fig F-48(a). Lincoln Village Station 201+51 – Waterside – Option 2: Wedges (Sr = 201 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- SIlty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP - SM	130	Nonlinear	Pore Pressure Interpolated
4	- Well Graded Sand - SW-SM	130	Nonlinear	Pore Pressure Interpolated

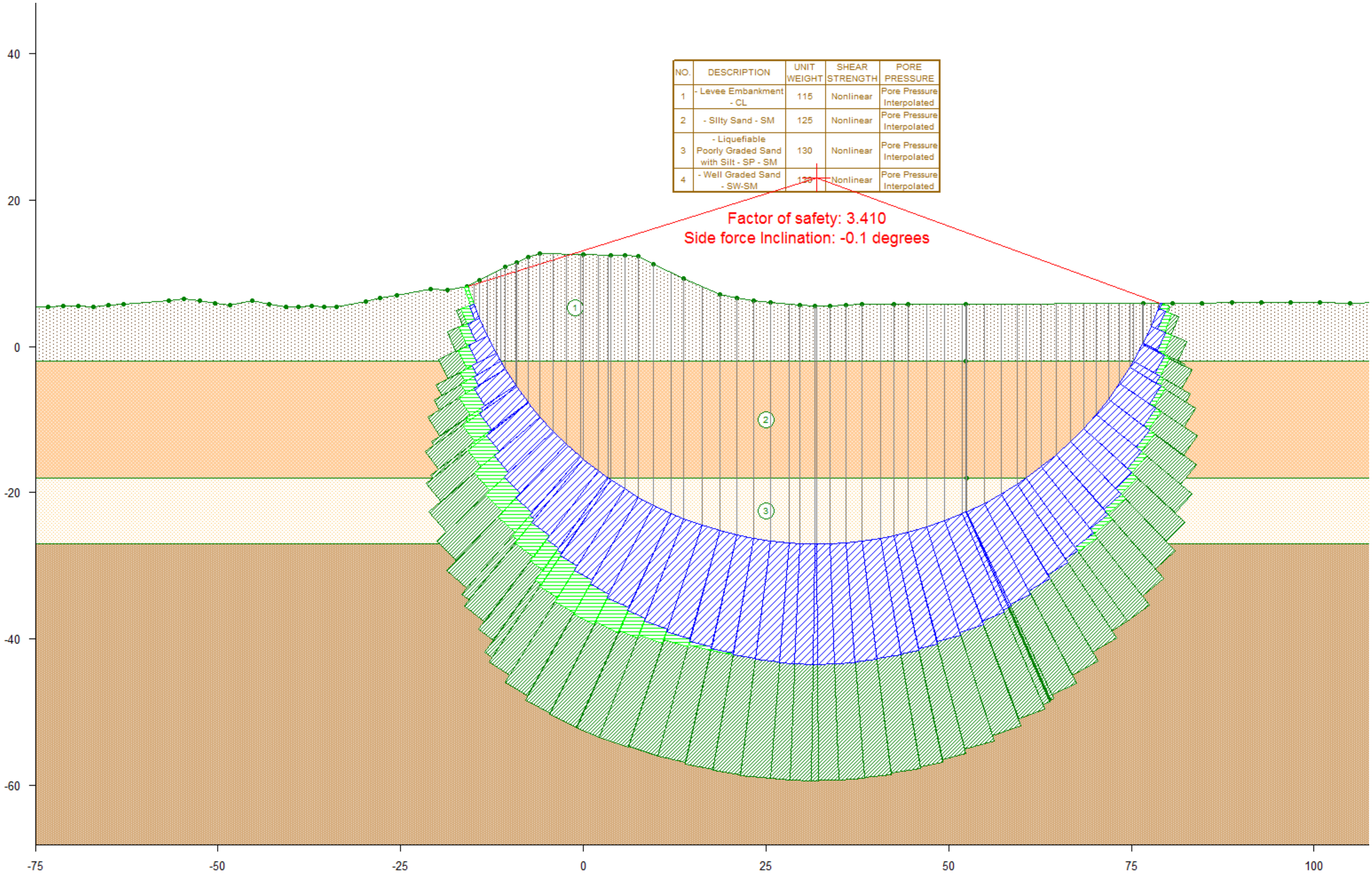
Factor of safety: 4.009
Side force Inclination: 1.19 degrees

Fig 48(b). Lincoln Village Station 201+51 – Waterside – Option 2: Wedges (PHI = 4.7 in liquefiable material)



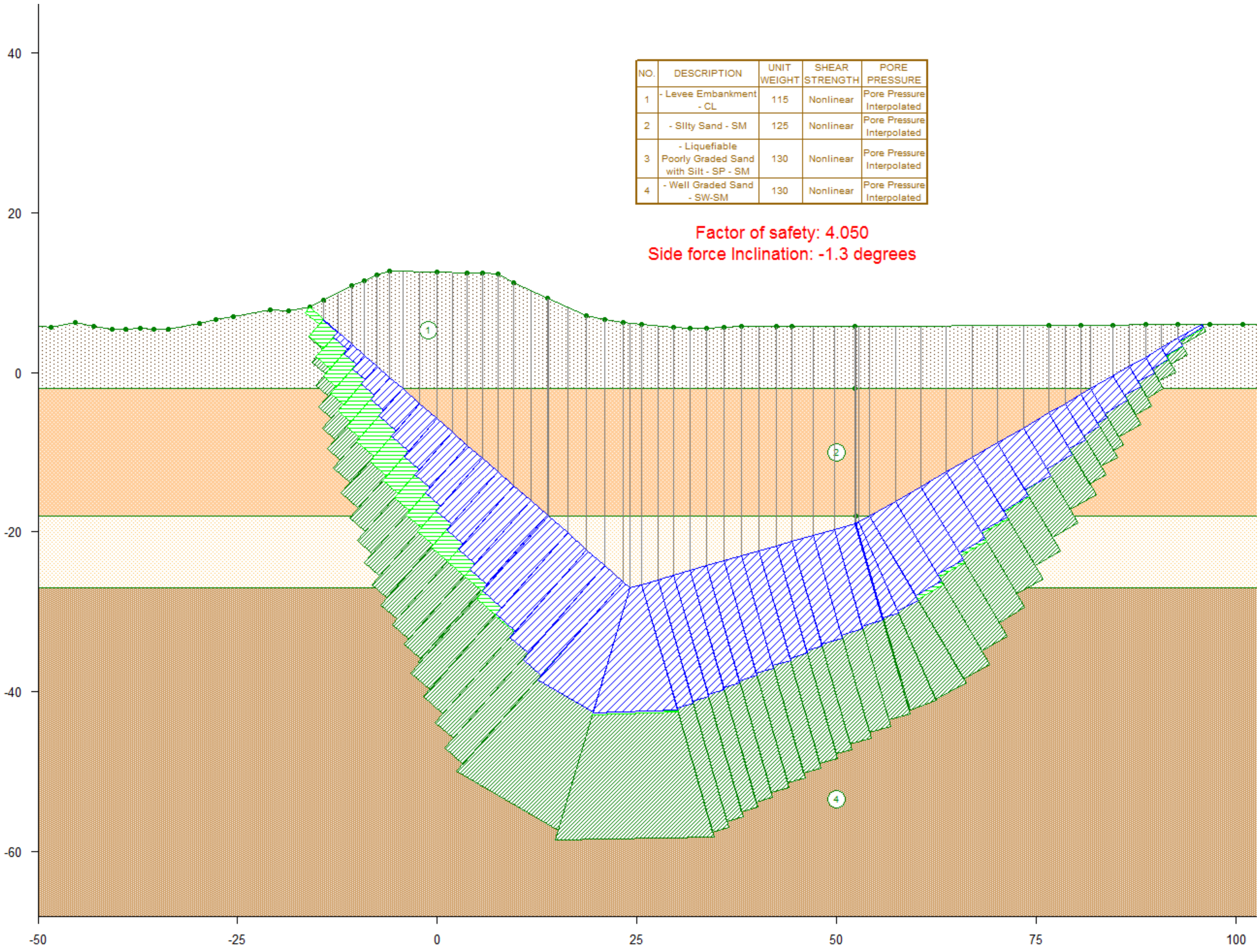
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP - SM	130	Nonlinear	Pore Pressure Interpolated
4	- Well Graded Sand - SW-SM	130	Nonlinear	Pore Pressure Interpolated

Fig F-49(a). Lincoln Village Station 201+51 – Landside – Option 3: Circular (Sr = 201 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- SIlty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP - SM	130	Nonlinear	Pore Pressure Interpolated
4	- Well Graded Sand - SW-SM	130	Nonlinear	Pore Pressure Interpolated

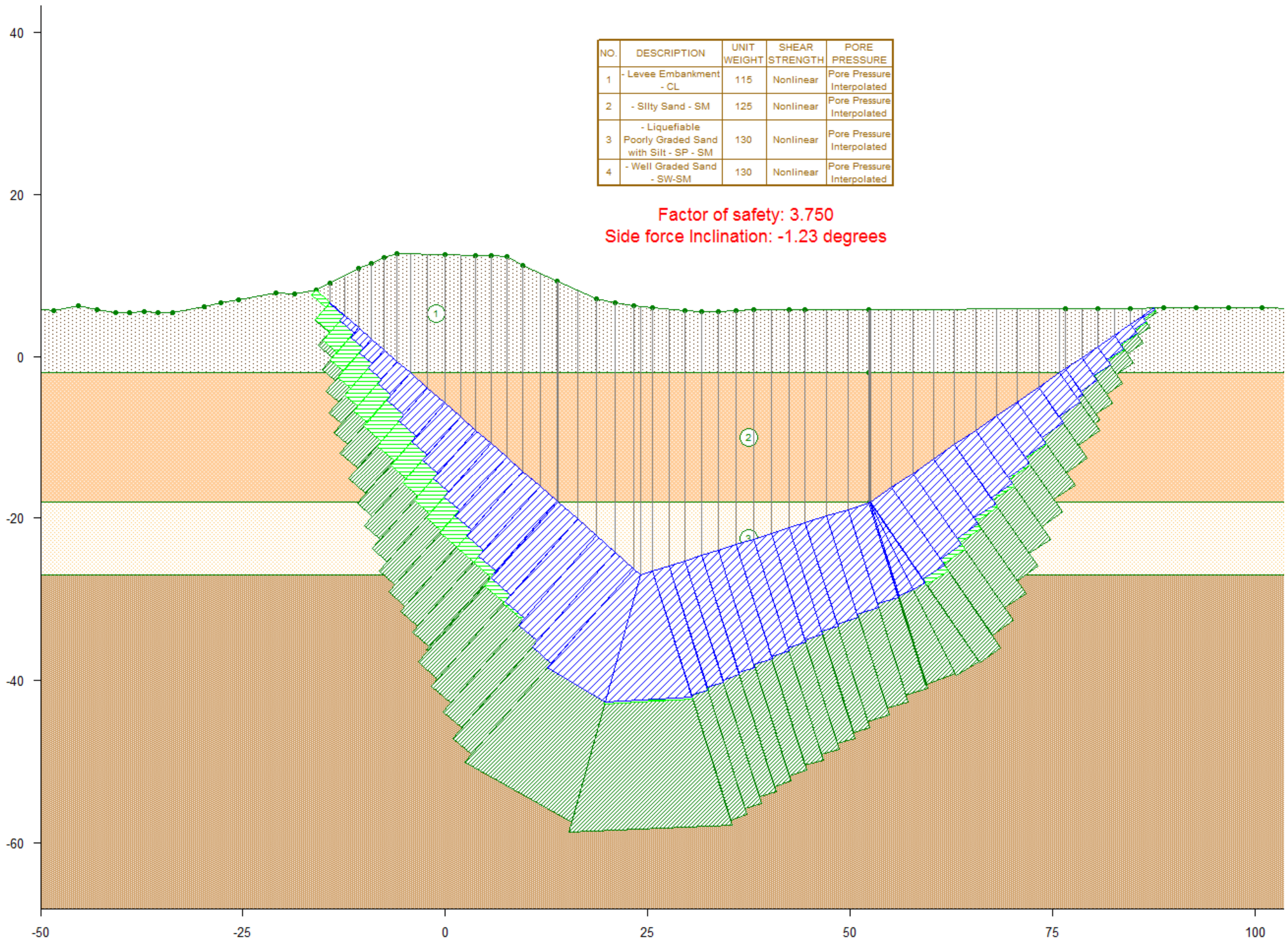
Fig 49(b). Lincoln Village Station 201+51 – Landside – Option 3: Circular (PHI = 4.7 in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP - SM	130	Nonlinear	Pore Pressure Interpolated
4	- Well Graded Sand - SW-SM	130	Nonlinear	Pore Pressure Interpolated

Factor of safety: 4.050
Side force Inclination: -1.3 degrees

Fig F-50(a). Lincoln Village Station 201+51 – Landside – Option 4: Wedge (Sr = 201 psf in liquefiable material)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	- Levee Embankment - CL	115	Nonlinear	Pore Pressure Interpolated
2	- Silty Sand - SM	125	Nonlinear	Pore Pressure Interpolated
3	- Liquefiable Poorly Graded Sand with Silt - SP - SM	130	Nonlinear	Pore Pressure Interpolated
4	- Well Graded Sand - SW-SM	130	Nonlinear	Pore Pressure Interpolated

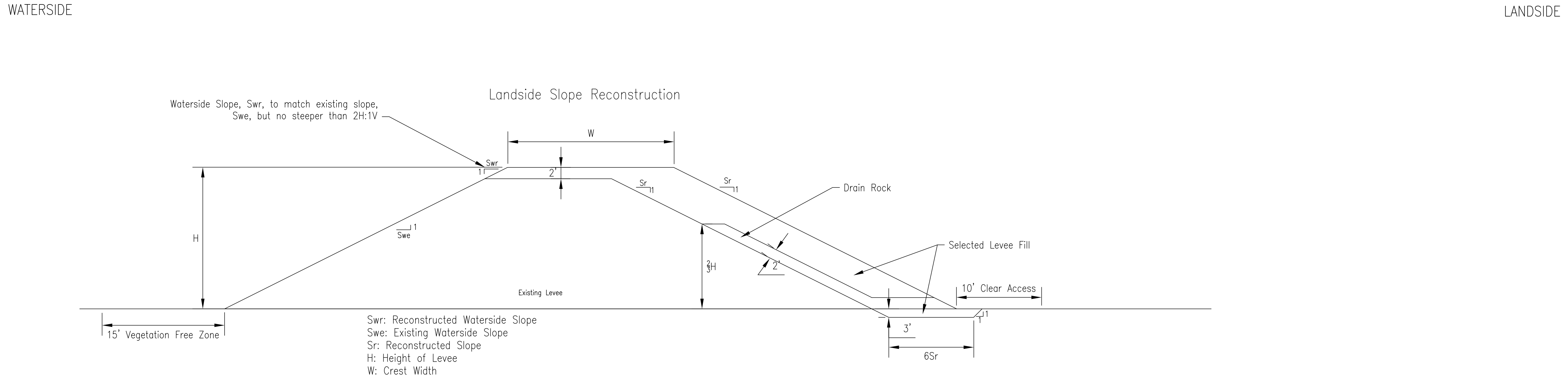
Factor of safety: 3.750
Side force Inclination: -1.23 degrees

Fig 50(b). Lincoln Village Station 201+51 – Landside – Option 4: Wedge (PHI = 4.7 in liquefiable material)

LOWER SAN JOAQUIN RIVER
FEASIBILITY STUDY

GEOTECHNICAL REPORT

ENCLOSURE E5
TEMPLATE OPTIONS FOR ASSIGNED MITIGATION MEASURES



Landside Slope Reconstruction

Swr: Reconstructed Waterside Slope
Swe: Existing Waterside Slope
Sr: Reconstructed Slope
H: Height of Levee
W: Crest Width

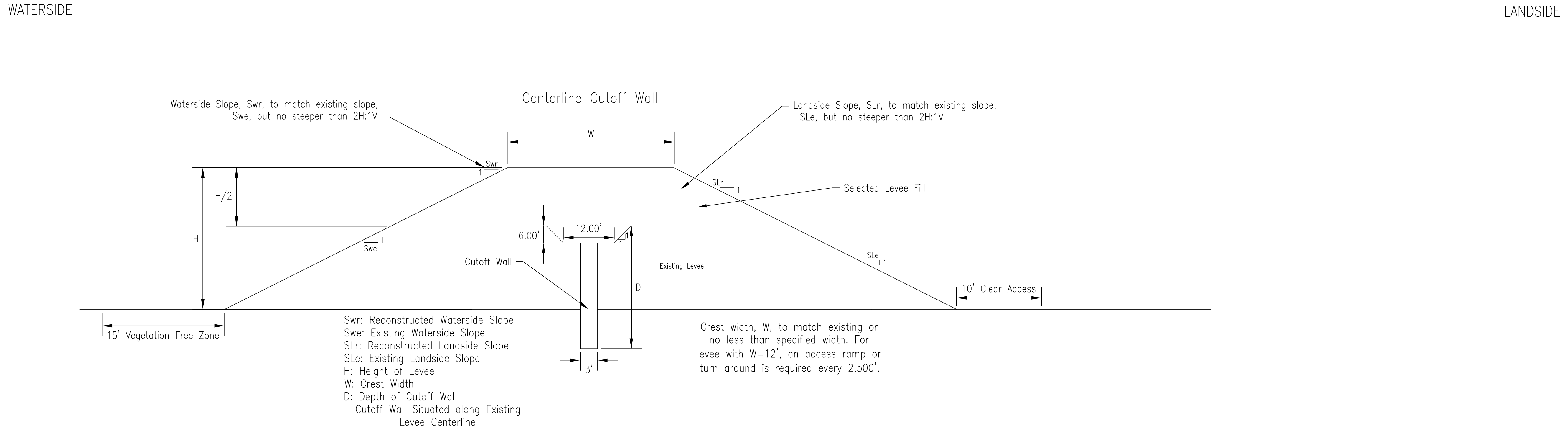
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Mitigation Measures
LANDSIDE SLOPE RECONSTRUCTION

DATE:	SCALE:	SHEET NO.
31 - July - 13	Not to Scale	1 of 11

PLOT BY: L2EDGGAJ - Jan 09, 2014 - 2:10:10pm
DRAWING: I:\CAD\Cad for Mitigation Measures\ComputationSheet1_LandsideSlopeReconstruction.10ftLSE.dwg



Swr: Reconstructed Waterside Slope
Swe: Existing Waterside Slope
SLr: Reconstructed Landside Slope
SLe: Existing Landside Slope
H: Height of Levee
W: Crest Width
D: Depth of Cutoff Wall
Cutoff Wall Situated along Existing Levee Centerline

Crest width, W, to match existing or no less than specified width. For levee with W=12', an access ramp or turn around is required every 2,500'.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

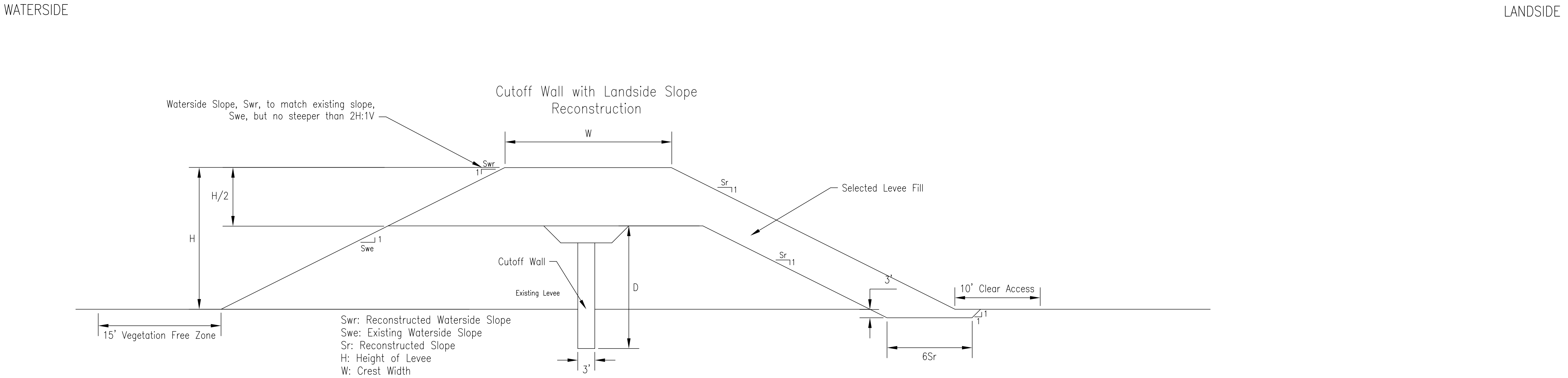
SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Mitigation Measures
CENTERLINE CUTOFF WALL

DATE:	SCALE:	SHEET NO.
31 - July - 13	Not to Scale	2 of 11

PLOT BY: L2EDGGAJ - Jan 09, 2014 - 2:09:24pm
DRAWING: I:\CAD\Cad for Mitigation Measures\ComputationSheet7_Template6.10ftLSE.dwg

Cutoff Wall with Landside Slope Reconstruction



Swr: Reconstructed Waterside Slope
Swe: Existing Waterside Slope
Sr: Reconstructed Slope
H: Height of Levee
W: Crest Width
D: Depth of Cutoff Wall
Cutoff Wall Situated along Existing Levee Centerline

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

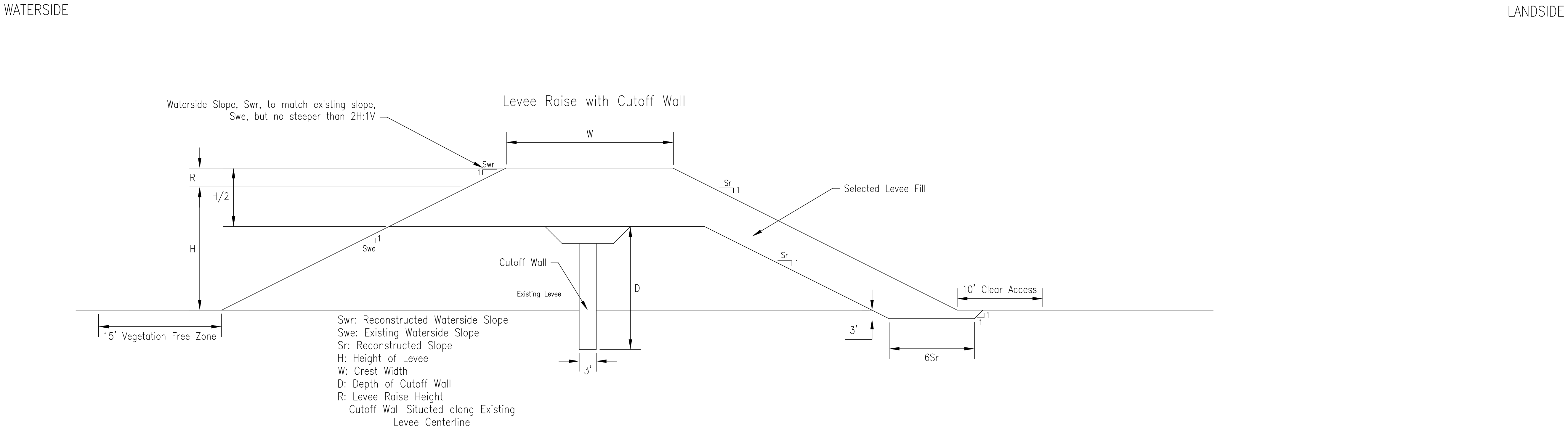
SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Mitigation Measures
CUTOFF WALL WITH LANDSIDE SLOPE RECONSTRUCTION

DATE:	SCALE:	SHEET NO.
31 - July - 13	Not to Scale	3 of 11

PLOT BY: L2EDGGAJ - Jan 09, 2014 - 2:08:27pm
DRAWING: I:\CAD\Cad for Mitigation Measures\ComputationSheet2_CutoffWallwithLandsideSLopeReconstruction.10ftLSE.dwg

Levee Raise with Cutoff Wall



Swr: Reconstructed Waterside Slope
Swe: Existing Waterside Slope
Sr: Reconstructed Slope
H: Height of Levee
W: Crest Width
D: Depth of Cutoff Wall
R: Levee Raise Height
Cutoff Wall Situated along Existing Levee Centerline

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Mitigation Measures
LEVEE RAISE WITH CUTOFF WALL

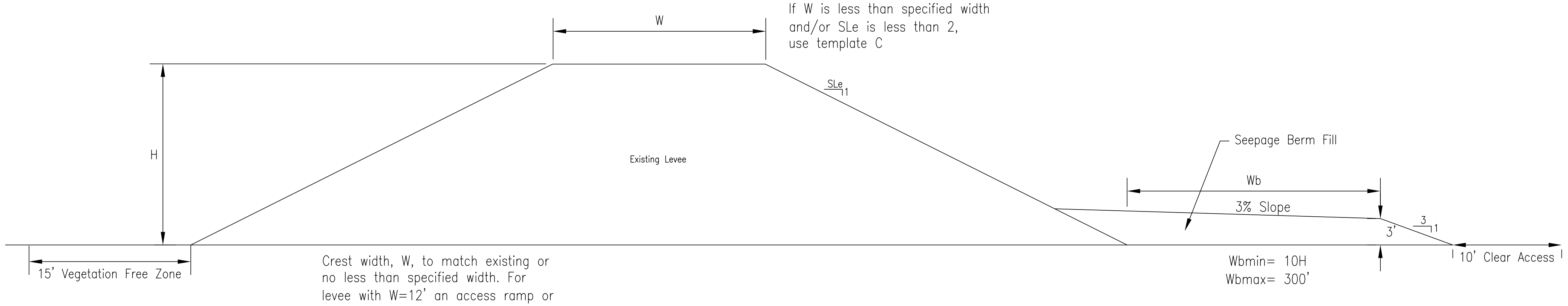
DATE:	SCALE:	SHEET NO.
31 - July - 13	Not to Scale	4 of 11

PLOT BY: L2EDGGAJ - Jan 09, 2014 - 2:06:59pm
DRAWING: I:\CAD\Cad for Mitigation Measures\ComputationSheet5_LeveeRaisewithCutoffWall.10ftLSE.dwg

WATERSIDE

LANDSIDE

Seepage Berm



If W is less than specified width
and/or SLe is less than 2,
use template C

Wbmin= 10H
Wbmax= 300'

Crest width, W, to match existing or no less than specified width. For levee with W=12' an access ramp or turn around is required every 2,500'.

SLe: Existing Landside Slope
H: Height of Levee
W: Crest Width
Wb: Width of Seepage Berm

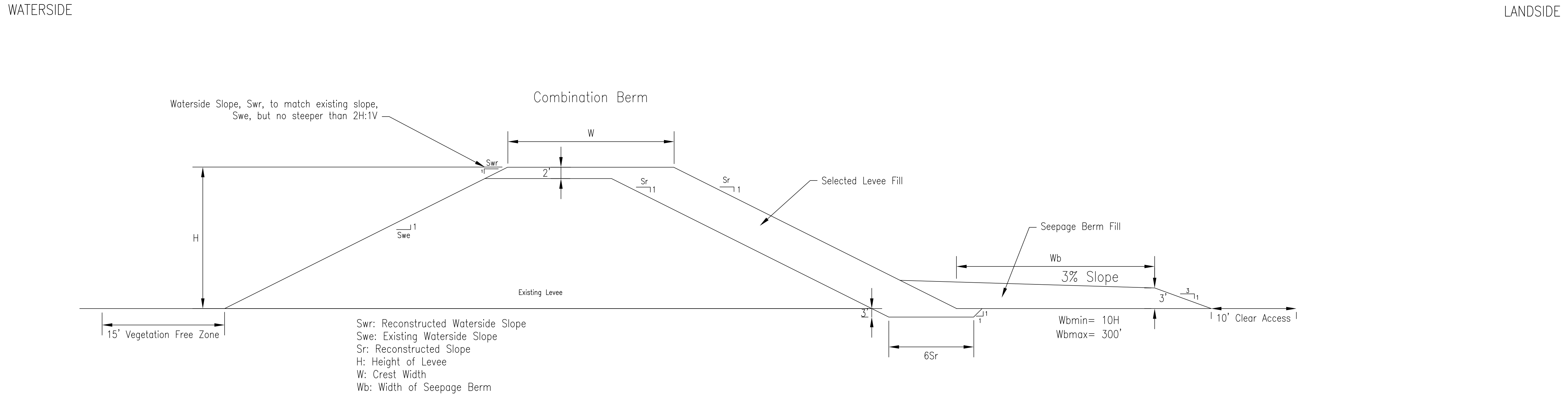
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Mitigation Measures
SEEPAGE BERM

DATE:	SCALE:	SHEET NO.
31 - July - 13	Not to Scale	5 of 11

PLOT BY: L2EDGGAJ - Jan 09, 2014 - 2:06:03pm
DRAWING: I:\CAD\Cad for Mitigation Measures\ComputationSheet8_Template3.10ftLSE.dwg



Swr: Reconstructed Waterside Slope
Swe: Existing Waterside Slope
Sr: Reconstructed Slope
H: Height of Levee
W: Crest Width
Wb: Width of Seepage Berm

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Mitigation Measures
COMBINATION BERM

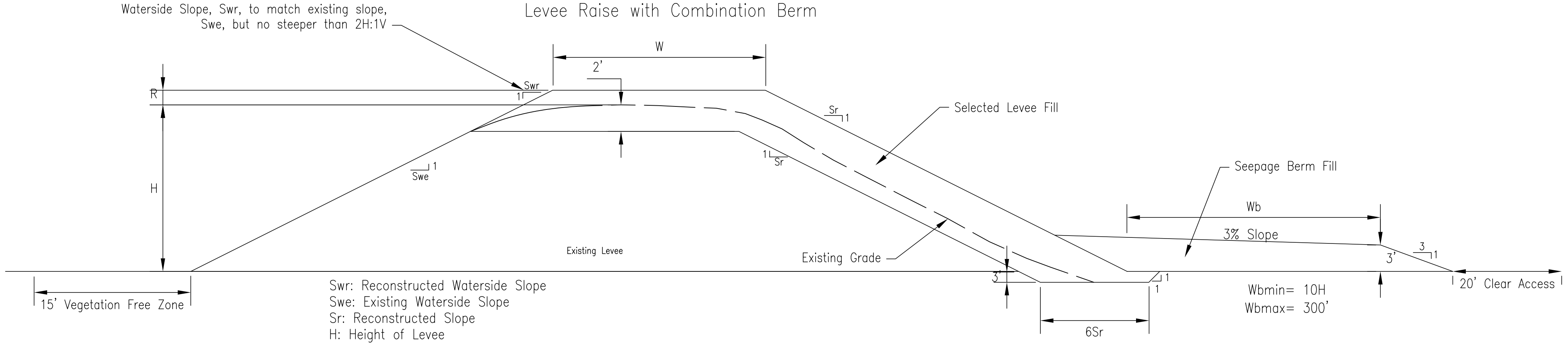
DATE:	SCALE:	SHEET NO.
31 - July - 13	Not to Scale	6 of 11

PLOT BY: L2EDGGAJ - Jan 09, 2014 - 2:05:09pm
DRAWING: I:\CAD\Cad for Mitigation Measures\ComputationSheet3_CombinationBerm.10ftLSE.dwg

WATERSIDE

LANDSIDE

Levee Raise with Combination Berm



Wbmin= 10H
Wbmax= 300'

Swr: Reconstructed Waterside Slope
Swe: Existing Waterside Slope
Sr: Reconstructed Slope
H: Height of Levee
R: Levee Raise Height
W: Crest Width
Wb: Width of Seepage Berm

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

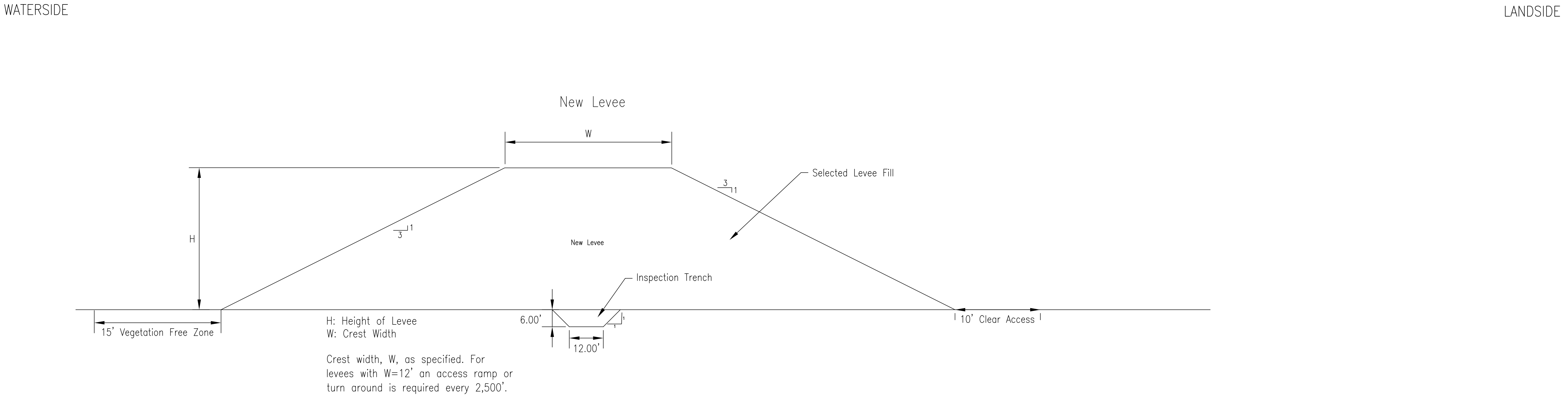
SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Mitigation Measures
LEVEE RAISE WITH COMBINATION BERM

DATE:	SCALE:	SHEET NO.
31 - July - 13	Not to Scale	7 of 11

PLOT BY: L2EDGGAJ - Jan 09, 2014 - 2:03:35pm
DRAWING: I:\CAD\Cad for Mitigation Measures\ComputationSheet6_LeveeRaisewithCombinationBerm.10ftLSE.dwg

PLOT BY: L2EDGGAJ - Jan 09, 2014 - 2:02:40pm
DRAWING: I:\CAD\Cad for Mitigation Measures\ComputationSheet9_Template7.10ftLSE.dwg



H: Height of Levee
W: Crest Width

Crest width, W, as specified. For levees with W=12' an access ramp or turn around is required every 2,500'.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

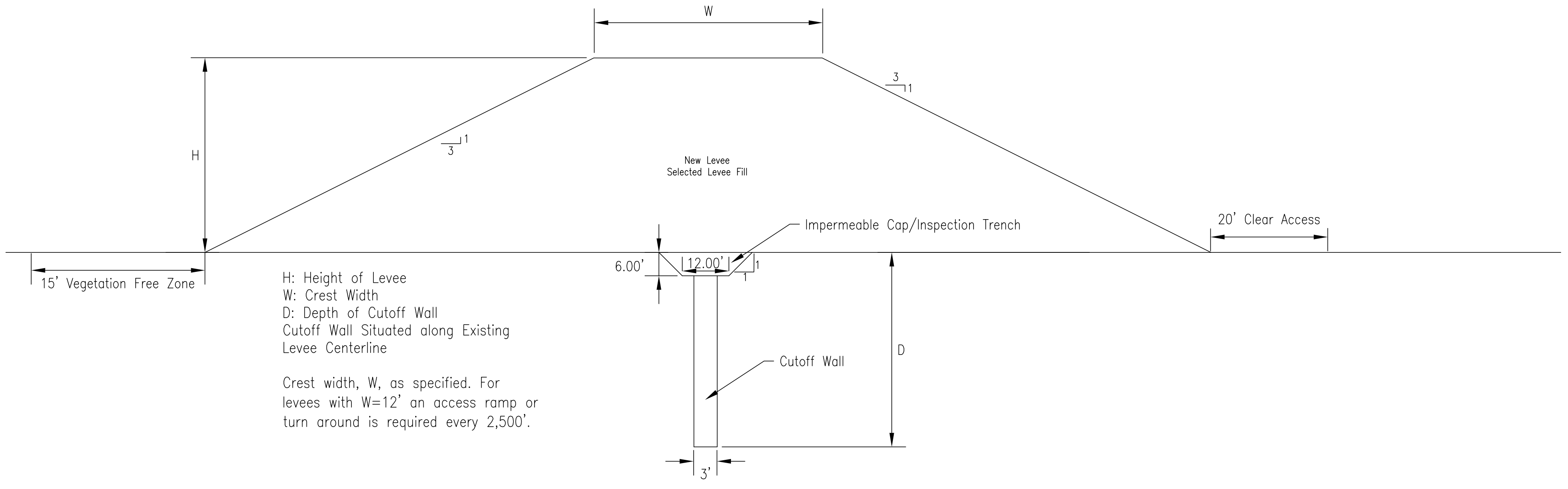
Lower Jan Joaquin
Feasibility Study
Mitigation Measures
NEW LEVEE

DATE:	SCALE:	SHEET NO.
31 - July - 13	Not to Scale	8 of 11

WATERSIDE

LANDSIDE

New Levee w/Cutoff Wall



H: Height of Levee
W: Crest Width
D: Depth of Cutoff Wall
Cutoff Wall Situated along Existing
Levee Centerline

Crest width, W, as specified. For
levees with W=12' an access ramp or
turn around is required every 2,500'.

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Mitigation Measures
NEW LEVEE W/CUTOFF WALL

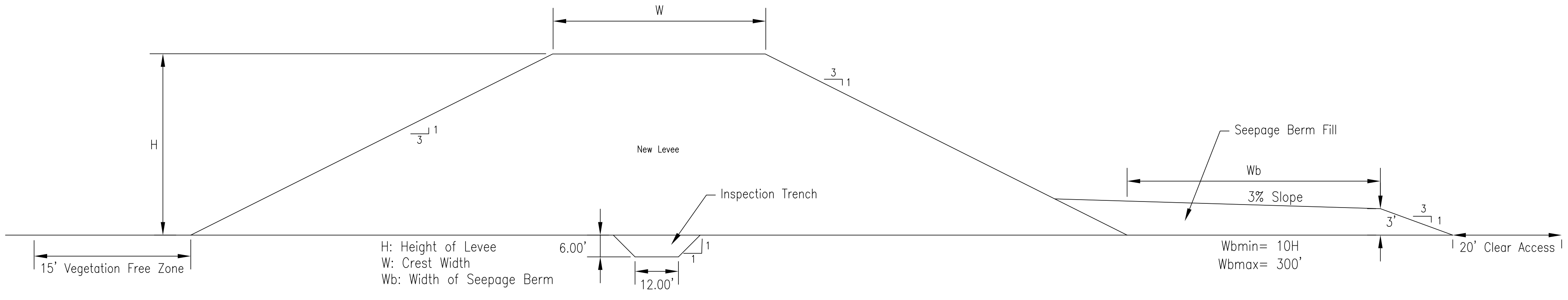
DATE:	SCALE:	SHEET NO.
31 - July - 13	Not to Scale	9 of 11

PLOT BY: L2EDGGAJ - Jan 09, 2014 - 2:01:37pm
DRAWING: I:\CAD\Cad for Mitigation Measures\ComputationSheet10_Template8.dwg

WATERSIDE

LANDSIDE

New Levee with Seepage Berm



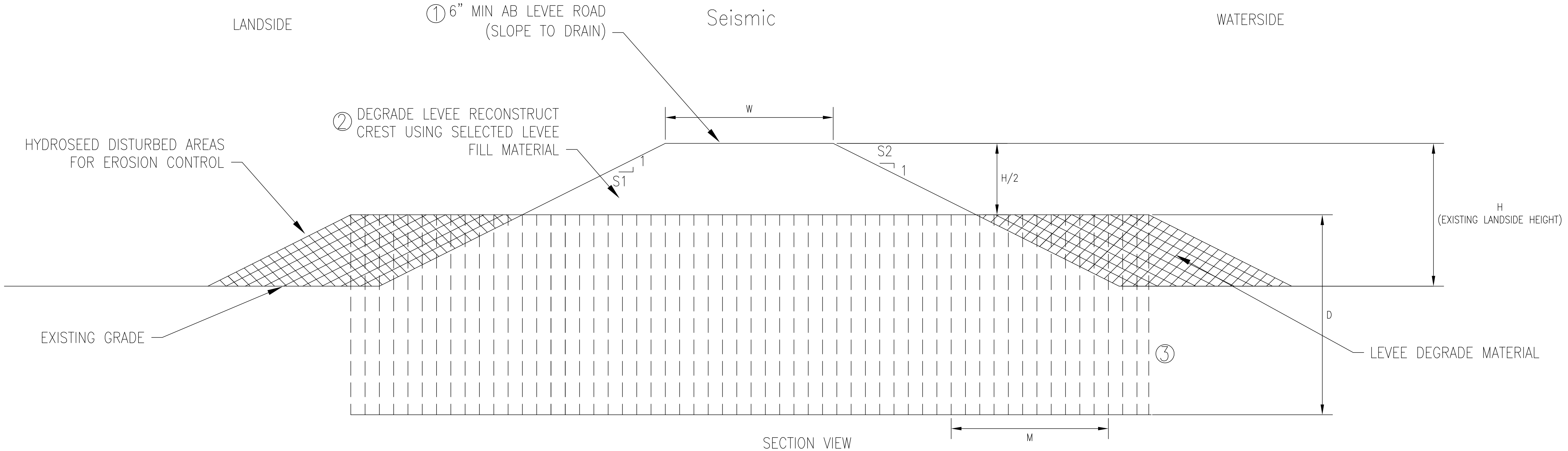
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Mitigation Measures
NEW LEVEE WITH SEEPAGE BERM

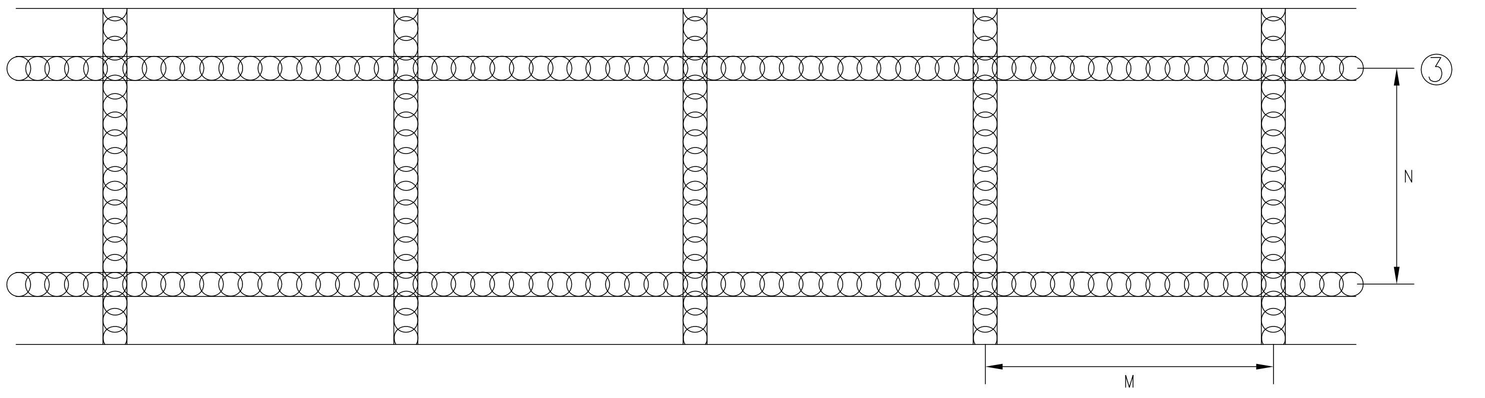
DATE:	SCALE:	SHEET NO.
31 - July - 13	Not to Scale	10of 11

PLOT BY: L2EDGGAJ - Jan 09, 2014 - 2:00:30pm
DRAWING: I:\CAD\Cad for Mitigation Measures\Computation\Sheet4_NewLeveewithSeepageBerm.dwg



SECTION VIEW

S1: Landside Slope
S2: Waterside Slope
M: DSM Cell Transverse Spacing
N: DSM Cell Longitudinal Spacing
D: DSM Wall Depth
W: Crest Width
1: Levee Road
2: Select Levee Fill
3: DSM Wall



PLAN VIEW

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

SACRAMENTO CALIFORNIA

Lower Jan Joaquin
Feasibility Study
Mitigation Measures
SEISMIC

DATE: 31 - July - 13
SCALE: Not to Scale
SHEET NO. 11of 11

PLOT BY: L2EDGGAJ - Jan 09, 2014 - 2:19:28pm
DRAWING: I:\CAD\Cad for Mitigation Measures\PCET_ULE - SeismicRem1.dwg

LOWER SAN JOAQUIN RIVER
FEASIBILITY STUDY

GEOTECHNICAL REPORT

ENCLOSURE E6
MEETING MINUTES FOR
EXPERT ELICITATION

American River Common Features GRR Geotechnical Expert Elicitation

DAY 1

Project: American River Common Features GRR
Date: Wednesday, June 17th, 2009
8:00 am to 5:00 pm
USACE – Sacramento District,
Room 1424
Facilitator: Michael Ramsbotham (MDR), USACE
Meeting Called By: Mary Perlea (MPP), USACE, Project Geotechnical Engineer

ATTENDEES
See Attendance Record (to be attached at end of finalized meeting minutes)

MEETING MINUTES

Call to order at 8:15 am
The meeting was called to order at approximately 8:15 am by the Facilitator, Michael Ramsbotham (MDR).

Introductions and Sign-In
A few minutes was spent on introductions and attendees signing the attendance list.

Identify EOE Team / Affiliation and Observers / Participants
The following attendees were recognized as Panel Members, meaning they would be voting on various items during this 2-day meeting:

- Paul Devereux, RD1000
- Les Harder, HDR, Inc.
- Mike Inamine (Mike I.), DWR
- Ed Ketchum, US Army Corps of Engineers
- Steve Mahnke, DWR
- Henri Mulder, US Army Corps of Engineers
- Mike Nolan (Mike N.), Consultant to City of Sacramento Utilities Department
- Tom Smith, Ayres Associates
- Mohsen Tovana, US Army Corps of Engineers

The following observers participated at the meeting

- Peter Ghelfi, SAFCA
- Jesse Hogan, US Army Corps of Engineers
- Dan Tibbitts, US Army Corps of Engineers
- Kevin Knuuti, US Army Corps of Engineers
- Jeff Taylor, US Army Corps of Engineers
- Joe Sciadrone, US Army Corps of Engineers

Introductory Comments by Attendees
Mary Perlea opened the meeting by requesting introductory comments from the audience.

Kevin Knutti thanked everyone for their time in being there. He stated he realized everyone's schedules are busy and really appreciates them making time for this meeting. Dan Tibbitts concurred with Kevin's comments and advised he hopes this meeting will bring about resolution on various tasks in which there is currently little-to none criteria in setting up judgment of the levee performance curves.

Pete Ghelfi commented that he is attending the meeting as an observer and will try to play that role. He feels it is important to be able to see within the black box a little bit and welcomes the opportunity to work together.

Kevin added that the Corps' Sacramento District is taking the lead for the Corps on a couple of items. It is recognized that this is one area where the Corps' policy has problems. While this issue is recognized by some, it will allow further discussion with others within the Corps to begin refining the Corps' policy.

Ed Ketchum concurred with Kevin's comment. He included the statement that this is very important work and the values that come out of this meeting will affect the national economic plan. This has a huge influence on Benefit/Cost ratio and everything else.

This part of the meeting concluded with Steve Mahnke noting that there is a partnering of many of the attendees, so it is very important to see this issue from the Corps' perspective.

Introductory Comments by Facilitator

MDR led the group in an informal discussion regarding the different meeting elements. Those discussion points included:

The Purpose/Expected Outcome of the 2-Day meeting:

- The purpose is to assist the Corps in development of the geotechnical judgment curves for the American River Common Features GRR (ARCF-GRR) project
- MDR added the judgment curves impact Economics and inquired as to the expected outcome. It was noted that Melanie Garland will provide meeting minutes of the 2-day discussion and Mary will provide a report that captures the summary, conclusions and recommendations. In addition, Mary will include revised judgment and fragility curves for the ARCF-GRR. The outcome of these discussions may lead to policy change, new Corps' guidance and/or a revised ETL.

Rules of Engagement

- Directions to accommodations was provided
- If a break is needed, the group was encouraged to suggest it
- MDR stated the discussions should be informal as he wanted everyone to be engaged and provide frank input freely
- MDR added that he hoped to see general information to final analysis and specific circumstances with the American River
- Side bar conversations were to be minimal
- Avoidance of "group" think and independent voice of opinions was supported

Review of Agenda / Scope

A brief review of the agenda and scope of discussion was held

Questions and Answers

- MDR led the attendees in an overall questions and answers period to familiarize themselves more on the general topic at hand. This was done to gain a better understanding of the role they were asked to play. The following discussion took place:

> Seepage and stability was brought up. Mary clarified they are only discussing judgment curves here as the seepage and stability components were straightforward. Mary added that the intent was to discuss poor performance first and then see if we can come to conclusion on chances of failure. Ed feels the seepage and stability will need to be discussed as well. Mary responded that they will not be left out; however, they will not be judged in this forum. She iterated that the final will include all of them, but the geotechnical analysis is already known and is not based on subjective discussion. Mary's scope is to decide on judgment curves first.
>
> Les Harder commented that he assumed "failure" would be clarified. Mary responded by saying that "failure" equals poor performance or breach. MDR added that this may continue to be refined during the meeting. If we are coming up with judgment curves on vegetation, encroachment, etc., it will depend on how robust the levees are. They may have a different set of curves for the levee based on this and seepage/stability. Mary stated information will be provided. Judgment (erosion, penetration, vegetation, encroachment) is what Mary needed the full panel for. The others have already been decided. Then, there is likelihood of failure being discussed.
>
> In the geotechnical analysis that includes stability, seepage and judgment, Mike Nolan inquired if judgment is weighted the same as seepage and stability, or if its weighting can be reduced in the risk-based / FDA model. MDR responded that the hope is to get into this

more in depth as they look into poor performance after taking a look from the judgment perspective. It was noted that FDA uses the total combined curve. Ed stated weighting will likely be based on folks' past experience. Pete added that in this forum, the group was hoping to make a judgment on judgment.

Mary discussed some of the work that had already been done by URS in regards to Erosion Analysis. She conveyed that she did not believe the Corps provided URS with the information needed for the evaluation, so erosion analysis will likely need to be revised. URS identified the highly erodible area which was considered by Mary on the initial judgment curves.

Ed asked if recommendations could be made to Headquarters (HQ) based on this meeting. Mary answered by stating this is the first time this has been done. The conclusion will be included in the CF GRR study that will be provided to the Headquarters, but the scope is not to provide recommendation to the Headquarter policies.

Paul Devereaux questioned whether the current procedure was over predicting or under predicting failure? Mary advised she provided all preliminary curves already. The curves will be revised based on the panel recommendation.

Henri Mulder asked about the current guidance ETL. Dan responded by advising him yes, the current guidance ETL 1110-2-556 was being used, however, it is only one paragraph regarding the judgment fragility curves and not much guidance provided. It is expected the guidance ETL will be revised, but in the meantime, that was part of the purpose for the 2-day meeting.

- At this point, MDR noted the discussion had gotten off track and reminded the group, that while flood fighting had been a huge discussion, the purpose was to resolve the judgment curve issue. This effort that includes erosion, vegetation, penetration and encroachment was a difference that he had seen in previous efforts. As far as he could tell, it had never been done consistently. In his opinion, whoever analyzes the "without project" conditions needs to be the same person to analyze it for "with project".

Mike Inamine questioned why the group wasn't just looking at failure and what in the FDA model came close to this. Ed responded it has a national impact so the benefits from this project will be for others as well. Mary added that poor performance is indicative of a weaker levee for future events and may lead to levee failure. While it may not be a "failure", it has the propensity for failure and damages. Mike I. countered that they are looking at a fuzzy area that would result in a breach or such poor performance that it would result to what?

Les added to combine them equally as the curves should be scaled the same. Mike I. commented that looking at poor performance as definition while Mike N. advised performance to him is no inundation if that is what is being used for economic analysis in the Corps' FDA model.

Mary asserted that for now we are looking at existing conditions of the levee as performance, however, Henri and Mike N. both felt the group should be looking at both.

Pete suggested displaying a probability curve with seepage and stability to reflect how judgment affects it by applying those components. In regards to economic analysis, he queried as to whether or not it needed to be limited. Les agreed, however, added that they should be applied under the same criteria or at least comparable in terms to what "failure" means.

MDR responded by explaining that is partly the way it has been done based on the current guidance and trying to be consistent nationwide. He conveyed that what is happening in the economic study is determining what the benefits are versus the cost. He further went on to express that he felt it was a mistake to take economic criteria and applying it to performance. He added that, in his mind, to get to the true level of protection, a different approach should be taken.

Background Presentation / Project Overview - MPP
Mary provided the team with a presentation of the ARCF-GRR with a description of the three primary areas: Natomas Basin, American River North Basin, and American River South Basin. These three primary areas were analyzed by URS who determined the critical reaches considering seepage, stability, and erosion based on 100-year high water elevation. The map Mary showed the group had seepage, stability, erosion and height deficiency plotted in reaches in the three different primary areas and reflected the areas that ARCF-GRR encompasses. Mary added that based on another URS analysis, for a 200-year event (not displayed), erosion was everywhere.

Mary reported that eventually, the ARCF-GRR team may breakout the Natomas Basin from the other basins due to priority.

It was noted that the damages shown on the map are determined based on a deterministic analysis considering a minimum factor of safety 1.4 for stability and 1.6 (gradient higher than 0.5) for under seepage for the 100-year flood event. The deterministic analysis was conducted determining the weakest cross sections within a reach considering the worst geotechnical parameters. Geotechnical R&U analysis made for the index points (as selected by the deterministic analysis as the critical points on a reach) uses the average values (or the most credible values) applying a coefficient of variation based on statistical analysis. The R&U determine the risk of failure due to stability and under seepage applying the coefficients of variation around the mean values considering the factor of safety of 1.

Mary walked the group through a specific sample to illustrate the engineering R&U fragility curves determined by seepage and stability R&U analysis versus the judgmental portion of the R&Y combined fragility curves.
Ed inquired if a variation across the levee for vegetation and encroachment were being looked at the same as is done for under seepage and stability. Mary responded no, that for the judgment curve, items are looked at within the reach where for the stability and under seepage it was considered the critical cross section representing a reach, with average parameters and their coefficient of variation. Ed countered by asking if they should look at the average condition along the reach. Mary answered by advising they have some index points where seepage and stability are not an issue, however, vegetation and encroachment are. Ed replied by asking if the integral of the area underneath is what is taken into consideration. Mary confirmed. She added that she will describe the specifics of each reach when they get to each reach section.

Most Likely Failure Modes Identification – Team
This part of the meeting consisted of the team being polled in relation to identifying what causes a levee to go into failure mode, that is, what causes levees to fail or breach. Nineteen different causes were identified as listed at the end of this section.

After the various factors where identified, the panel was asked to vote which ones are most likely to cause a levee to fail. The number listed to the side reflects the number of votes it received during this particular exercise in relation to their view of its significance to causing a failure mode.

- Under seepage – piping / stability - 9
- Overtopping - 4
- Stability - 6
- Erosion – waterside, scour - 7
- Through – seepage (internal erosion) - 4
- Closure structures - 0
- Penetrations through foundation - 1
- Seepage through animal holes – 6
- Uprooted trees - 0
- Human intervention - 0
- Seismic – overtopping - 0
- Seismic – seepage - 0
- Seismic – stability - 0
- Through – seepage (stability) - 4
- Penetrations through levee -5
- Encroachment (pools) - 0
- Wave/Wind erosion leads to overtopping - 0

- Wave erosion - 0
- Ditches (seepage / encroachment) - 0

After this vote, much discussion was held as to how the different failure modes interact and impact one another.

Mohsen inquired about the levee failure in RD 784 in '97. Ed advised the erosion moved back faster than they could do the flood fighting and it became larger at the crescent as it worked its way back to the levee. Mohsen stated his point is that some of these breaches have occurred on some good levees in relation to the inspection point. Ed advised he said that he's seen where erosion has affected the seepage, which has impacted the stability.

Identification of Significant Failure Modes - Panel Votes
The panel was asked to consider the top seven significant failure modes identified from the previous exercise and vote in regards to how they see the likelihood of a failure mode caused by one of these factors. The results (with the number of votes received) are provided below:

- Under seepage – 10
- Through seepage - 8
- Erosion = Analysis* - 7 / *Research analytical methods – use existing tools to form judgment.
- Overtopping - 4
- Penetrations - 6
- Stability - 6
- Rodents - 6

It was determined that when considering "Other Failure Modes" (sense on how these relate to those identified as most important), judgment is very important, but should not be more about 20%.

Relative Ranking and Contribution of Significant Failure Modes (weighting factor 0 – 100%) – no flood fighting - Team
The panel was then asked to conduct a relative ranking of the significant failure modes with no flood fighting involved. The results were as follows:

- Erosion
- Penetrations
- Rodents
- Others

After another vote, it was determined that the Top 3 may contribute 10-25% to a levee breach or failure.

Discussion of Importance of Judgment Curve – Team
A lengthy discussion was held with the team as far as the importance of the judgment curve and the various components that should be included.

It was noted that certain components are currently being considered in the evaluations and analytical models. These include erosion, penetration, vegetation (includes rodents, beavers, squirrels, etc.), and encroachment. The team felt there were other components that should be considered as well. These include as-builts/knowledge of construction/maintenance, the separation of rodents from vegetation, swimming pool encroachments, penetrations through the levee, and penetrations through levee foundation.

After much discussion, the team came to the consensus that the following components are what need to be considered:

- Encroachments
- Erosion
- Penetrations
  - Through levee
  - Through foundation
- Rodents
  - Beaver
  - Squirrel
- Vegetation
  - Trees

  - Brush
- Maintenance – Overall

It was noted that failure considers the overall reliability of the levee.

Dan advised they are trying to define a methodology of performance curves to apply to both "with" and "without project" conditions. Mike N. responded by asking if this shouldn't be done in parallel to Economics. Dan explained there is a difference between the two based on the performance of the levee. Mary added to this by explaining the goal in their economic analyses is to determine damages based on levee failure. MDR then conveyed to the team that where Mary needs the most support is in determining how to do this.

Mike I. stated that collectively there is not a way to quantify how they feel about a specific section. Les asked Mike I. If there was a way to tell how the seepage and stability curves are being used. Mike I. responded by stating there was, as another category of judgment. He went on to say that on its own, erosion may not be an issue, however, when the section is looked at collectively, it causes "heartburn". Further, individually they may not add up to such a bad score, however, collectively it poses an issue.

Pete contributed to the discussion by inquiring as to how much should judgment affect the curve. Tom Smith added that how comfortable one is with the data they have is an important component. Dan stated in his mind it is more reach-specific.

Les expressed concern about using the term "judgment". He wanted to look at analytical components and temper them. MDR agreed we need to revise the agenda to include "relative importance of judgment". Judgment can be based on non-analytical info as well as analytical inputs. Non-analytical should look at best estimates; while analytical is the best estimate with Co-efficient Of Variations (COV). Henri and Paul both commented that the analytical stuff is what points to failure on the weaker levees. Judgment is still important.

It was noted that consideration of agreement in failure modes & influence, importance of the economic model versus level of protection & public safety can have a difference on the basis of risk and communication. It is important to define the level of performance versus economics.

Discussion of Need for Specific Performance Curve for Unique Flaw / Failure Mode – Team
MDR led the group in a discussion of specific performance curves needed for unique flaws or failure modes. In this discussion failure modes or flaws not covered in typical analysis were looked at. MDR advised it is important to recognize these specific potential failures as they may need to be included in a special curve for special instances, current or future.

Pumping stations/plants, drainage ditches, and farmer water supply wells were some items that were mentioned as having an impact on levee performance. Henri noted that some items could be categorized under "maintenance". Mary commented that while she agrees it can be a failure mode, the problem with maintenance is that it cannot be added in remediation (the sponsors are responsible for the maintenance) or included in the remediation action for the feasibility study.

A question was posed as to whether or not the failure modes should be analyzed or just included in the judgment. It was suggested that special / unique failure modes should be considered for inclusion as a special curve if analytical methods are available. Les commented that his sense was that this should be captured under the various categories under judgment. Mike N. cautioned the team not to double-up and compounding the "unknowns".

Change in Agenda
At this point of the meeting, a decision was made to change the agenda by fast forwarding to looking at the various sites individually versus the development of generalized performance curves for each component.

Site-Specific Performance Curves for Various Situations / Flaws – MDR / MPP
The purpose of this section was to provide Mary with feedback on specifics. For the first site, Mary presented a specific scenario for components of the judgment curve. The team discussed and provided input to the judgment curve.

SITE 1 – Natomas Basin, Sacramento River close to American River at location of Pump Station #1 on the Sacramento River

- **GENERAL CONDITIONS:**
    - Sandy foundation and seepage issues. Seepage analysis shows a very high risk due to under seepage (high hydraulic gradients). Based on URS erosion analysis, this area is flagged as high risk when the water is at the highest elevation, but Mary isn't sure the analyses assessed the existing conditions such as vegetation, riverbank protection and encroachments on the waterside including apartment houses constructed on fill placed on the river berm to the crest of the levee. Mary also sees penetration issues here from pipes from the RD 1000 pump station, pressurized pipes and other. Ed advised the Corps found old wood, concrete, etc. when the Corps studied the area for improvement. Paul noted there are a lot of structures within the entire reach such as restaurants, businesses, etc. On some areas of the reach the levee is oversized, with the crest as much as 60 feet wide. The existing conditions include the following:
    - A deep soil/cement/bentonite wall to be constructed under WRDA'99 authorization
    - No gap
    - An existing shallow slurry wall (30' to 40')
    - Generally the levee crest is 40 feet wide except the area where it is further overbuilt
    - The levee is constructed of sand (typical dredge fill) with containment berm
    - The side slope is as everywhere else 1V:3H on the waterside and 1V:2H on the landside
    - Tom added that this is a unique piece of the river and high water elevations should have lower velocities due to Sacramento Bypass on the upper end which diverts the water in the Yolo Bypass

Scenario #1 – VEGETATION

- **CONDITIONS (and discussion on conditions):**
    - In specific to vegetation, the trees go up to the top of the levee on both sides (water and land). Rodents are an issue, too.
    - Trees - 10 years old in levee
    - Possible roots
    - Henri feels the numbers on Mary's proposed curves are way too high on vegetation
    - Les drove a clarification discussion regarding openness to changing the categories. It was decided the Corps is willing to do this, however, Mary advised she cannot drop vegetation based on Corps policy
    - Clarifying point: vegetation goes to extent of the levee. It is everywhere and oversized
    - Mohsen asked how the tree roots behave near slurry walls. Do they penetrate the wall or what? Ed advised composition of the wall influences the behavior of the roots and their strength.
    - Tom advised the wind affects the trees on levees more than anything else, so he is challenging the current curve result. He thinks the failure mode for trees on levees is windfall.
    - MDR advised we are now looking at redefining failure in this case as poor performance. The meeting's objective is to redefine the judgmental curves based on people opinion with experience on the Sacramento River system.
    - Trees are in 40' crown width section in vicinity of the pump station and at the top of the levee. Are they so bad that they would require human intervention such as flood fighting or levee repairs later? The scenario would be something that might affect the performance of the levee with tree gone needing immediate action such as flood fight:
        - For 60' crown width reach on the overbuilt levee (vote taken after earlier misunderstanding on issue / scope):
            - After removing the high and low factors, the average was 5.14%
        - For 40' reach considering the water at top of levee:
            - After removing the high and low factors, the average was 5.14%
        - For 40' reach considering the water at half of levee height :
            - After removing the high and low factors, the average was 9.14%

    - Results must be consistent with other analytical approaches
    - Mary wants to know how much does water velocity change impact the removal of the trees from the levee slope and cause holes in the slope. The Sacramento Bypass Weir is open at elevation 27 feet and at some point the velocity goes to 0 and then upstream it goes to 2 feet per second back towards the Weir (per Tom Smith). Tom advised this is such a small

percent as associated with vegetation. The problem with trees is wind and erosion. Ed recommended 2% from 28 all the way across to top of levee.

Scenario #2 - ENCROACHMENTS

- **CONDITIONS (and discussion on conditions):**
  - Homes on waterside (difficult to inspect) – multi-million $ homes
  - All of the housing on the water side brings water & utilities together, which makes it difficult to inspect.
  - Restaurants
  - Apartments
  - On the land side, this is an Urban area. The city has a pump station there and there are some ranchettes further up.
  - Most of the encroachments are on the waterside and at the top of levee & berm.
  - Lack of inspection due to fences and hedges
  - Visibility is poor and access is difficult as people will not permit inspections
  - Paul advised there has been work in regards to the inspection – not resolved, but in progress
  - Interventions can be done
    - Inspections
    - Maintenance
  - Mary is most concerned with encroachment (particularly swimming pool and landscaping) causing seepage issues
  - Les noted that they need to be looking at this as a serious condition – safety factor of 1. Problem of Encroachments commensurate with limiting P(S) = 1
  - Ed noted both the seepage and stability analytical methods cannot include the encroachments, however, encroachments can impact seepage and stability
  - Mohsen stated he was more concerned about the leach fields that were put in this area some years ago. He doesn't believe there was anything to regulate their placement.
  - The question was posed if encroachments contribute to the development of a problem in regards to the safety of the levee. It was determined it was higher than trees, but lower than utilities.
    - For 40' crown width reach considering the water at top of levee:
      - After removing the high and low factors, the average was 6.57%
      - Influence factors
        - Operational issues
        - Impact on seepage & stability
        - Water at top of levee
  - MDR brought up the issue of whether or not encroachments should be kept in our evaluation. In some areas, they are significant and others are not. Henri stated he didn't think it is significant enough. He felt in cases where we aren't able to drive or walk on the levees, they should be considered. Paul agreed with Henri on the American River, but on Sacramento River he felt it should be considered. Mary advised she has to include them for consistency, however, she can put the impact as 0 wherein that's the case.
  - Pete & Les suggested we continue this process and see where we are on it after we've looked at few more areas and then revisit it.

Scenario #3 - PENETRATIONS

- **CONDITIONS (and discussion on conditions)**
  - Shallow slurry cut-off wall
  - Utility lines through the levee
  - Pump 1A and Pump 1B are constructed differently and Corps is evaluating this matter per Joe S and is being evaluated under WRDA 96-99. There could be some potential seepage under the boxed culvert. This should be analyzed as a seepage model.
  - Structure was built in 1915. Inspection of the inside is being done and the Corps is awaiting the results.
  - The discharge lines from the pump station have flap gates and hand cranks that are 1914 vintage. There is seepage at joints into conduit.
  - This is the only issue in this area that is not characterized.
  - Mary stated she needs to know if seepage in an issue in regards to the culvert. The response was that seepage is an issue with the culvert and it is being looked at. However, the authorized repair is only for the cut-off wall, does not include discharge line replacement or

repair so seepage along the conduit and structural failure of the culvert remain issues. For the existing condition, Mary has no idea as to what is there. Repair of the conduit would be considered in the CF GRR alternatives.

- A question was posed a far as what the chance is the culvert would damage the levees. MDR noted that if this culvert is this big of a problem, then they need to get engineering involved. This culvert is critical for the entire reach.
- Paul advised this has been an ongoing issue with SAFCA for some time.
- Ed commented that if we pulled the culvert out, then we need to look at the utilities along the rest of the reach. His concern that this one spot will mask things for the entire reach.
- MDR made a decision that at this point we are going to discuss utility penetrations along the reach eliminating the discharge lines from the pumping plant, accepting that these need further civil investigation and special design.
- Paul advised there are some other utilities along the waterside as well as some utility crossings. It is a mixed bag. There is also a big sewer force main and some irrigation lines. These are the ones that Paul is aware of.
- Steve Mahnke mentioned there was a sewer line along I-80 that caved at the installation by directional drilling and this is a concern. The levee settled a couple of inches and a big subsidence was observed under an abandoned house. Ed stated he thought that was going to be put into a judgment. He added that he was not planning to pull that out. Ed asked Steve if the collapse was mitigated. Steve responded that he did not think so. Paul advised pressure grouting was added and impact of seepage was looked at. Mary was involved in the repair of the site that included compaction grouting and backfilling the subsidence. The levee is monitored monthly for any further movements and the reports provided to Mary for information. So far, the repair of the area shows to be satisfactory so there is no more concern regarding this line.
- Paul advised there are some pressurized gas lines as well. These are transmission gas lines and fuel lines that go under the levee.
- It was noted there are lots of utilities; some of which go high, some go low, some are in good shape and others are not.
- A vote was called in regards to Utilities' impact on the levee for the reach from the Sacramento Bypass to the American River:
  - For 40' reach at top of levee (with the water at the upper 3 feet):
    - After removing the high and low factors, the average was 10.29%
    - Influence factors
      - Uncertainty biggest failure
      - Slurry wall cut off shallow, the pipes were not relocated during cut-off wall construction
      - Sewer problem
      - Rectified/Fixed
      - Concerns on directional drilling
      - Sewer line controlled closer
  - Another vote was called for the same conditions with the sewer line being considered:
    - Considering the high and low factors, the average was 19.44%
    - After removing the high and low factors, the average was 16%
  - A third vote was called for the same conditions without sewer line, but considering penetrations in general for this reach:
    - Considering the high and low factors, the average was 6.11%
    - After removing the high and low factors, the average was 5.43%
- Les noted that we need to remember what was said earlier today and not to look at worse conditions. The group is supposed to look at standard deviations. Mary's point was that it must be included in this case because it's the worse condition and the best is zero. In order to get average, she must consider it.
- Pete commented that it sounds like it's the same type of thing as the culvert.

SUGGESTIONS FOR DAY 2

The meeting shifted to a discussion led by MDR as to what could make the discussions better on Day 2.

- Ed suggested Mary go back and provide the details on the scenarios she wants answers to.

- A question was raised if other panels are going to be held on GRR. Ed said perhaps and MDR recommended they make the panels smaller if they do.
- Mike I said he saw the discussions as useful. He thinks we need to go back to our original premise that all of these together only contribute 20% to the judgment. It was agreed that the reach the team just reviewed is different. After this one, is 20% appropriate for judgment?
- Mike N. asked as far as the overall scope was the objective still to get all areas done as originally laid out in the agenda. Dan advised that all areas are needed in order for them to breakout Natomas.
- Tom added that each reach is different and expressed he didn't think the team was going to race through them.
- Les suggested that, for tomorrow, to pick the ones that have the best range of things, i.e., typical versus extreme. Mary advised she doesn't have any "typical".
- A need to prioritize work was expressed
- A recommendation was given to pick a range of sites to get broad feedback.

Day 1 Concluded at 5:15 pm

DAY 2

Project: American River Common Features GRR

Date: Thursday, June 18th, 2009
8:00 am to 4:30 pm
USACE – Sacramento District,
Room 1424

Facilitator: Michael Ramsbotham (MDR), USACE

Meeting Called By: Mary Perlea (MPP), USACE, Project Geotechnical Engineer

ATTENDEES
See Attendance Record (to be attached at end of finalized meeting minutes)

MEETING MINUTES

Sign-In
Day 2 of the meeting commenced at 8:00 am with team members signing in.

Introductory Comments - MDR
MDR led the group with introductory comments. Mary iterated where the meeting ended yesterday in regards to Utilities and the sewer line. She expressed a desire to revisit it this morning in regards to its impact on the levee safety due to the age of the pipe. This is unknown to her at this point.

MDR conveyed his belief that the conclusion drawn was that it should be analyzed separately, giving it a full engineering evaluation and not "lump summed" in this evaluation. He advised we are not going to review it under this judgment curve, but on its own curve supported by additional analysis. He iterated that it should not be "eliminated" but handled separately by a civil engineer, possibly as its own reach.

Ed stated he understood WRDA 96-99 was going to take care of the under seepage portion. The pipe itself was where we were going to do a separate evaluation. Henri said if WRDA 96 covers it, it's probably not going to be the weak link anymore; in addition, it's being maintained. Steve added that with it being made of concrete, it should have long life. Mike I stated he thought it could be a weak link. Ed expressed concern about the pipe joints. Additional concern was expressed regarding who has authority. Ed advised they need to go back and discuss with the PM organization and see where it stands with the WRDA 96. Dan stated they have already made the argument and can argue that repair/replacement of pipe may be accomplished under WRDA 96-99, if needed.

MDR reminded the group the purpose of the meeting is to get through as many of these scenarios as possible in order to give Mary guidance in completing the curves.

RESUMPTION OF SITE 1 DISCUSSIONS FROM DAY 1

Scenario #4 – ANIMAL BURROWS (RODENTS)

- **CONDITIONS (and discussion on conditions)**
    - Animal burrows (low density)
        - 4' to ? in depth
    - There is no history of beaver dens / damage
        - Beaver – low
        - Squirrel – located more near the toe, but can be anywhere on the slope
    - Rodent abatement program is reactive
    - Levee is average of 40' wide
    - There is lots of housing and development (on both sides)
    - Cut off wall = 35'
    - A vote was called for these conditions:
        - Considering the high and low factors, the average was 2.78%
        - After removing the high and low factors, the average was 2.71%
        - Conclusion: Animal burrows not a significant issue at this site

Scenario #5 – EROSION

- **CONDITIONS (and discussion on conditions)**
    - No Sacramento Bank Erosion Site documented per Tom Smith
    - Houses & Encroachments add some problem
    - Per Tom Smith, no history of erosion; the Sacramento Bypass Weir is at elevation 27 ft, no issue; velocity changes upstream
    - Sand covers the site. It is a very sandy site and there is a unique hydraulic condition that keeps that site scoured out. It has been fixed, so Tom stated he doesn't see a threat of erosion to the reach
    - Erosion from the river at high flow is not a problem; however, it could be with one of those intermediate flows with the water below the elevation 27 feet (below the Sacramento Bypass Weir)
    - Wind wave erosion may be an issue as much as stream velocity?
    - Tom advised they have documented no erosion in this part of the river due to wind wave – short term duration.
    - A vote was called for these conditions:
        - Considering the high and low factors, the average was 4.11%
        - After removing the high and low factors, the average was 3.86%
        - Conclusion: Erosion not an issue overall at this site

SUMMARY OF COMPONENTS ON THIS REACH (PREDICTING ALL WOULD EQUAL 10-25%)

(General) Utilities (without sewer)	6%
Vegetation	2-3%
Erosion	4%
Encroachment	7%
Rodents	3%
TOTAL	22-23% … not in the formulary method

- FORMULARY METHOD / JUDGMENT = 80.6% … 19.5% PROBABILITY OF FAILURE

The group decided to take a different rating approach on the subsequent sites. It was decided to discuss all conditions at the individual sites and then vote on all judgment components at the same time. If further discussion is needed, additional votes could be taken. The numbers next to each of the components reflect the average after excluding the highest and lowest factor.

SITE 2 – NATOMAS **CROSS CANAL – DOWNSTREAM OF HIGHWAY 99 / VESTAL DRAIN (24' TO 43.5' landside of the levee toe)**

- **GENERAL CONDITIONS:**
    - Vestal Drain Canal is near the levee
    - Historical seepage problems / remediated
    - Waterside stability at one location
    - Other slips on water side

- Several phases of remediation
- Grass only on the levee they regularly burn
- Embankment constructed of fat clay
- Cracks – 3' deep
- There is a landside berm and chimney drain
- Crest at 43' high / 20' wide
- A vote was called with these conditions at the top of levee elevation of 43.5'. The results and additional discussion points follow:
  - Utilities – 5%
    - Few, but old
    - 2 Pump Stations
    - Water intake
    - Pipes are 3' wide and are penetrating the levees a little over mid-height
    - Pressurized coated steel pipes that are coated below the 200-year water level

  - Vegetation – 1%
    - Agricultural area on the landside
    - A few trees on water side
  - Erosion – 2.7%
    - Erosion from wind wave pretty low, not an issue
    - Flow velocity is low
    - Erosion at outfall structures mostly
  - Encroachments – 1%
    - Highway 99
  - Rodents - 6.5%
    - Yes, east end – beaver and beaver dams in the berm; no ground squirrel
  - Total 16%
    - The group was satisfied with these numbers

SITE 3 - AMERICAN RIVER SOUTH - CLOSE TO CAPITAL CITY FREEWAY BRIDGE

- **GENERAL CONDITIONS:**
  - Deep slurry cut-off wall except the window at the bridge that will be closed as WRDA 99
  - SAFCA is placing additional rock onto the levee, but doesn't go up to the crest
  - River Park flood fight in'55 for erosion
  - Cap City Freeway flood fight in '86 for erosion
  - H Street Bridge
  - All part of historical Erosion – Vegetation covers portion of the levee; Stone protection placed on 5 sites
  - Tom provided Dan's team last week with a report about the erosion and the existing hard layers in lower American River. This has a lot of the detail that will be included in the CGF GRR alternatives.
  - Downstream of Watt North bank and head cut to sewer line there is potential for channel erosion
  - In regards to velocity on levee, 1 – 2 fps for a discharge of 145,000 to 160,000 cfs. The discharge when the water is at the top of the levee is 192,000 cfs.
  - Significantly Encroached with houses, swimming pools and other
  - Trees on Levee / Some toppling with wind events
  - Considering entire Reach A from Mayhew to end of River Park, a vote was called with these conditions considering the water at the top of levee elevation of 60'. The results and additional discussion points follow:
    - Utilities – 3.86%
      - Many gravity lines penetrations
      - Some windows in the slurry cut-off wall remain but supposed to be closed
    - Vegetation – 3.00%
      - Vegetation reaches top of levee on both land and water side of levee
    - Erosion – 31.43%
      - Some historical erosion issues
    - Encroachment – 3.57%

            - Lots of houses with swimming pools
            - Homes close to the levee
        - Rodents – 2.43%
            - Rodent issues (not bad – rodent abatement and grouting programs are active)
        - Total is 44% / Overall average was 31%
            - Conclusion: >
    - A second vote was taken under the same conditions for erosion only considering the water at the top of the levee. The results were:
        - Average of 60%
    - A third vote was taken under the same conditions for erosion only at 145 cfs at 6 feet below the top of the levee. The results were:
        - Average of 36%
    - Mary inquired if we could consider the same threat on the North side. The response was yes, the same mechanism should be considered. Paul noted the North side is not encroached, so the encroachment may be less on the North side.
    - With the significant erosion risk, the group noted that this failure method should be pulled out of the judgment curves on this reach and treated with an analytical approach similar to the seepage and stability.

SITE 4 – SACRAMENTO RIVER SOUTH – FROM AMERICAN RIVER DOWN TO LITTLE POCKET

- **GENERAL CONDITIONS:**
    - Levee is 14'high
    - There is a small floodwall, about 4 feet on the landside that works mainly as a retaining wall for the fill placed on the landside. The floodwall is high on the waterside. Railroad lines are on the landside fill. The City will construct the Riverside Promenade along this reach.
    - Numerous encroachments
    - Lot of seepage, mostly clear water, particularly at I-5.
    - 'Boat' I-5 Section is problematic
    - Pioneer Reservoir – relief wells and seepage berm
    - Erosion - "Concrete" rumble placed on the waterside slope that is less efficient for erosion but attracts rodents
    - Mary doesn't know if penetrations are controlled, but there are many of them
    - Closure sections are upstream of Old Sac
    - Just downstream of confluence with American River – some erosion
    - Sutter Road presents a weak link
        - *highest-tallest levee section*
        - *erosion issue*
        - *small slips at entrance*
    - Sac Bank sites are not finished
    - Erosion site at downstream end of reach jus above Little Pocket = at RM 55.2
    - I-5 higher than levee
    - Section very steep
    - Nothing "typical" about this reach.
    - Beavers are active
    - Stan Solida Cave in void at Sac RM 56.7L
    - Erosion site at Captain's Table is being considered as part of this
    - There are some relief wells
    - A vote was called with these conditions considering the water at the top of levee elevation. The results and additional discussion points follow:
        - Utilities – 5.43%
        - Vegetation – 4.71%
        - Erosion – 15.71%
        - Encroachment – 5.71%
        - Rodents – 7.86%
    - 2nd vote taken after discussion had the following results:
        - Utilities – 7.14%
        - Vegetation – 3.14%
        - Erosion – 13.57%

- Encroachment – 6.00%
- Rodents – 6.43%
- Medians were as follows:
    - Utilities - 7
    - Vegetation - 3
    - Erosion - 15
    - Encroachment - 5
    - Rodents – 5
- On lower Sacramento River, it's not just erosion from wind wave, but velocity is involved as well.

SHAPE OF THE CURVES DISCUSSION:
The group diverted from ranking the components for specific sites to holding a brief discussion regarding the shape of the curves. Highlights of the discussion included:

- The shape of the curve may vary
- 0 P(f) not necessarily at toe of levee
- 0 P(f) could be somewhere above the toe
- Specific characteristics of levee will impact shape / inflection points
- Generally concave up to design walls surface of defect
- Risk may not start at elevation of landside levee toe.
- Judgment curves are to deal with miscellaneous conditions not analyzed in seepage and/or stability analyses.

SITE 5 – SACRAMENTO RIVER - LITTLE POCKET (RM 54 to 56)

- **GENERAL CONDITIONS:**
    - Top of Levee is 41' with 20' wide
    - Steep waterside slopes
    - Deep Cutoff wall
    - We do not own right-of-way / access is limited / no immediate access/fences and gates all along the levee slopes and crown
    - A lot of room on the waterside for rodents – hard to mitigate, but not an apparent problem
    - A lot of vegetation / trees & plants
    - Seepage a problem before cutoff wall
    - Lots of penetrations
    - Bend in the river – large berm / erosion not an issue
    - A lot of encroachments
        - *Swimming Pools – some go to the toe of the levees*
        - *Tennis Court – cracked up due to under seepage or perhaps just normal wear?*
        - *Sprinklers all over the place*
    - A vote was called with these conditions at the top of levee elevation. The results and additional discussion points follow:
        - Utilities – 4.43%
        - Vegetation – 2.71%
        - Erosion – 8.43%
        - Encroachment – 6.43%
        - Rodents – 3.43%
        - Medians:
            - Utilities - 5
            - Vegetation - 2
            - Erosion - 8
            - Encroachment - 6
            - Rodents – 3
    - After further discussion it was determined that a second vote was not needed.
    - A special note:
        - *It will be important for Mary to go back and compare the feedback on various sites for the same issue. It should also be noted that information is based on conditions today and are subject to change.*

SITE 6 – ARCADE CREEK

- **GENERAL CONDITIONS:**
    - There is a pump station
    - Levee height deficiency - Water is at top of levee
    - Levee embankments aren't as bad as the others
    - Levee constructed of clay material and it is less erosive
    - No trees on these levees
    - Levees were raised in the 1990s
    - T-wall exists
    - Arcade Creek is a narrow, deep and fast-acting canal
    - Some of the tallest floodwalls – up to 20'
    - Beavers are an issue
        - Have had collapses due to them upstream of Norwood bridge on the north side
        - Not many squirrel
    - Deep drainage canal on North side where it meets NEMDC. The city has an 8 foot deep concrete line channel
    - No slurry walls
    - Some older utilities cross the levees
    - Several pump stations that came in with the Folsom Dam Project and are likely around 60-years old
    - Protected agricultural area at one time, now highly developed
    - Access is good
    - Few encroachments
    - Water has high velocity, but not aware of erosion issues
    - A vote was called with these conditions at the top of levee elevation. The results and additional discussion points follow:
        - Utilities – 3.86%
        - Vegetation – 1%
        - Erosion – 2.71%
        - Encroachment – 2.86%
        - Rodents – 5.43%
        - Medians:
            - Utilities - 5
            - Vegetation - 1
            - Erosion - 3
            - Encroachment - 3
            - Rodents – 5
    - A second vote with the same conditions was called for utilities and rodents only after further discussion. The results and additional discussion points follow:
        - Utilities – 6.86%
        - Vegetation –
        - Erosion –
        - Encroachment –
        - Rodents – 8.29%
        - Medians:
            - Utilities - 7
            - Vegetation -
            - Erosion -
            - Encroachment -
            - Rodents – 8

SITE 7 – SACRAMENTO RIVER BIG POCKET

- **GENERAL CONDITIONS:**
    - This is a narrow levee, only about 20' wide
    - It is asphalt paved
    - Sump132 is an active seepage site. Relief wells have been put in to fix and bring the new intake into compliance

- Slurry wall stops at Cliff's Marina, where railroad track leaves the levee
- Known utilities were cut and relocated
- Old irrigation line was plugged last summer
- Encroachments are dramatic (same as in Little Pocket, but may have some going into the levee)
    - *Cliff's Marina*
    - *Railroad prohibits inspection of the levee*
    - *Swimming Pools*
    - *Houses and fences*
- Public highway at toe
- Trees go to the crest of the levee and cover most of the levee center line
    - *6 ft tree in diameter on the levee*
- Erosion issues? Yes, numerous erosion sites at this part of the levee; on West Sac after Mason's Bend, there is a scour / straightens up downstream at Garcia Bend There have been a lot of repair work in this area (6-8 sites repaired) after 2006 flooding. Critical site repair has been completed. Repairs may not include key in trench
- No berm. It is right at the toe of the levee
- Made of silty sand and sand; there is also some sort of organic crust, not clay
- Soil / Cement / Bentonite slurry wall
- Active Erosion Reach
- Minimal rodent activity
- Wind wave – minimal erosion
- Boat wake / wave issue at lower water, but this is a summer elevation issue
- A vote was called with these conditions at the top of levee elevation. The results and additional discussion points follow:
    - Utilities – 3.86%
    - Vegetation – 3.29%
    - Erosion – 13.14%
    - Encroachment – 7.43%
    - Rodents – 3.29%
    - Medians:
        - Utilities - 3
        - Vegetation - 2
        - Erosion - 15
        - Encroachment - 7
        - Rodents – 3
    - Conclusion: The group feels this erosion is just as bad as Little Pocket (although Little Pocket higher).
- A second vote with the same conditions was called for erosion only after further discussion. The results and additional discussion points follow:
    - Utilities –
    - Vegetation –
    - Erosion – 16.29%
    - Encroachment –
    - Rodents -
    - Medians:
        - Utilities -
        - Vegetation -
        - Erosion - 16
        - Encroachment -
        - Rodents –
        - Encroachment -
        - Rodents –

Site 7 concluded the rankings portion of the meeting for specific sites.

QUESTION FROM DAN:

MDR advised the team he had a question from the Project Manager, Dan Tibbitts, to pose to the panel:

> "On the components below, are there any other problem reaches that we did not cover, i.e., "reaches of concern"?

Les stated he feels the 5-6 sites that we've rated should cover the other 21 sites. Mike I agreed.

After further discussion, the following areas were identified to be of concern for the component described:

UTILITIES:
- Natomas: Pump Station 1 & 2
- Pleasant Grove Creek Canal
- Del Paso Blvd Flood Gate

VEGETATION:
- North of I-5 along Sacramento River

EROSION:
- Wind wave – Sacramento River just below Cross Canal

ENCROACHMENTS:
- None

RODENTS:
- None

QUESTION FROM MARY: SPD1 SAYS SENSITIVITY ANALYSIS NEEDS TO BE DONE IF THE LEVEE FAILS OR JUST POOR PERFORMANCE? PROBABILITY OF POOR PERFORMANCE VERSUS PROBABILITY OF BREACH?
The group proceeded to have a lively discussion on these questions. Highlights / comments of the discussion included:

- As water goes up, human intervention will be less successful. You would be pulling your crews off at that point due to danger level.
- Ability to mitigate the risk with human intervention increases as water surface goes down.
- Can you easily translate P(f) to P(breach)?
- Do we have any chances to prevent failure?
- What is the affect of flood fighting?
- What are the chances of going from poor performance to failure?
- Intervention is either successful or not; if successful, no breach; if not successful, can have breach or no breach (depends if the correct problem has been detected).
- No intervention?
- Success is defined as stopping the progression of the levee failure / breach.
- Don't want to count flood fighting first
- Henri commented it is almost like you need another curve
- Economics group is wanting these sensitivity analysis
- This can be looked at as a "correction factor", however, one is the real curve
- Paul noted that the curves will be different depending where you are in the country.
- Toe of levee does not appear to be an issue
- 33% of the levee height eventually to be considered as less likely the poor performance to lead to failure
- Mike I suggested Mary refers back to historical data and that this discussion is purely conjecture. He doesn't feel it can be done in this forum without empirical data.
- MDR iterated to Mary that she has to look at each curve and evaluate them on this topic individually. She would need another Expert Elicitation to cover this topic
- This topic of discussion ended without resolution

LESSONS LEARNED / RECOMMENDATIONS TO CORPS – Discussion started at 4:20 pm
MDR led the team in a discussion on the lessons learned, to include recommendations to the Corps, as a result of this 2-day meeting and the feedback they have provided. Highlights / comments include:

- Vegetation does not contribute significantly to P (poor performance)
- Local sponsors with knowledge & experience in maintaining the levee is extremely valuable to the discussion as well as the history of such information
- Need biased and unbiased opinions
- Confidence in prediction were on the reaches where folks had experience and knowledge
- Need better "read ahead" performance history
- Les asked MDR what he thought about having nine panelists. Les commented that he thought it worked out well in regards to consensus. MDR responded that in order to get to what we needed to talk about, it was good to have a broad group; but to try to accomplish 27 sites, it was too

many people. Smaller groups normally result in faster answers; however, larger groups likely produce better answers. For this, he felt it went well. Having a panel of nine was valuable in this case.

- Ed expressed he felt the generalized discussion first was good and then going to site specific worked well. Start up with general discussion was helpful for him.
- Les added having clear set of definition and purpose/goal would have been helpful. Further, he said he thought we got there, it just took a while.
- Mike I felt the way we got through things this afternoon went very well.
- Paul suggested that a more expedient voting method would have been helpful and helped things to move forward.
- Mike N noted that judgment curves are important and can significantly affect performance / economic results. He would like to see a cap on how judgment affects the overall decision. Inclusion of judgment curves make “flaws” / failures more frequent and likely increases average annual damages: as components increase, P(f) increases. He expressed a summary of data developed simultaneously as debate proceeds would be good.
- Need separate evaluation for critical site P(f) high and not included in judgment.
- Mike N. inquired about how rodents are being looked at. From discussion, it seems that beavers are of much more concern than squirrels.
- There was an determined need to separate out:
    - Pump Plant 1?
    - Sewer Line?
- What happens now as far as information collected these past two days?
    - Melanie will compose a draft of the meeting minutes to be distributed to the Expert Elicitation attendees
    - Attendees will be asked to provide comments by tracking changes within a specified time
    - Melanie will finalize minutes
    - Mary will then compile report to include summary, statistical information as well as the revised curves. The report will require the signatures of everyone.
    - Once produced, she will provide a copy to all
- Henri noted that while the curves developed by the panel are much lower than Mary’s, it doesn’t mean the existing conditions considering encroachment, penetration and vegetation are desirable. He advised there is a need to keep probability approach separate from deterministic.
- Dan advised the team they have an array of alternatives that will comply with environmental or with SAFCA’s (for which they will likely need a variance).

Wrap-Up Comments - Team

MDR solicited wrap-up comments from the team.

Ed told the team of a vegetation issue he experienced in Lompoc with cottonwood after a large storm. It took out the bridge and flooded the area. It was a big hindrance.

Day 2 Concluded at 5:10 pm

US Army Corps
of Engineers
Sacramento District
Engineering Division

Lower San Joaquin River Feasibility Report - Environmental Impact Report / Environmental Impacts Statement

San Joaquin County, California

HYDRAULIC DESIGN APPENDIX

February 2015

This page was left blank to facilitate two-sided photocopying.

Table of Contents

List of Tables

List of Plates

Attachments

Attachment A - Geotechnical Fragility Curves

Acronyms and Abbreviations

ACE	Annual Chance of Exceedance
CNRFC	California Nevada River Forecast Center
CVFED	Central Valley Floodplain Evaluation and Delineation
CVFPP	Central Valley Flood Protection Plan
Comp Study	Sacramento and San Joaquin River Basins Comprehensive Study
DWR	Department of Water Resources
FRM	Flood Risk Management
HEC	Hydrologic Engineering Center
HTOL	Hydraulic Top of Levee
NAD83	North American Datum of 1983
NAVD88	North American Vertical Datum of 1988
NGVD29	National Geodetic Vertical Datum of 1929
NLDB	National Levee Database
NWS	National Weather Service
PBI	Peterson Brustad Incorporated
RD	Reclamation District
SD	Standard Deviation
SJAFCA	San Joaquin Area Flood Control Agency
ULDC	Urban Levee Design Criteria (State of California)
USGS	United States Geological Survey
USACE	United States Army Corps of Engineers
UPRR	Union Pacific Railroad
VE	Value Engineering

1.0 Introduction

1.1 Purpose and Scope

The purpose of this report is to describe the hydraulic analysis conducted in support of the Lower San Joaquin Feasibility Study. This report provides a description of the sources of potential flooding and documents the analysis of the final array of alternatives to reduce flood risk. Analysis of the preliminary and focused array of alternatives is summarized in the main feasibility report. The level of detail is limited to that necessary to differentiate the final plans. Further analysis may be necessary after public and agency review of the draft report to address comments and support feasibility level design of the Tentatively Selected Plan (TSP).

1.2 Background

The U.S. Army Corps of Engineers, together with the State of California and San Joaquin Area Flood Control Agency (SJAFCA) conducted this feasibility study to select a flood risk management plan that reduces flood risk and provides ancillary ecosystem restoration and recreation benefits within the study area. The goal of the study is to identify a cost effective, technically feasible and locally acceptable project that best reduces flood risk and flood damages and complies with all Federal, State, and local laws and regulations.

1.3 Location

The Lower San Joaquin study area is located within the Stockton metropolitan area of the State of California, approximately 50 miles south of Sacramento. The study area includes approximately 64 square miles of urban and agricultural lands subject to comingled flooding from multiple sources. A map of the San Joaquin River watershed is included as Plate 1. A map of the Sacramento-San Joaquin Delta is provided as Plate 2. A map of the study area topography is included as Plate 3 and a map of economic damage areas is presented in Plate 4.

The study area includes portions of communities of Stockton, Lathrop, and Manteca. Based on 2010 census data and floodplain mapping presented herein, approximately 235,000 people reside within the study area 0.2% (1/500) Annual Chance Exceedance (ACE) Floodplain. A map of population density within the study area is provided in Plate 5. The population within hypothetical natural floodplains is tabulated in Table 1. The hypothetical natural floodplain represents the area potentially at risk if a levee was to fail along any of the primary sources of flooding identified in this study.

The majority of land use in the study area is urbanized, comprising approximately 60% of land use. A map of land use types in the study area is presented in Plate 6. The amount of land that is currently developed, protected from development (parks, refuge lands, etc), and potentially developable is provided in Table 2. The primary sources of flooding within the study area are the San Joaquin River Delta, San Joaquin River, Mormon Slough, Calaveras River, and local interior drainage.

Table 1. 2010 Population, Lower San Joaquin Study Area

Economic Evaluation Area	Population within Natural ACE Floodplain						
	50% (1/2)	**10% (1/10)**	**4% (1/25)**	**2% (1/50)**	**1% (1/100)**	**0.5% (1/200)**	**0.2% (1/500)**
NS-02	13600	18700	19400	20400	21400	22800	23000
NS-03	11900	16100	16700	18400	18500	18800	18800
NS-04	0	0	0	26600	32300	35900	38800
CS-01	14300	19000	19900	22000	22600	22900	23100
CS-02	0	0	0	36200	42900	47300	47900
CS-03	0	0	0	24900	28500	31000	38800
RD17	0	0	25800	38200	43600	44600	44600
Total	39800	53800	81900	186600	209800	223300	235000

Table 2. Land Use Types, Lower San Joaquin Feasibility Study Area

Economic Evaluation Area	**Total Area Within 0.2% ACE Floodplain (Acres)**	**Area Protected from Development (Acres)**	**Developed Area (Acres)**	**Undeveloped or Unprotected Area (Acres)**
NS-02	2300	200	1800	300
NS-03	2400	0	1900	500
NS-04	3500	0	3000	400
CS-01	2600	100	2300	300
CS-02	6400	300	5200	900
CS-03	4200	100	3800	400
RD17	19600	200	6600	12800
Total	41200	900	24700	15500
Numbers may not total correctly due to rounding				

1.4 Plan Formulation

The final array of alternative plans described in this report were selected through a risk informed plan formulation process involving multi-disciplinary analysis using an appropriate level of detail for decision making. At each level of screening and analysis the level of detail was improved and the relative uncertainty was assessed. A measure or alternative was carried forward if the level of detail was insufficient to screen it out. Throughout this process the concept of absolute accuracy versus relative accuracy was considered in alternative comparisons. Although it would appear that every plan should be compared to the most accurate assessment of existing conditions, this is not necessary because the relative accuracy between plans is sufficient to select the most optimal plans to move forward. The plan formulation process is summarized below and described in detail in the feasibility report.

The study area was defined based on an initial screening of flood risk management opportunities within the study area. The screening resulted in limiting the flood damages within the economic impact areas shown on Plate 4.

An initial array of alternatives was derived from an evaluation of the without project conditions. The initial array included incremental levee improvements, setback levees and bypass channels.

A focused array of alternatives was derived from an initial array of alternatives. The focused alternatives were evaluated using qualitative and quantitative engineering analyses. Analyses included floodplain hydraulic modeling, cost estimating, and economic benefit estimations. The level of detail was limited to that required to decide which plans to carry forward. Results were evaluated at a combined Value Engineering (VE) study and planning charette attended by the project sponsors and subject matter experts. At the conclusion of the VE and planning charette, refinements to the focused array of alternatives were identified for further, more detailed analysis. The analysis of the focused array of alternatives included an evaluation levee raises to meet the ULDC requirements. The levee raises were found to produce greater net benefits than without raises. Therefore, the final alternatives included the levee raises. This is discussed in the Feasibility Study Report and Economic Appendix.

Only the final alternatives are described in this report. Final alternatives were selected from the focused alternatives to be studied in increased detail. This level of detail included additional qualitative and quantitative engineering analyses. Analyses included refined cost estimating, economic benefit estimates, and impacts analysis. The level of detail was limited to that required to decide which plan to carry forward as the Tentatively Selected Plan (TSP). Additional details describing hydraulic analysis performed for the study are available in internal memorandums on file within the Sacramento District Hydraulic Analysis Section. A summary of the final alternatives described in this report is provided in Table 3.

Table 3

Comparison of Final Alternative Features

Alternative	Improve Delta Front Levees	Improve North and Central Stockton San Joaquin River Levees	Improve RD17 San Joaquin Levees	Improve Lower Calaveras River Levees	Improve Stockton Diverting Canal Levees	Construct Mormon Channel Bypass	Raise levee height as needed to meet DWR ULDC (a)
1							
7A	X	X		X			(b)
7B	X	X	X	X			X
8A	X	X		X	X		(b)
8B	X	X	X	X	X		X
9A	X	X		X		X	(b)
9B	X	X	X	X		X	X

(a) DWR Urban Levee Design Criteria (ULDC) requires the levee height to be a minimum of 3 feet above the mean 0.5% or wind wave runup associated with the ACE stage estimate for 2070 sea level conditions.
(b) Height based on RD17 levee also meeting the ULDC requirements. However, the alternative does not include RD17 improvements to meet ULDC.

1.5 National Flood Insurance Program (NFIP).

NFIP levee accreditation is not a specific USACE planning objective. Estimates of Flood Risk Management (FRM) performance presented in this report are limited to the level of detail needed to support economic analysis and comparison of alternatives during the feasibility study process. Results presented herein may not be sufficiently detailed to support NFIP levee accreditation and do not address all of the guidance requirements in EC 1110-2-6067, USACE Process for the National Flood Insurance Program Levee System Evaluation. In addition, hydrologic and hydraulic results presented in this report may be superseded by results from hydrologic and hydraulic models currently being developed by the State of California and local sponsors. The non federal sponsor is responsible for demonstrating a plan meets the sponsor's NFIP objectives.

The U.S. Department of Homeland Security's FEMA is the federal agency responsible for administering the NFIP. As part of the NFIP, FEMA develops Flood Insurance Rate Maps (FIRMs) to identify areas that may be subject to flooding, for both determining flood insurance rates and flood plain management activities (USACE, 2010). FEMA accredits a levee as providing adequate risk reduction on the FIRM if the levee is certified and an adopted operation and maintenance plan provided by the levee owner are confirmed to be adequate (FEMA, 2012). An area impacted by an accredited levee is still considered within the base floodplain but is shown as a moderate-risk area and is labeled Zone X (shaded) on a FIRM. In this case, the National Flood Insurance Program (NFIP) floodplain management regulations do not have a mandatory flood insurance purchase requirement (FEMA 2012). If the levee is not accredited, the area will be mapped as a high-risk area, known as a Special Flood Hazard Area, or SFHA (FEMA, 2012). In this case, the NFIP floodplain management regulations must be enforced and the federal mandatory purchase of flood insurance applies (FEMA, 2012).

Certification consists of documentation, signed and sealed by a registered Professional Engineer, as defined in Chapter 44 of the Code of Federal Regulations (44 CFR), Section 65.2 (FEMA, 2012). This documentation must state the following:

- The levee meets the requirements of 44 CFR, Section 65.10
- The data is accurate to the best of the certifier's knowledge
- The analyses are performed correctly and in accordance with sound engineering practices

This documentation is provided to FEMA to demonstrate that a registered Professional Engineer certified the levee, and meets the specific criteria and standards to provide risk reduction from at least the one-percent-annual-chance flood (FEMA, 2012).

44 CFR, Section 65.10 provides two options for determining if a levee meets the hydrology and hydraulics requirements for levee certification.

- Freeboard Option. Riverine levees must provide a minimum freeboard of three feet above the water-surface level of the base flood. An additional one foot above the minimum is required within 100 feet in either side of structures (such as bridges) riverward of the levee or wherever the flow is constricted. An additional one-half foot above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required.

- Risk and Uncertainty Option. Exceptions to the minimum riverine freeboard requirement may be approved by FEMA. Appropriate engineering analyses demonstrating adequate protection with a lesser freeboard must be submitted to support a request for such an exception. The material presented must evaluate the uncertainty in the estimated base flood elevation profile and include, but not necessarily be limited to an assessment of statistical confidence limits of the 100-year discharge; changes in stage-discharge relationships; and the sources, potential, and magnitude of debris, sediment, and ice

> accumulation. It must be also shown that the levee will remain structurally stable during the base flood when such additional loading considerations are imposed. Under no circumstances will freeboard of less than two feet be accepted. In the case of USACE certification, EC 1110-2-6067 requires specific assurance levels be met. For assurance less than 90% the levee does not pass the EC 1110-2-6067 NFIP criteria. For assurance between 90 and 95% the levee must have minimum of 3 feet of freeboard to pass the EC 1110-2-6067 NFIP criteria. For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass the EC 1110-2-6067 NFIP criteria.

Both approaches also require minimum geotechnical, geometry, erosion control (including wind wave action), vegetation, right of way, encroachment, and penetration standards, plus a number of other standards.

Once the levee meets all the requirements of 44 CFR 65.10, FEMA can accredit the levee and show the area behind it as being a moderate-risk area on a Flood Insurance Rate Map (FIRM) (FEMA, 2012). Levee certification does not warrant or guarantee performance, and it is the responsibility of the levee owner to ensure the levee is being maintained and operated properly (FEMA, 2012). Should USACE be requested to provide an NFIP levee system evaluation, USACE will review all components of the entire levee system as outlined in EC 1110-2-6067, not only design and construction issues as noted in the CFR (USACE, 2010).

Since NFIP accreditation is not a USACE planning objective in the formulation of the National Economic Development (NED) plan, the ability of an NED plan to meet the NFIP criteria is uncertain. An NED plan could appear to meet these criteria during Feasibility. However, an NED plan has no specific authorizing language that requires these criteria are to be met. As a result, it is possible that further analysis during Planning Engineering and Design could determine a NED plan does not meet the NFIP criteria. On the other hand, an NED plan could appear to NOT meet the NFIP criteria during feasibility but could be found to meet those requirements after final design or construction.

1.6 California State Urban Level of Protection.

A local sponsor objective is to meet the California State Urban Level of Protection (ULOP) requirement defined in California Government Code 65007(I). However, this is not a Federal planning objective or requirement. Estimates of Flood Risk Management (FRM) performance presented in this report are limited to the level of detail needed to support economic analysis and comparison of alternatives during the feasibility study process. In addition, hydrologic and hydraulic results presented in this report may be superseded by results from hydrologic and hydraulic models and analysis currently being developed by the State of California and local sponsors. The non federal sponsor is responsible for demonstrating a plan meets the sponsor's ULOP objectives or requirements.

The requirements for a levee to be recognized as contributing to an ULOP are defined in the May 2012 State of California report "Urban Levee Design Criteria" (DWR, 2012). The purpose of the Urban Levee Design Criteria (ULDC) is to provide engineering criteria and guidance for civil engineers to follow in meeting the requirements of California's Government Code Sections

65865.5, 65962, and 66474.5 with respect to findings that levees and floodwalls in the Sacramento-San Joaquin Valley provide protection against a flood that has a 1-in-200 chance of occurring in any given year (Annual Chance of Exceedance (ACE)), and to offer this same guidance to civil engineers working on levees and floodwalls anywhere in California (DWR, 2012).

The ULDC provides two options for determining if a levee meets the urban and urbanizing area levee system design.

- The freeboard option (option 1) requires 3 feet of freeboard above the median 0.5% (1/200) ACE flood event.
- The risk and uncertainty option (option 2) allows for a lesser amount of freeboard if a high level of assurance can be demonstrated. For assurance less than 90% the levee does not pass the ULDC criteria. For assurance between 90 and 95% the levee must have minimum of 3 feet of freeboard to pass the ULDC criteria. For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass the ULDC criteria.

Both ULDC approaches require that modeled water surface profiles assume other levees in the system can overtop, but not fail. Other urban area levees throughout the system are assumed to be at their existing elevation or 0.5% (1/200) plus 3 feet of freeboard, whichever is higher, and non-urban levees are assumed to be at their existing elevation or their authorized design profile, whichever is higher. Both ULDC approaches require that additional freeboard be provided if the wind wave run-up from a 1.3% ACE wind event would exceed the top of levee for the 0.5% (1/200) ACE event. Both ULDC approaches also require minimum geotechnical, geometry, erosion control, vegetation, right of way, encroachment, and penetration standards, plus a number of other standards.

Since a ULOP finding is not a USACE planning objective in the formulation of the National Economic Development Plan (NED) plan, the ability of an NED plan to meet the ULOP criteria is uncertain. An NED plan could appear to meet these criteria during Feasibility. However, an NED plan has no specific authorizing language that requires these criteria are to be met. As a result, it is possible that further analysis during Planning Engineering and Design could determine an NED plan does not meet the ULOP criteria. On the other hand, an NED plan could appear to NOT meet the ULOP criteria during feasibility but could be found to meet those requirements after final design or construction.

1.7 Approach

This report describes the hydraulic design and performance analysis of the final alternative array of the Lower San Joaquin Feasibility Study. Each feature of an alternative was designed following USACE criteria. The performance of each alternative was then evaluated by adjusting inputs in the USACE FDA program to reflect the features of the alternative. The approach of simulating an alternative's performance by changing FDA inputs is described in Section 9 of EM 1110-2-1619, Risk Analysis for Flood Damage Reduction Studies. Inputs to the FDA program were unregulated flow frequency, unregulated flow versus regulated flow, regulated flow versus stage, levee fragility, and stage-damage relationships and their uncertainties. Flow charts describing the hydraulic analysis performed to evaluate the alternatives are provided in Plates 7 and 8 for the San Joaquin and Calaveras Rivers respectively.

a. Levee Height Scenarios. Many of the hydraulic features are identical in the final plans presented herein. Hydraulic models were developed to represent two scenarios to support the evaluation of these plans, the without project scenario and the levee raise scenario. The results of the following two scenarios were utilized to develop the FDA inputs to the four alternatives.

(1) No Action Scenario (NAS). The no-action scenario reflects the hydraulic design features of the existing conditions. Hydraulic model geometry and flows were based on existing levee heights, Manning's roughness, etc.

(2) Levee Raise Scenario (LRS). The levee raise scenario reflects increasing the height of levee reaches (if required) to meet the California Urban Levee design criteria of 0.5% flood with 3 feet of freeboard assuming 2070 sea level conditions. No modifications to the inflow hydrology were necessary because urban areas are significantly upstream and would likely have no impact on flows in the study reach.

b. Project Reach Segments. The study area was divided into project reach segments described in Plates 9A through 9D. The segments were defined based on similar hydrologic, hydraulic, design, and geotechnical characteristics. The engineering design and costs were developed for each of the project reach segments and combined to estimate the costs of each alternative. The estimated cost of each alternative is provided in the feasibility study report.

c. Economic Impact Areas. Economic impact areas were defined based on the concept of "separable area". Separable areas or elements are defined as the subdivision of a study area's flood risk based on hydrologic and hydraulic characteristics with identifiable and distinct economic benefits. A "separable element" is defined in 33 United States Code (U.S.C.) Section 2213(f) as a portion of the project that (1) is physically separable from other portions of the project; and (2)(a) achieves hydrologic effects, or (b) produces physical or economic benefits, which are separately identifiable from those produced by other portions of the project.

Within the Lower San Joaquin study area, the floodplain has a relatively low gradient and topographic relief and the separable areas are not clearly defined by basic topographic features alone. The physical separation was estimated by analyzing the hydrologic characteristics. In general, there are eight separable hydrologic areas. The separation is evident in levee breach simulations conducted for the study and described below. The delta region defines many of the separable areas. The stage within the delta region is affected by coincident ocean tides and inflows from the Sacramento and San Joaquin River system. The physical separation between portions of the Lower San Joaquin study area is described below.

(1) North Stockton 01 (NS-01). This area was screened from the final study area early in the plan formulation process. This area is subject to flooding if a breach were to occur in the levees along the upstream reaches of Bear Creek or Mosher slough and the downstream delta reaches. The eastern limit of the NS-01 area defines the limit of delta flood sources.

(2) North Stockton 02 (NS-02). This area is subject to flooding if a breach were to occur in the levees along the upstream reaches of Mosher Slough, Calaveras River, and downstream

delta reaches including Fourteenmile Slough. The eastern limit of the NS-02 area defines the limit of delta flood sources.

(3) North Stockton 03 (NS-03). This area is subject to flooding if a breach were to occur in the levees along the upstream Calaveras River, and downstream delta reaches including Fourteenmile Slough. The eastern limit of the NS-03 area defines the limit of delta flood sources.

(4) North Stockton 04 (NS-04). This area is subject to flooding if a breach were to occur in the levees along the upstream Calaveras River. The area is not subject to flooding from downstream delta reaches.

(5) Central Stockton 01 (CS-01). This area is subject to flooding if a breach were to occur in the levees along Calaveras River, Stockton Diverting Canal, delta reaches, French Camp Slough, and San Joaquin River.

(6) Central Stockton 02 (CS-02). This area is subject to flooding if a breach were to occur in the levees along Stockton Diverting Canal, French Camp Slough, and San Joaquin River.

(7) Central Stockton 03 (CS-03). This area is subject to flooding if a breach were to occur in the levees along Stockton Diverting Canal and Calaveras River. The area is not subject to flooding from the San Joaquin River or delta reaches. The western limit of the area defines the limit of delta flood sources.

(8) Reclamation District 17 (RD17). This area is subject to flooding if a breach were to occur in the San Joaquin River levee or the tieback levee at Weatherbee Lake and Walthall Slough.

1.8 Datum

As required by ER 1110-2-8160 all elevation data provided herein are referenced to the NAVD88 vertical datum. All horizontal data provided herein are referenced to the North American Horizontal Datum of 1983 (NAD83) Horizontal datum. All horizontal coordinates are projected to the California State Plane Zone III coordinate system.

Historical elevation data were converted to NAVD88 from their original legacy reference datum. The method of conversion followed the requirements in ER 1110-2-8160 and the uncertainty in the conversion was accounted for in the study results. In some cases, the original data used for this study was based on NAVD88 and required no conversion.

The following generalized conversion is provided to compare NAVD88 elevations provided in this study to previous studies presented in the legacy NGVD29 datum. Expressed as an equation, Elevation (NGVD29) = Elevation (NAVD88) minus 2.3 to 2.4 feet. The conversion between NAVD88 and NGVD29 ranges from 2.3 to 2.4 feet in the study area.

2.0 STUDY AREA

2.1 Overview

The study area is situated within the Sacramento-San Joaquin Delta watershed. A map of the watershed is included as Plate 1. The contributing drainage area to the Sacramento-San Joaquin Delta encompasses approximately 40,000 square miles. The main contributors of the drainage area are the Sacramento River (25,200 square miles), San Joaquin River (13,500 square miles), and the Mokelumne River (1,200 square miles). Runoff within the study area is highly influenced by upstream reservoir regulation.

2.2 Topography

A topographic map of the study area is presented in Plate 3. The study area has a general slope from east to west. Elevations within the study area range from 50 ft NAVD88 in the east to -20 ft NAVD88 in the west. The general slope of the study area is interrupted by roadway and railway embankments and levees. These features significantly influence the direction of shallow floodwaters within the floodplain.

2.3 Principle Sources of Flooding

The study area is susceptible to comingled flooding from six principle sources including the Sacramento-San Joaquin Delta, San Joaquin River, Calaveras River and Mormon Slough system, Bear Creek, French Camp Slough system, and Mosher Slough. Interior drainage is not considered a principle source of flooding. The following describes the flood sources within the study area.

a. Sacramento and San Joaquin Delta. The Sacramento and San Joaquin Delta covers more than 1,000 square miles of Central California. A map of the delta is provided as Plate 2. The delta is located at the confluence of the Sacramento and San Joaquin Rivers at the head of Suisun Bay, the most easterly extending arm of the San Francisco Bay system. In general, the Delta extends from about Sacramento on the north, to Stockton on the south, and near Pittsburg on the west. This region, which is very flat, has been reclaimed from a natural tidal area by hundreds of miles of levees along natural and manmade waterways that divide it into about 100 tracts locally know as "islands".

Before the islands were reclaimed, much of the Delta was covered by water from the daily tide cycle. During times of high runoff from the Sacramento and San Joaquin Basins, much of the Delta would be flooded. Reclamation of the many of the Delta islands has subjected the peat soils to oxidation. As a result, the interior of most islands have subsided well below sea level. Elevations within the islands now range from just above mean sea level to 10 feet below mean sea level.

Maximum stages within the Delta result from runoff from storms of different origins which do not have the same annual exceedance frequency at all locations, and from tides of varying magnitudes which seldom reach their maximum stages concurrently with the peak flows. In some years the annual maximum stage at all locations occurs during the same storm event. However, in other years, the peak stages in the northern part of the Delta occur during a different time period than those in the southern part of the Delta and vice versa. The differences are caused by the geographical distribution of the contributing drainage basin, antecedent conditions such as snowpack and soil moisture, and the fluctuation of the storm tracks over California. If the flood runoff is from the Sacramento River basin, the stages will be higher in the northern part of the Delta. If the main flood runoff is from the San Joaquin River, then the stages will be higher in the southern part of the Delta.

The Delta Front reaches of the study area is susceptible to flooding from Fourteenmile Slough and Ten Mile Slough. These delta sloughs have relatively small tributary areas. However, the levees along these sloughs provide flood risk reduction from the large volume of water in the Sacramento San Joaquin Delta. If a breach in were to occur in a delta front levee, the floodwaters would likely equalize with the high stage of the delta due to the enormous volume of water.

b. San Joaquin River. The San Joaquin River is the principle stream in the southern half of the Central Valley of California. The San Joaquin is a perennial stream sustained through the summer by melting snow and releases from reservoirs. Its main headwater tributaries, the south and middle forks, rise in glacial lakes in the southern Sierra Nevada. They join at about elevation 3600 feet NAVD88 to form the main stem, which flows west-southwesterly to the valley floor, thence northwesterly down the main trough of the valley to the study area and its terminus at Suisun Bay. Upstream from the study area, the river is joined by several major tributaries flowing from the higher elevations of the Sierra Nevada Mountain Range. There are also a number of minor low elevation tributaries that flow from the east and west and have little effect on flood flows and stages.

The major tributaries flowing from the east are the Stanislaus, Tuolumne, Merced, Chowchilla, and Fresno Rivers. Less significant eastside tributaries comprise French Camp Slough (terminus of Duck and LittleJohns Creeks systems). The principal Westside tributaries are Panoche, Los Banos, San Luis, and Orestimba Creeks. Fresno Slough, a distributary of the Kings river that cuts through the valley-floor barrier ridge separating the Tulare Lake Basin from the San Joaquin River Basin proper, could contribute runoff to the San Joaquin River during extreme flood events. Reaches of the San Joaquin River within the study area are described below.

(1) Stanislaus River to Paradise Cut. The confluence of the San Joaquin and Stanislaus Rivers defines the upstream extent of the hydraulic model used for this study. The USGS San Joaquin River at Newman stream gage is located at the upstream end of this reach approximately 2 miles downstream of the Stanislaus River. Within this reach the San Joaquin River has a meandering plan form consisting of oxbows and cutoffs. The main channel varies in width from 300 to 600 feet. The floodway is contained by left and right bank levees that are approximately 10 to 15 feet tall. The floodway between the levees varies in width from 900 feet to 4000 feet. The distance between the waterside levee toe and channel bank ranges from zero feet to over 2000 feet. Flood stages within this reach are dominated by runoff from the San Joaquin River.

2) Paradise Cut to Old River. Paradise cut defines the upstream extent of this reach. Paradise cut is a distributary from the San Joaquin River and conveys floodwaters west into the Sacramento-San Joaquin Delta. The flow split into paradise cut is managed by Paradise Dam which is a 230 foot long rock weir along the left bank of the San Joaquin River. The flow split is defined by the hydraulic characteristics of the dam and a meander cutoff levee located on the San

Joaquin River downstream of the dam. The meander cutoff levee extends west from the right bank levee and impinges on the San Joaquin River downstream of Paradise Cut.

Within this reach the San Joaquin River transitions to a less sinuous plan form. The main channel varies in width from 300 to 600 feet. The floodway is contained by left and right bank levees that are approximately 10 to 15 feet tall. At the upstream end of the reach, the floodway width between the levees varies from 900 feet to 4000 feet and the distance between the waterside levee toe and channel bank ranges from zero feet to over 2000 feet. At the downstream end of the reach, the floodway width narrows to approximately 500 feet. However, there is one oxbow reach where the floodway is approximately 2000 feet wide. Flood stages within this reach are dominated by runoff from the San Joaquin River.

Approximately 1 mile downstream of Paradise cut on the right bank is Wetherbee Lake and the upstream tieback levee of RD17. The Wetherbee Lake levee segment along the San Joaquin River was a feature of the San Joaquin Flood Control Project which cut off Walthall slough from the San Joaquin River to reduce damages to a resort development along the river. The RD17 tieback is located downstream of Walthall Slough and extends east along the right bank of the slough to high ground. The RD17 tieback levee is higher than the right bank levee of the San Joaquin River and diverts any floodwaters on the right overbank back into the San Joaquin River. This situation occurred in the flood of January 1997 and is shown on Plate 10. Flood stages within this channel reach are dominated by runoff from the San Joaquin River. Flood stages in the right overbank are dominated by runoff from the San Joaquin River and Stanislaus River.

(3) Old River to French Camp Slough. Old River defines the upstream extent of this reach. Old River is a distributary from the San Joaquin River and conveys floodwaters west into the Sacramento-San Joaquin Delta. There is no hydraulic structure to manage the flow split. The flow split is defined by the hydraulic characteristics of Old River and the San Joaquin River downstream of the flow split.

Within this reach the San Joaquin River further transitions to a less sinuous plan form. The main channel varies in width from 200 to 300 feet. The floodway is contained by left and right bank levees that are approximately 10 to 15 feet tall. From Burns Cutoff to approximately 4 miles downstream right bank levee is approximately 3 feet taller than the left bank. The floodway width between the levees varies from 300 feet to 400 feet and widens to 1400 feet at a few meander bends. The waterside levee face forms the channel bank along most of this reach. Flood stages within this reach are dominated by runoff from the San Joaquin River.

(4) French Camp Slough to Burns Cutoff. French camp slough defines the upstream extent of this reach. French camp slough is a tributary to the San Joaquin River. The reach characteristics of French Camp slough are described below. The main channel varies in width from 200 to 300 feet. The floodway is contained by left and right bank levees that are approximately 10 to 15 feet tall. The floodway width between the levees varies from 300 feet to 400 feet. The waterside levee face is next to the channel bank along most of this reach. Flood stages within this reach are dominated by runoff from the San Joaquin River. However, influence of ocean tides is evident in flood stage hydrographs.

(5) Burns Cutoff to Deep Water Ship Channel. Burns Cutoff defines the upstream extent of this reach. Burns cutoff is a secondary channel of the San Joaquin River which conveys water on the west side of Rough and Ready Island. Burns cutoff flows back to the San Joaquin River/Stockton Deep Water Ship Channel just downstream of the Calaveras River.

The San Joaquin River main channel is approximately 300 feet wide in this reach. The floodway is contained by left and right bank levees that are approximately 10 to 15 feet tall. The right bank levee height tapers to high ground at the downstream end of the reach where it meets the San Joaquin Deep Water Ship Channel. The floodway width between the levees varies from 300 feet to 400 feet. The waterside levee face is next to the channel bank along most of this reach. Flood stages within this reach are dominated by runoff from the San Joaquin River. However, influence of ocean tides is evident in flood stage hydrographs.

(6) Deep Water Ship Channel to Calaveras River. The Stockton Deep water ship channel turning basin defines the upstream extent of this reach. Within this reach the San Joaquin River is maintained as a navigation channel through periodic dredging to a minimum draft of 35 feet below Mean Low Low Water (MLLW). Within this reach the channel is approximately 600 feet wide and is contained by high ground on either side. Smith canal is located along the right bank of this reach approximately one mile downstream of the turning basin. The Calaveras, a tributary to the San Joaquin River is near the downstream end of this reach. Flood stages within this reach are dominated by runoff from the Sacramento and San Joaquin Rivers in combination with ocean tides. Inflows from the Calaveras River and Smith Canal have a negligible influence on the stage in this reach because flood flows are not coincident with the San Joaquin River. In addition the San Joaquin River has a relatively large cross sectional area due to the channel dredging.

c. Calaveras River and Mormon Slough. The Calaveras River is a tributary of the San Joaquin River. Elevations in the Calaveras River drainage vary from about 6,000 feet in the highest headwater areas to about 30 feet in the lower part of the study area. A map of the watershed is provided in Plate 11. In the study area, the Calaveras River is distributary in nature. The stream divides into the north and south branches at Bellota, where a diversion structure was constructed as part of the Federal Mormon Slough Project. The northern branch Calaveras River, flows westerly across the valley floor to join the San Joaquin River just west of Stockton. Very little flow enters this branch except during the summer when diversions are made for irrigation and ground-water replenishment. The southern branch, Mormon Slough, carries most of the flow. Its course extends in a general southwesterly direction from Bellota to the Stockton Diverting Canal diversion dam. The structure diverts all flood flows to the diverting canal which discharges into the Calaveras River. The Mormon Slough reach below the diverting dam is referred to locally as Mormon Channel. The source of flow in Mormon Channel is the local tributary area downstream of the diversion structure.

d. Bear Creek. Bear Creek is a tributary to Disappointment Slough of the San Joaquin Delta. Bear Creek is located near the city of Stockton. A map of the watershed is provided as Plate 12. At its confluence with Disappointment Slough, Bear Creek has a drainage area of approximately 115 square miles. The watershed drains the western slopes of the Sierra Nevada foothills and has a maximum elevation of 1,000 feet NAVD88. The watershed is significantly below the average snowline elevation. Based on preliminary hydrologic and hydraulic model analysis, Bear Creek was not found to be a source of flood risk to the economic impact areas defined within the study area boundary. Therefore, the results of the detailed hydraulic analysis for Bear Creek are not provided in this report.

e. Duck Creek. Duck Creek is a small tributary of the French Camp Slough, south of the City of Stockton, lying between the Calaveras River-Mormon Slough system and Littlejohn Creek. It has a total drainage area of 54 square miles. A map of the watershed is included in Plate 13. Reduction of flood flow in the stream is accomplished by the Farmington Reservoir Project, which prevents overflow of Littlejohn Creek floodwater into Duck Creek, and the Duck Creek Diversion which diverts floodwater from upper Duck Creek into the improved channel of Littlejohn Creek. Approximately half of the Duck Creek drainage area lies above the Duck Creek Diversion Dam. The upstream area, about 28 square miles in extent, lies below 500 feet in elevation and is a typical foothill area, with an overall streambed slope of about 20 feet per mile. Downstream of the diversion structure the gently sloping flat valley floor is a poorly defined tributary drainage area. This creek has no effect on major flood flows in the San Joaquin River.

f. French Camp Slough. French Camp Slough is a tributary to the San Joaquin River south of the City of Stockton. The slough receives waters from Duck Creek and Littlejohn Creek. A map of the watershed is provided as Plate 13. At its confluence with the San Joaquin River, French Camp slough has a drainage area of approximately 430 square miles. The watershed drains the western slopes of the Sierra Nevada foothills and has a maximum elevation of 2,100 feet NAVD88. The watershed is significantly below the average snowline elevation. This slough, with or without upstream reservoirs has no effect on major flood flows in the San Joaquin River (USACE, 1955).

g. Mosher Slough. Mosher slough is a small tributary to Bear Creek which discharges to Disappointment Slough of the Sacramento-San Joaquin Delta. Mosher Slough is located near the City of Stockton in San Joaquin County, California. A map of the watershed is provided in Plate 14. The majority of the watershed is located in the urbanized area of Stockton between Interstate-5 and Highway 99 with the watershed area totaling approximately 16 square miles (SJAFCA, 2012). The watershed's terrain has moderate slopes and reaches a maximum elevation of 65 feet NAVD88. Based on hydrologic frequency analysis the runoff from the area upstream of Thornton Ave is less than 800cfs for a 10% event and does not meet the minimum flow required to establish Federal Flood Control Authority in CFR 238.7(a). However, extension of flood risk management measures upstream of Thornton Ave to address high stages of the Sacramento-San Joaquin Delta would meet the requirements of CFR238.7 (a) (4). It is estimated that flood risk from the Sacramento-San Joaquin Delta extends to Highway 99 and this defines the limit of Federal Interest required by CFR238.7.

2.4 Related Federal Flood Risk Management Projects.

Development of water resources in the basin began in the 1850's and currently includes large multiple-purpose reservoirs, extensive levee and channel improvements, bypasses, and local diversion canals (USACE, 1993). Numerous agencies have been involved in water resources development within the study area. Some of these agencies include the USACE, United States

Bureau of Reclamation (USBR), State of California, county irrigation districts, local reclamation districts, and local levee districts. Design flows for flood risk management projects within the study area are provided in Table 4. Reservoir projects upstream of the study area with dedicated federally authorized flood control space are described in Table 5. The following describes existing Federal Flood Risk Management Projects affecting the study area.

Table 4 Project Design Flood Flows

Reach	Design Flow (cfs)	Design Freeboard (feet)	Source:
Mormon Slough			
Bellota to Potter Creek	12,500	3 with levee 1.5 w/o levee	USACE, 1974
Potter Creek to Diverting Canal	13,500	3 with levee 1.5 w/o levee	USACE, 1974
Stockton Diverting Canal			
Mormon Slough to Calaveras River	13,500	3	USACE, 1974
Lower Calaveras River			
Diverting Canal to San Joaquin River	13,500	3	USACE, 1974
Potter Creek			
Jack Tone Road to Mormon Slough	1000		
San Joaquin River			
Stanislaus River to Paradise Dam (at head of Paradise Cut	52,000	3	USACE, 1993
Paradise Dam to Old River	37,000 (a)	3	USACE, 1963
Old River to French Camp Slough	22,000	3	USACE, 1963
French Camp Slough to Stockton Deep Water Ship Channel	18,000	3	USACE, 1963
French Camp Slough			
French Camp turnpike to San Joaquin River	3000	3	
Duck Creek			
Duck Creek Diversion to Mariposa Road	700	Not Available	USACE, 1967
Mariposa Road to French Camp Slough	900	Not Available	USACE, 1967
Bear Creek (b)			
Highway 99 to Western Pacific Railroad	5,500	3	USACE, 1963
Western Pacific Railroad to Pixley Slough	6,350	3	USACE, 1963
Pixley Slough to San Joaquin River	7,060	3	USACE, 1963

(a) Design diversion capacity of Paradise Cut is 15,000 cfs
(b) Change in design flows by WRDA 2007 per revised Operations and Maintenance Manual, Federal Project levee ends at Disappointment Slough (about 4000 feet upstream of Pixley Slough).

Table 5 Reservoir Projects with Dedicated Flood Storage, San Joaquin River Basin

Reservoir	Owner	Year Constructed	Objective Flow (cfs)	Objective Flow Location	Gross Pool Storage (ac-ft)	Max Dedicated Flood Space (ac-ft)
Friant	USBR	1942	8,000 6,500	Little Dry Creek at Mendota Gage	520,500	170,000
Big Dry Creek	FMFCD	1948	700	Wasteway	30,200	30,200
Farmington	USACE	1951	2,000	Town of Farmington	52,000	52,000
Camanche	EBMUD	1963	5,000	Below Dam	430,900	200,000
New Hogan	USACE	1963	12,500	at Belota	317,100	165,000
Los Banos	USBR	1965	1,000	Los Banos	34,600	14,000
New Exchequer	Merced ID	1967	6,000	Cressey	1,024,600	350,000
Don Pedro	Turlock ID	1971	9,000	Modesto	2,030,000	340,000
Buchanan	USACE	1975	7,400 7,000	Below Dam Chowchilla River at Madera	150,000	45,000
Hidden	USACE	1975	5,000	at Medara Canal	90,000	65,000
New Melones	USBR	1979	8,000	Orange Blossom	2,400,000	450,000

a. New Hogan Lake. New Hogan Lake was authorized by the Flood Control Act of 1944 (Public Law 534, December 22 1044, 78th Congress, 2nd Session). The project is located on the Calaveras River about 28 miles northeast of Stockton, Ca and comprises a rockfill dam with an impervious earth core and a maximum height of about 200 feet. The project also includes four dikes, with a maximum height of 18 feet, and a gated spillway to create a reservoir with a gross storage capacity of 325,900 acre-feet for flood control, irrigation and other water conservation purposes. Construction was initiated in May 1960, dam closure was made in November 1963, and the project was completed for operational use in June 1964.

b. Stockton and Mormon Channels (Diverting Canal). Improvement of Stockton and Mormon Channels was authorized by the River and Harbor Act of June 13, 1902 (H. Doc. 152, 55th Congress, 3d Session, and Annual Report for 1899, p. 3188), to provide for diversion of the waters of Mormon Slough before reaching Mormon and Stockton Channels, for the purpose of preventing deposits of material in the navigable portions of Mormon and Stockton Channels and to divert flood flows past the city of Stockton, California. The results were obtained by construction of (1) a dam across Mormon Slough; (2) a diverting canal 150 feet wide, extending 4.63 miles to the north branch of the Calaveras River; (3) enlargement of the Calaveras River to cross-sectional area of 1,550 square feet, thence to its mouth at San Joaquin River, 5 miles; and (4) a levee along the left bank of the diverting canal and Calaveras River, using material excavated for the channel enlargement.

Construction of new work was initiated in November 1908; the initial construction phase was completed in September 1910. No further new work was accomplished until fiscal year 1922; the project was completed in fiscal year 1923. Most of the silt formerly deposited in Stockton and Mormon Channels is diverted by this canal, obviating serious inconveniences to navigation in the harbor area.

Federal maintenance of these channels for navigation purposes has been discontinued due to completion of levee and channel improvements constructed under provisions included in the Mormon Slough, Calaveras River, project authorized by the Flood Control Act of 1962 (Public Law 874, October 23, 1962, 87th Congress, 2d Session). No Federal maintenance costs have been incurred since Fiscal Year 1969. The project capacity was increased by the Mormon Slough project which was completed in 1971. The Mormon Slough project is described below.

c. Mormon Slough Project. The Mormon Slough project was authorized by the Flood Control Act of 1962 (Public Law 874, October 23, 1962, 87th Congress, 2nd Session). The project provides for the improvement of the Calaveras River system between the town of Bellota and the city of Stockton, California, and consists of minor channel enlargement of Mormon Slough between Bellota and Jack Tone Road; substantial channel enlargement of lower Mormon Slough and the Diverting Canal; new levees along the north bank of the Diverting Canal, along both banks of lower Mormon Slough, and along the south bank of Potters Creek between Jack Tone Road and Mormon Slough; and bank protection on lower Calaveras River levee. The project is an element of the comprehensive development of the Calaveras River basin, contains the flood flows which originate in the area downstream from New Hogan Reservoir and contains the flood control releases for efficient operation of that reservoir.

Preconstruction planning was initiated in January 1964. Construction was initiated in October 1967. Work was substantially completed in February 1970; remaining miscellaneous minor work was completed in December 1971. Project design flows are described in Table 4.

The project was extended with local funding to include levee modifications to achieve 3.3 feet above the median 1% (1/00) ACE water surface along Mormon Slough, Potter Creek, Upper Calaveras River, and Stockton Diverting Canal. Additional project works added include the following:

- Improvement of levees on both banks of the Mormon Slough upstream from the Stockton Diverting Canal to the confluence with Potter Creek. The right bank of Mormon Slough has been modified 400 feet upstream from its confluence with Potter Creek.

- Improvement of levee on left side of Potter Creek from Mormon Slough to Jack Tone Road.

- Improvements of levee on both sides of Stockton Diverting Canal from the Mormon Slough northwest to the confluence with the Upper Calaveras River. Intermittent floodwall construction was also included on the right bank along the same reach.

- Improvements of Levee on both sides of Upper Calaveras River from the junction with the Stockton Diverting Canal to the Central California Traction railroad tracks.

The above improvements to the authorized project were constructed from August 1997 to October 1998.

d. Farmington Dam and Reservoir. Farmington Dam was authorized by the Flood Control Act of 1944 (Public Law, 534, December 22, 1944, 78th Congress, 2nd Session). The project is located on Littlejohn Creek about 2.5 miles upstream from Farmington and about 18 miles east of Stockton, California and consists of an earthfill dam, maximum height 58 feet, and an ungated saddle spillway, creating a reservoir gross storage capacity of 52,000 acre feet (USACE,1974).

Also included in the Farmington project were appurtenant facilities for diverting Duck Creek floodwaters to Littlejohn Creek. However, several of the appurtenant features were later updated by the Little Johns Creek and Calaveras River Stream Group Project and the Duck Creek Project. All facilities are for the exclusive purpose of flood management.

The Duck Creek diversion is located about 0.5 miles east of Farmington California and approximately 3.5 miles downstream from Farmington Dam. The diversion works consist of a low compacted earth dike across Duck Creek with on 72” gated and one 60” ungated outlet discharging into Duck Creek, and an ungated concrete spillway 73 feet long discharging into the diversion channel. According to exhibit B of the operations and maintenance manual, the 72” gate is to remain fully open unless closure is authorized or directed by the District Engineer, Sacramento District, Corps of Engineers (USACE, 1952).

The Duck Creek Diversion Unit also includes dike “B” built across the North Branch of Duck Creek approximately 4 miles downstream from the diversion works; and dike “C” built across the North Branch of Duck Creek approximately 9 miles downstream from the diversion works and just upstream from Jack Tone Road.

Construction was initiated in July 1949; the main dam and spillway were completed in June 1951; the Duck Creek channel improvements were completed in November 1951; and the downstream improvements along Littlejohn Creek were completed in May 1955. Enlargement of the Duck Creek channel downstream of the diversion structure as part of the later Duck Creek Project was authorized under Public Law 685, 84th Congress, 2nd Session. The Duck Creek project is described below.

e. Bear Creek Project. The Bear Creek project is a small tributary of the Sacramento and San Joaquin Delta within the City of Stockton, San Joaquin County. The levee and channel improvements extend along the south channel of Bear Creek from Jack Tone Road about 2 miles south of Lockeford, to Disappointment Slough, a Delta channel which connects with the San Joaquin River. Completed construction provides for channel capacity of 5,500 cfs with 3 feet of freeboard. The project was authorized by the Flood Control Act of 1944 (Public Law 534, December 22, 1944, 78th Congress, 2nd Session). Advance planning on the project was initiated in Fiscal Year 1947 and suspended in Fiscal Year 1951 awaiting agreement with local interests regarding the plan of improvement. The project was classified as “Deferred” in Fiscal Year 1954. A review report was completed during Fiscal Year 1962. Construction was initiated during June 1963 and completed 20 July 1967.

Reclamation Board permits Nos. 15183 and 15214 permitted the diversion of Pixley Slough into Bear Creek and raising the Bear Creek levees to provide 3 feet of freeboard above the 100-yr flow (USACE, 2012). The levees were raised from the downstream end of the project upstream to the Western Pacific Railroad. The modification was completed in about 1990. SJFCA raised the Bear and Pixley levees in 1998.

e. Duck Creek Project. The Duck Creek Project is a small tributary of the San Joaquin River south of the City of Stockton, San Joaquin County, lying between the Calaveras River-Mormon Slough system and Littlejohn Creek. The Duck Creek channel extends from the Duck Creek Diversion (Unit of the Farmington Project) located about 0.5 miles northeast of Farmington California and meanders downstream a distance of about 20 miles to French Camp Slough. Authority to improve the Duck Creek channel was approved by the Chief of Engineers under the small flood control project program authorized by Section 205 of the 1948 Flood Control Act as amended by Public Law 685, 84th Congress, 2nd Session. The project works consist of channel improvements along approximately 20 miles of the Duck Creek channel from 1/2 mile upstream of Escalon-Bellota Road to French Camp Slough. The project includes a short reach of levee on the lower end of Duck Creek along the left and right banks. The design flows are 700 cfs from the Diversion Dam to Mariposa Road and 900cfs below the diversion dam. Construction of the project was initiated May 1965 and completed by January 1967. Project design flows are described in Table 4.

f. Lower San Joaquin River and Tributaries Project. Improvement of lower reaches of the San Joaquin River and Tributaries was authorized by the Flood Control Act of 1944 (Public Law 534, December 22, 1944, 78th Congress, 2nd Session), as modified by Public Law 327, 84th Congress, 1st Session). The project provided for improvement by the Federal Government of the existing channel and levee system on the San Joaquin River from the delta upstream to the mouth of Merced river, and on the lower reaches of the Stanislaus and Tuolumne Rivers, by raising and strengthening of existing levees, construction of new levees, revetment of river banks where required, and removal of accumulated snags in the main river channel. The project also provided for protection of flood plain areas about the mouth of Merced River through local interests construction of levee and channel improvements. The Upper Delta is defined roughly as that portion lying within the influence of flood flows while the lower Delta is that portion influenced mainly by tides. The line of demarcation is considered to be the downstream limits of the San Joaquin Flood Control Project and passes across the Delta from the confluence of the Stockton Deep water ship Channel and the San Joaquin River at the Port of Stockton, to Williams Bridge on Middle River, and to the junction of Paradise Cut and Salmon Slough with Grant Line Canal near Tracy.

The local interest plan of improvement was coordinated with that of the Federal Government to insure the effectiveness of the Federal portion of the projects. In addition to bearing the cost of improvements as required along the San Joaquin River upstream of the mouth of Merced River, Local interests were required for the Federal improvement downstream from Merced River, to furnish flowage rights to overflow certain lands along the San Joaquin River, to furnish all lands, easements, and rights-of-way for construction of improvement of levees; to accomplish all necessary utility alterations and relocations; to hold and save the United States free from damages due to the construction works and their subsequent maintenance and operation; and to maintain all levees and channel improvements after completion in accordance with regulations prescribed by the Secretary of the Army.

Federal construction was initiated in June 1956 and was completed in November 1968 except for the left bank levee along the San Joaquin River, Tuolumne to Merced River reach, which at that time was in the “inactive” category. This work was restored to “active” status on 25 June 1969 as required assurances of local cooperation for the reach were furnished after a change in land ownership. Contract for construction of this reach was initiated in November 1971 and completed in September 1972. The State of California has completed construction of the non-federal portion of the project above the mouth of the Merced River, comprising about 193 miles of new levees, including appurtenant features and about 80 miles of surfacing of existing levees.

The Federal Project levees within RD17 were improved by local interests as a part of the development of Weston Ranch in the City of Stockton. The purpose of the improvement project was to meet FEMA’s National Flood Insurance Program (NFIP) 1% (1/100) ACE floodplain regulatory requirements. FEMA accredited the levee as meeting the National Flood Insurance Requirements in February 1990.

g. Friant Dam. Friant Dam was authorized by the River and Harbor Act (Public Law No. 392) of August 26, 1937 (50 Stat. 850), and the River and Harbor Act of October 17, 1940 (ch 895, 54 Stat. 1198, 1199) extended the authorization to include irrigation distribution systems. The project is located about 25 miles northeast of Fresno and an equal distance east of Madera. It is a concrete gravity structure, 319 feet high and 3,488 feet long at the crest. The spillway is 332 feet wide and is located near the center of the dam. It has three 100 by 18-foot drum gates and a discharge capacity of 83,000 cfs at gross pool elevation.

Initial construction was started in October of 1939 and was completed in November 1942. Work deferred during the war, including spillway gates, outlet valves, Friant-Kern Canal stilling basin, etc., was again started in March of 1946 and the project was completed for operation in 1949.

h. Big Dry Creek Dam. Big Dry Creek Dam was authorized by the Flood Control Act of 1941 (Public Law 288, August 18, 1941, 77th Congress, 1st Session). The project is located about 10 miles northeast of Fresno, California, and about 4 miles northeast of Clovis, California and comprises and earthfill dam across the channel of Big Dry Creek, with a maximum height of 40 feet, creating a reservoir with a maximum capacity of 16,250 acre-feet, all for flood control, together with appurtenant diversion facilities both upstream and downstream from the dam. Construction of the project was initiated in April 1947 and completed in February 1948. Construction of remedial work consisting of erosion control structures to control side-hill erosion was initiated in October 1952 and completed in March 1955.

Modification of the Big Dry Creek Reservoir and Diversion project was included as one of five features that made up the Redbank and Fancher Creeks Flood Control Project in California. The Redbank and Fancher Creeks Flood Control project was authorized for construction on November 17, 1986 by the Water Resources Development Act of 1986. Modifications included raising the dam and spillway crest, constructing a new outlet works on Little Dry Creek and modification to the Big Dry Creek Outlet Works. Construction of the modifications was completed 22 August 1993 (USACE, 1994).

i. Camanche Dam. Federal participation in the construction of Camanche Dam was authorized by the Flood Control Act of 1960 (Public Law 86-645, 14 July 1960, 86th Congress, 2d Session). Camanche Dam and Reservoir is a multiple-purpose dam and reservoir on the Mokelumne River about 20 miles northeast of Stockton. The dam and reservoir was constructed by the East Bay Municipal Utility District which owns and operates the project facilities. Federal interest in the project is in the flood protection afforded by the dam and reservoir commensurate with the flood control benefits to be derived. The project comprises a rock fill dam with impervious earth core, maximum height 171 feet, together with six dikes totaling 19,250 feet in length and a gated spillway, creating a reservoir gross storage capacity of 431,500 acre-feet for flood control and water supply.

In consideration of the Federal contribution toward the first cost of Camanche Reservoir, the East Bay Municipal Utility District provides a flood-control reservation of 200,000 acre-feet, under an agreement with the Department of the Army providing for operation of the reservoir in such manner as will produce the flood-control benefits upon which the monetary contribution is predicated, and will operate the flood-control reservation in accordance with the rules and regulations prescribed by the Secretary of the Army.

The cost allocation for the project was approved by the President on 9 March 1962. Contract for Federal payment for flood control benefits to be attained was consummated 19 March 1962 with the East Bay Municipal Utility District and approved by the Secretary of the Army 19 April 1962. Contract for construction of the main dam and appurtenances was awarded in March 1962; dam closure was completed 7 November 1963. The project was operationally completed in April 1964.

j. Los Banos Dam. Los Banos Dam was authorized by the Central Valley Project, California Act of 1960 (Public Law 488, June 3, 1960, 86th Congress, 2nd Session) and was constructed by the US Bureau of Reclamation, with funds contributed in part by the Federal Government in the interest of flood control, and are operated by the State of California. The project is located on Los Banos Creek, a west side tributary to San Joaquin River, approximately seven miles southwest of the small city of Los Banos in Merced County, California and comprises of a earthfill dam, with a maximum height of 167 feet, creating a reservoir with a maximum capacity of 34,600 acre-feet, most of which is for flood protection, with a provision of a pool for recreation and other purposes. There is also an uncontrolled concrete chute spillway located in the left abutment of the dam with a discharge capacity of 8,600 cfs. Outlet works, including an intake structure, conduit, emergency gate, and control gates are located in the left abutment of the dam and discharge the water into a stilling basin which, in turn, empties into the existing channel of Los Banos Creek downstream from the structure. Construction of the project began in May 1964 and completed by November 1965.

k. New Exchequer Dam. New Exchequer Dam was authorized by the Flood Control Act of 1960 (Public Law 645, July 14th, 1960, 86th Congress, 2nd Session). The project is located in the southern half of the Central Valley in Mariposa County, California. It is on the Merced River about 60 miles above its confluence with the San Joaquin River. New Exchequer Dam and Reservoir were constructed for the purposes of irrigation, power, recreation, and flood control. The reservoir includes a maximum of 400,000 acre-feet of flood control space. New Exchequer Reservoir has a capacity of 1,024,600 acre-feet. The dam is a rockfill dam, concrete faced with a height of 490 feet and is located immediately downstream from the old concrete Exchequer Dam, which is incorporated into the upstream toe of the embankment. A dike of similar gravel fill construction is located about ¾ of a mile northwest of New Exchequer Dam. A spillway, located approximately one mile northwest of the right abutment of New Exchequer Dam consists of a gated spillway and an ungated emergency spillway, each with a concrete ogee crest. The total combined discharge capacity of the gated and emergency spillways is 375,000 cfs. The outlet works consists of a single conduit under the right abutment of both the old and new portions of the dam. Construction of the project was initiated in June 1964 and completed in December 1967.

l. Don Pedro Dam. Don Pedro Dam was authorized by the Flood Control Act of 1944 (Public Law 534, December 22nd, 1944, 78th Congress, 2nd Session). The project is located on the Tuolumne River about 35 miles east of Modesto. The dam is a combination rock and earthfill dam with a maximum height of 585 feet and a total capacity of 2,030,000 acre-feet which is primarily to store irrigation water and has additional benefits including power generation, flood control, and recreation. A spillway located on the abutment ridge west of the dam, consists of both a gated spillway and an ungated emergency spillway, each with a long concrete ogee section. The total combined discharge capacity of the spillway is 472,500 cfs. The outlet works is located in a concrete plug centered approximately on the axis of the dam. Three separate parallel outlets are provided, each controlled by two high-pressure slide gates in tandem. The combined capacity of the three outlets is 7,370 cfs. Construction of the project was initiated in August 1967 and completed in March 1971.

m. Buchanan Dam. Buchanan Lake was authorized by the Flood Control Act of 1962 (Public Law 874, 23 October 1962, 87th Congress, 2d Session). The project provides for construction of a dam on Chowchilla River, about 16 miles northeast of the city of Chowchilla, California, to create a reservoir with gross storage capacity of about 150,000 acre-feet for flood control, irrigation, recreation, and other purposes. The project plan provides for approximately 20 miles of levee and channel improvements along Ash and Berenda Sloughs, distributaries of Chowchilla River. Construction of the project was initiated in June 1972 and completed in June 1978.

n. Hidden Dam and Lake. Hidden Dam and Lake was authorized by the Flood Control Act of 1962 (Public Law 874, 23 October 1962, 87th Congress, 2d Session). The project provides for construction of a dam on Fresno River, about 15 miles northeast of Madera, California, to create a reservoir with gross storage capacity of about 90,000 acre-feet for flood control, irrigation, recreation, and other purposes. The project plan as authorized also provides for approximately 13.3 miles of levee and channel improvements on Fresno River downstream from the damsite. Construction of the project was initiated in June 1972 and completed in June 1978.

o. New Melones Dam. New Melones Lake was authorized by the Flood Control Act of 1944 (Public Law 534. December 22, 1944. 78th Congress, 2d Session), as modified by the Flood Control Act of 1962 (Public Law 874, October 23, 1962, 87th Congress, 2d Session). The project is located on Stanislaus River, about 35 miles northeast of Modesto, California. The project plan provides for construction of a 625 foot high earth and rockfill dam to create a reservoir with a gross storage capacity of 2,400,000 acre-feet for flood control, irrigation, power, recreation, fish and wildlife and water quality control. The plan of improvement also includes construction of a 300,000 KW capacity hydroelectric power plant immediately below the dam. Construction of the project was initiated in 1966 and completed in October 1978.

2.5 Stream Gages.

A list of stream gages applicable to the study area is provided in Table 6. The stream gages are operated by the United States Geological Survey (USGS) and California Department of Water Resources (DWR).

Table 6 Stream Gages, Lower San Joaquin Study Area

Gage Name	Area (Sq Mi)	Agency	Gage Number	Type
San Joaquin River near Vernalis	13,539	USGS	11303500	S,Q
San Joaquin River at Mossdale	15,809	DWR	B95820	S,Q
San Joaquin River at Brandt Bridge	NA	DWR	B95740	S,Q
San Joaquin River below Garwood Bridge	16,177	USGS	11304810	S,Q
Stockton Ship Channel at Burns Cutoff	NA	DWR	B95660	S
Middle River at Borden Highway	NA	DWR	B95500	S
Middle River at Mowry Bridge	NA	DWR	B95540	S
Old River at Clifton Court Ferry	NA	DWR	B95340	S
San Joaquin River at Ringe Pump	NA	DWR	B95620	S
Grant Line Canal at Tracy Road Bridge	NA	DWR	B95300	S
Calaveras River blw New Hogan Dam	363	USACE	NHGQ	Q
Mormon Slough at Bellota	473	USACE	MRS	S,Q
Littlejohn Creek blw Farmington Dam	212	USACE	FRM	S,Q
Littlejohn Creek at Farmington	248	USACE	FRG	S,Q
Bear Creek near Lockeford	48	USGS	11312000	S,Q
Duck Creek Diversion near Farmington	28	USACE	DUC	S,Q
Duck Creek near Farmington	8	USACE	DCK	S,Q
S - Stage Q - Discharge				

2.6 Climate Change.

The primary impacts of climate change on Flood Risk Management projects are related to changes in sea level, changes in inland flood frequency estimates, and their associated uncertainties. These impacts were included in the analysis by assessing performance and economic analysis for existing (2010) and future (2070) climate conditions. The economic analysis conducted during evaluation of the focused array of alternatives evaluated if increases in levee height would be economically justified. It was determined that increases in levee height to meet the DWR Urban Levee Design criteria for 2070 sea level conditions had higher net benefits. Therefore, all alternatives presented in the final array include levee raises that meet ULDC requirements in 2070 as a design assumption. This design assumption was based on all levees in the study area meeting this design assumption. Alternatives that do not include RD17 levee improvements would result in higher stages within the study area. Therefore, for those alternatives that do not include RD17 levee improvements, not all levee reaches would meet the ULDC requirements in 2070.

a. Sea Level Change. The downstream reaches of the study area are within the Sacramento and San Joaquin Delta and are subject to changes in sea level. Hydraulic analysis presented in this study was conducted for existing 2010 sea level conditions and for future conditions in the year 2070. The 2070 condition was selected because it is near the end of the economic period of analysis used for alternative evaluation. In addition, the year 2070 fulfilled the sponsor's objective of determining if the project meets the State of California's Urban Levee of Flood Protection requirements in 2070. The assumption had to be made early in the study, prior to estimates of the beginning and end years for economic analysis. The year used for the hydraulic analysis may not be identical to the economic assumption. However, the change in sea level change between 2010 and 2015 is estimated to be only 0.07 feet and would not have a significant impact on the results. The 2070 conditions were based on the sea level trend described in Curve II of EC 1165-2-212. Additional details are provided in the description of the alternatives.

b. Inland Climate Change. Future changes in the Inland flood flow-frequency estimates related to climate change are less certain than changes in sea level. Climate model research presented in Das, 2011 indicates potential for increases or decreases in flood magnitudes in the year 2049 with all three climate models showing increases by the year 2099 (Das, 2011). The uncertainty of inland climate change was assumed to be within the range of uncertainty already accounted for in the flood frequency analysis utilized in this study. The most likely estimate of future inland flood flow-frequency was assumed to be the same as the existing condition.

3.0 FLOOD EVENTS

The frequency of observed historical floods is not directly comparable to each other due to historical changes in the flood management system. Damage to the study area during most of the known past floods would have been significantly reduced if the floods had occurred with presently existing flood risk management facilities completed and in operation.

The San Joaquin River near Vernalis and Mormon Slough at Belota gages provide a record of large historical floods within the study area. The largest ten floods based on conditions that existed at the time of the flood are provided in Table 7. The largest ten San Joaquin River floods based on regulated conditions is provided in Table 8. Only flood events since 1979 were considered because completion of the last major reservoir project occurred in l979.

Unregulated estimates are useful in the evaluation of hydrologic frequency estimates because they are based on a similar basin condition throughout the record. The largest ten floods based on unregulated conditions from 1930 to 2014 are presented in Table 9. Hypothetical flows, based on unregulated conditions, represent the magnitude of floods without regulation. These are computed by adjusting observed flows to remove the effects of reservoir regulation, which has varied over time as reservoirs were constructed.

The largest flood since 1930 (assuming unregulated conditions) occurred in January 1997. The flood flow was the largest to have occurred since completion of major reservoir projects in 1979. It is estimated the 1997 flood would have been the largest flood since 1930 if the current reservoirs were in place by 1930. The December 1950 flood had a higher peak discharge. However the peak flow would have been less than the 1997 flood if reservoir projects had been completed at that time. A graph of historical floods on the San Joaquin River is provided as Plate 15.

The following are descriptions of significant flood events within the study area.

Table 7
Ten Largest Historical Flood Flows

WY1930-WY2014, San Joaquin River near Vernalis

Annual Ranking	Water Year	Date of Peak	Peak Flow (CFS)
1	1951	12/09/50	79000
2	1997	01/05/97	75600
3	1969	01/27/69	52600
4	1938	03/16/38	51200
5	1955	12/25/55	50900
6	1983	03/07/83	45100
7	1958	04/05/58	41400
8	1943	03/12/43	38900
9	1940	04/02/40	37300
10	1986	03/19/86	36900

Note: Floods prior to 1979 do not reflect existing reservoir regulation system.

Table 8
Ten Largest Floods since completion of Major Reservoir Projects

WY1979-WY2010, San Joaquin River near Vernalis

Annual Ranking	Water Year	Date of Peak	Peak Flow (CFS)	Annual Chance Exceedance
1	1997	01/5/1997	75600	1%
2	1983	3/7/1983	45100	3%
3	1986	3/19/1986	36900	6%
4	1998	2/13/1998	35200	10%
5	2006	4/13/2006	34800	13%
6	1980	2/27/1980	33900	16%
7	1984	01/06/1984	33000	20%
8	1982	04/18/1982	29800	23%
9	1995	3/19/1995	26100	27%
10	1996	03/10/1996	18000	30%

Table 9
Ten Largest Floods based on Unregulated Flow Conditions
WY1930-WY2014, San Joaquin River near Vernalis

Unregulated Condition						
Annual Ranking	Water Year	Date of Peak	1-Day Duration		3-Day Duration	
			1-Day Avg Flow (CFS)	Annual Chance Exceedance	3-Day Avg Flow (cfs)	Annual Chance Exceedance
1	1997	01/4/1997	219,100	1%	191,200	1.1%
2	1956	12/26/1955	187,800	2%	157,200	1.9%
3	1986	2/20/1986	156,600	3%	145,800	3%
4	1951	11/22/1950	135,400	4%	120,800	4%
5	1965	12/25/1964	115,000	6%	98,300	6%
6	1980	01/15/1980	112,300	6%	99,500	6%
7	1963	02/02/1963	101,500	8%	86,900	8%
8	1995	03/13/1995	100,900	8%	91,200	7%
9	1969	01/27/1969	94,400	9%	87,000	8%
10	1938	12/13/1937	90,800	10%	75,000	10%

Unregulated conditions are hypothetical conditions assuming no regulation by upstream reservoirs.
Source: Sacramento and San Joaquin River Basins Comprehensive Study (March 2002)
Annual Ranking based on average flow over 1-Day duration.

a. Late 19th Century. Floods that occurred in 1861-62 were the most severe known during the last half of the 19th century. Flooding on the valley floor was deep enough to permit riverboats to reach almost any locality in the inundated area (USACE, 1975). The “Great Flood” of 1862 was remarkable for the exceptionally high stages reached on most streams, repeated large floods, and prolonged and widespread inundation in the San Joaquin Valley (SJAFCA, 2013).

b. Early 20th Century. The major floods that occurred in the earlier part of the 20th Century (March 1907, January 1909, January-February 1911, and January 1921) were all very similar on their impact on the study area (USACE, 1975). In the Calaveras system, flooding was widespread, frequently extending across the area between Mormon Slough and the Calaveras River in the vicinity of Linden, which was entirely flooded a number of times during the period (USACE, 1975). Subsequent to construction of the Stockton Diverting Canal in 1910, floodwater ponded on its north side and extended far to the north and east (USACE, 1975). In 1911 floodwater extended in a solid sheet west from the Southern Pacific crossing of Mormon Slough to the Diverting Canal, a distance of about 7 miles. During that flood the levee along the south side of the Diverting Canal was overtopped. During all the floods of the first quarter of the 20th century, the study area was frequently described as an inland sea (USACE, 1975).

c. February 1938. Completion of New Hogan Dam and Reservoir in 1936 had a tempering effect on flooding in the study area. A flood that would have reached major proportions was largely averted by the project in February 1938. Runoff was estimated to be the greatest since 1911, but detention of floodwater in the reservoir and opportune cold weather and snowfall in the mountains, which halted runoff, limited overflow in the study area to such an extent that only a few roads were closed at the Diverting Canal and flood damage was minimal (USACE, 1975). The 1938 flood on Bear Creek was severe and a large area was inundated in the vicinity of the

Highway 99 crossing. Levees in the Delta breached on Mandeville, Quimby, Rhode, and Venice Islands and Pescadero and Stewart Tracts. A total of about 21,000 acres were inundated. The 100-acre Rhode Island was never reclaimed. Franks Tract was flooded and never reclaimed (SJFCA, 2013).

d. December 1950. The December 1950 flood was the fourth largest unregulated peak flow recorded at the San Joaquin River at Vernalis Gage from 1930 to 2010. The following description of the December 1950 flood is provided in the reference USACE, 1975. A series of unusually severe storms from November 13 to December 8, 1950 resulted in extensive flooding in the study area in early December. Rainfall which extended to high elevations in the Sierra Nevada and melted most of the shallow snowpack, averaged 31.58 inches over the major tributary areas of the San Joaquin River and totaled 15 inches over the tributary areas of Littlejohns and Duck Creeks. Regulation of runoff to the lower San Joaquin River was such that flow was not exceptionally great in November. In early December, however, upstream reservoirs were nearly full or already spilling, and maximum releases were being made to maintain flood control space. The result was a record breaking 79,000 cubic foot per second flow at Vernalis on December 9. High flows, combined with the highest tides in 10 years, breached the east levee along the San Joaquin River and inundated a large part of Reclamation District 17. Ultimately, most of the study area west of Highway 50 (now Interstate 5) and French Camp road was inundated. Floodwaters remained on the land for as long as 2 weeks and were reported as 17.5 feet deep in the vicinity of Mossdale.

San Joaquin River floodwater inundated thousands of acres of prime farmland, forced the evacuation of about 2000 persons from rural residences, closed and severely damaged highways and roads, inundated the County Honor Farm and threatened the County Hospital. Flood damage totaled about $900,000 in Reclamation District 17. Agricultural losses (about 750,000) included damage to crop and pasture land by erosion, deposition of sand and debris, and weed infestations; damage to farmsteads, including irrigation facilities; destruction of livestock and poultry; increased cost of upkeep and operation, and the cost incurred for protection, evacuation, cleanup and reconstruction.

Calaveras River floodwaters did not contribute to flooding in the study area. Duck Creek overflow inundated residential areas on the edge of Stockton and forced the evacuation of about 300 families. Runoff from Littlejohns and Duck Creeks caused high flows in Walker and French Camp Sloughs where extensive sandbagging was required to prevent overflows and further inundation. Flow in French Camp Slough also threatened the County Hospital which was enclosed by a temporary ring dike, and ultimately protected from flooding by a cut made in the slough levee to prevent breaching or overtopping and flooding south towards the hospital.

The west levee of Paradise Cut breached, causing Delta flooding on the Pescadero Tract and the Stewart Tract, and washed out the Southern Pacific Railway tracks. Levees breached and flooded 3,220 acres on Venice Island and 5,490 acres on Webb Tract. (SJFCA, 2013).

e. December 1955. The December 1955 flood was the second largest unregulated peak flow recorded at the San Joaquin River at Vernalis Gage from 1930 to 2010. Photographs of 1955 flooding within the study area are provided in Plates 16 and 17. The following description of the

1955 flood is presented in the effective FEMA Flood Insurance Study. In December of 1955, approximately 1500 acres along Mormon Channel were inundated by floodwaters breaking out of Mormon Slough. Residential and commercial damage in Stockton amounted to $1,500,000. Damage to utilities and public facilities such as roads and streets totaled about $370,000. During the flood, 3000-3500 residents of Stockton were evacuated from their homes, traffic was severely interrupted and telephone service was disrupted. About $250,000 was spent to aid flood victims. The floodwaters remained in the city for as long as 8 days and reached a depth of 6 feet in some areas. In total, 125 city blocks were flooded; the most severely damaged area was south of Charter Way and east of French Camp Turnpike. The flood occurred prior to flood management improvements made to Calaveras River, Mormon Slough, Duck Creek, Littlejohn Creek, Farmington Dam, and the New Hogan Dam and Reservoir. Therefore, the flood does not reflect existing hydrologic conditions.

f. April 1958. The following description of the April 1958 flood was obtained from USACE, 1975. During the 1958 floods, runoff on the Calaveras River was the greatest experienced since 1911. Hogan Reservoir filled and spilled for the first time since its completion in 1936. In total, about 22,000 acres in the study area were flooded. Most of the area was farm, crop and orchard land except for some developing rural residential and commercial areas along Highway 99 and north of the Diverting Canal. About 3,000 acres of farmland in the vicinity of Linden were flooded by the Calaveras River where two levee breaks occurred. Linden was threatened but not damaged. Levees along Mormon Slough were breached in a number of locations and about 7,000 acres of land flooded in a strip extending from Bellota to the Diverting Canal. A major levee break occurred near the head of the Diverting Canal. Flooding also occurred on 1500 acres along the north side of the Diverting Canal. About 11,000 acres were flooded by Bear Creek; the areas inundated extended across the entire study area and ranged from about 3 miles wide in the upper portion to about 5 miles wide at Highway 99. Floodwaters averaged about 2 feet deep and remained on the land for 2-10 days in the Calaveras River portion of the study area. They reached a maximum depth of 3 feet and remained on the land for as long as 3 weeks in the Bear Creek portion.

g. December 1964-January 1965. Widespread flooding occurred in northern and central California and western Nevada in December 1964 and January 1965. Severe storms occurred over the watershed tributary to the study area. However flooding and flood damage was minimal because the levee and channel improvement project was nearly finished at the time and functioned effectively to prevent an estimated $500,000 damage to agricultural and suburban residential developments. Flood losses in the Bear Creek study area during the flood period consisted of minor damage to electrical utility facilities and cost of levee repair. New Hogan Lake, which became operational just prior to the flood season stored runoff from a moderate large flood and controlled flows downstream to non damaging amounts.

h. November 1982 - March1983. Water year 1983 was a result of the “El Niño” weather phenomenon. Northern and Central California experienced flooding incidents from November through March due to numerous storms. In early May, snow water content in the Sierra exceeded 230 percent of normal, and the ensuing runoff resulted in approximately four times the average volume for Central Valley streams. Reservoir releases into the Delta resulting in prolonged high waters over period of weeks with very high Spring Tide peaks. Venice Island subsequently failed on November 30th and Mildred and Shima Tracts in January. High Lower SJR flows in March from continuing rainfall and snowmelt led to flooding of RD2064 at the confluence of the Stanislaus and San Joaquin Rivers (SJFCA, 2013).

i. February 1986. Local runoff and releases from New Hogan Dam during the February 1986 flood produced a short duration peak of 16,700 cfs in Mormon Slough at Bellota (USACE, 1999). This flow exceeded the design capacity of 12,500 cfs by 4,200 cfs, but remained in the channel. New Hogan Dam held back the majority of the volume, preventing extensive flooding downstream. Without New Hogan Dam, peak flows at Bellota could have been as high as 40,000 cfs.

The peak flow at Bellota exceeded 12,500 cfs during the February 1986 flood because a portion of the release from New Hogan Dam contributed to the peak flows at Bellota before releases could be reduced to minimum flow. Releases ranged from 6,000 cfs several hours prior to the peak at Bellota to 2,000 cfs during the peak. (The travel time from the dam to Bellota is about three hours). However, the flows above 12,500 cfs occurred for only a very short duration and therefore no failures or major damages were experienced.

Since 1986, several improvements have benefitted flood control operation of New Hogan Dam. A real-time model of the river above Bellota was developed and a telemetered gage was installed on Cosgrove Creek, a tributary just downstream of New Hogan Dam. The real-time flow at the Cosgrove Creek location provides a good indication of timing and magnitude of downstream local flows.

j. January 1997. December 1996 was one of the wettest Decembers on record. Watersheds in the Sierra Nevada were already saturated by the time three subtropical storms added more than 30 inches of rain in late December 1996 and early January 1997. The third and most severe of these storms lasted from December 31, 1996, through January 2, 1997. Rain in the Sierra Nevada caused record flows that stressed the flood management system to capacity in the Sacramento River Basin and overwhelmed the system in the San Joaquin River Basin. Emergency releases from Friant and Don Pedro Dams occurred on the San Joaquin River system. RD 2095, 2058, 2107 & 2062 on the west bank of the San Joaquin River all flooded in 1997. Major flood fight efforts on Mokelumne and Lower San Joaquin Rivers with lesser event in the tidal Delta (SJFCA, 2013). Photographs of flooding upstream of RD17 are provided in Plate 10.

k. December 2005 - January 2006. Between 28 December 2005 and 9 January 2006, the State of California experienced a series of severe storms which impacted the levees within the Sacramento District's boundaries. Water rose a second time in April 2006, and remained high in some parts of the system until June. Many rivers and streams within the Sacramento and San Joaquin River systems ran above flood stage during these events, and there were significant erosion and seepage problems with the levees. The State of California Department of Water Resources and/or their maintaining agencies conducted the actual flood fight activities while the U.S. Army Corps of Engineers provided technical assistance to the State.

4.0 ALTERNATIVE 1 (No Action Plan)

4.1 Hydraulic Design Summary

The no action alternative is based on the without project conditions and does not include the new project features. The following describes the assumptions used to evaluate the existing conditions.

a. General Design. All project features in the no action plan assumed to be the same as existed in 2014.

a. Levee Design Height. All existing levees are assumed to be maintained to the existing height or federally authorized height (federal project levees) whichever is higher. The design top of levee is based on the authorized design water surface profiles and the minimum freeboard specified in the Operations and Maintenance Manuals.

The San Joaquin River design water surface profiles are described in the drawing set, San Joaquin River and Tributaries Project, California, Levee Profiles, Drawing File Number SJ-20-30, 23 December 1955. The derivation of the 1955 water surface profiles is described in the general design memorandum. The 1955 design freeboard is described in the Operations and Maintenance manuals. The project adopted multiple existing levees of varying height. The Operations and Maintenance manuals indicates the adopted levee segments met or exceeded the design freeboard.

b. Upstream Reservoir Operation. The hydraulic analysis assumes all upstream reservoirs are operated as described in their respective water control manuals.

c. Interior Drainage Facilities. The hydraulic analysis assumes all drainage facilities are maintained to their design capacities.

d. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions.

e. Geotechnical Performance. The hydraulic analysis assumes the geotechnical performance is represented by the no action fragility curves presented in the geotechnical appendix to the feasibility study. The curves assess the probability of levee failure from under-seepage, through-seepage, stability, vegetation, animal burrows, encroachments, utilities, erosion, and judgment.

f. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (*Design of I-Walls*, 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority. The upstream end of the RD17 and French Camp slough tie back levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile rather than overtopped. The outflanking is considered to be a safer condition because it would occur only during the peak of the event and would reduce the flow and stage along the levee reaches.

g. Erosion Protection. The existing levee system includes erosion protection along several reaches.

h. Diversion structures. The Mormon Slough and Duck Creek diversion structures are assumed to be operated as described in the operations and maintenance manual.

4.2 Hydrology

Hydrology for the San Joaquin River was based on analysis conducted by the California Department of Water Resources (DWR) and USACE for the 2002 Sacramento-San Joaquin Comprehensive Study. Hydrology for the Calaveras River and Mormon Slough was based on analysis conducted for the feasibility study between 2010 and 2014 by the Local Sponsors and USACE and followed procedures compatible with the California Department of Water Resources Central Valley Hydrology Study (CVHS). The following provides a summary of the hydrologic flow frequency analysis utilized as inputs to hydraulic analysis. The hydrology appendix provides additional details.

a. San Joaquin River. The upstream boundary for the San Joaquin River hydraulic model is the USGS stream gage San Joaquin River near Vernalis. The drainage area at the stream gage is 13,536 square miles. Records at the USGS stream gage only account for flow in the channel and do not account for overbank flow. During large floods, flow on the waterside of the right bank levee outflanks the gage before discharging into the main channel at the RD17 tieback levee. Hydrologic frequency analysis presented herein accounts for all flow passing the gage, including channel and right overbank flow.

The Sacramento-San Joaquin Comprehensive study included the entire Sacramento and San Joaquin Valleys. Balanced 30-day regulated flow hydrographs developed for 50% (1/2) Annual Chance Exceedance (ACE), 10% (1/10) ACE, 4% (1/25) ACE, 2% (1/50) ACE, 1% (1/100) ACE, 0.5% (1/200) ACE, and 0.2% (1/500) was used in the hydraulic analysis.

The synthetic hydrology investigated unregulated flood frequencies at mainstem and tributary locations throughout the San Joaquin Basin. The flood frequency analysis involved evaluations of long term historical records at the stream gages. The unregulated flow frequency statistics and period of record for the San Joaquin River near Vernalis were used to estimate hydrologic uncertainty for San Joaquin River reaches within the study area. The adopted statistics and period of record for the unregulated conditions are provided in Table 10. A tabulation of the flood frequency estimates for flood durations between 1-day and 30-days is provided in Table 11.

Table 10
Rain Flood Frequency Statistics, San Joaquin River near Vernalis Unregulated Conditions

Flood Duration	Adopted Log Mean	Adopted Log Standard Deviation	Adopted Log Skew	Record (Years)	
				Years Evaluated	Years Used
1-Day	4.375	0.450	-0.1	1917 - 1998	82
3-Day	4.333	0.445	-0.1	1917 - 1998	82 (1/)
7-Day	4.251	0.433	-0.2	1917 - 1998	82
15-Day	4.148	0.412	-0.2	1917 - 1998	82
30-Day	4.042	0.392	-0.2	1917 - 1998	82

(1/) 82 year Equivalent Record adopted for use in FDA analysis

Table 11
Flood Frequency Flow Estimates, San Joaquin River near Vernalis Unregulated Conditions

Flood Duration	Duration Average Discharge by ACE (CFS)						
	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
1-Day	24100	88400	140300	188300	244700	310400	412900
3-Day	21900	79100	124900	167000	216500	273900	363100
7-Day	18400	62500	95200	124000	156500	193000	247300
15-Day	14500	46400	69200	89000	111100	135600	171700
30-Day	11400	34300	50200	63800	78700	95200	119200

The Comp Study formulated 5 mainstem and 22 tributary storm centerings to represent the many different possibilities of aerial storm distributions and antecedent watershed conditions. For each centering, synthetic 30-day natural flow hydrographs were computed at locations throughout the Central Valley. Typically, each tributary basin was composed of several hydrographs representing inflow to headwater dams, flood control dams, and local flow. The various hydrographs were then routed to specific index points to create an unregulated hydrograph (such as San Joaquin River at Vernalis). These natural flow hydrographs represent flood time series produced by a wholly unimpaired drainage area. The unimpaired hydrographs do not reflect the influence of headwater reservoirs. The hydrographs were balanced so the average flow for all durations matched the given frequency. For example, the peak, 1-day, 3-day, 5-day, 15-day, and 30-day volumes match the family of unregulated frequency curves computed for this location.

To simulate existing conditions, a 3-step process was required to conduct simulations of reservoir regulations for each storm centering. To begin the sequence, the headwaters reservoirs upstream of the flood control reservoirs were simulated. Then, using the resulting storage time series for select headwater facilities, top of conservation storage for those flood damage reduction projects with established credit space agreements were computed. Next, using the results of the headwater simulations and the computed top of conservation series, the lower basin reservoir models were simulated, thereby completing the reservoir simulation procedure.

A regulated set of hydrographs was obtained from "hand off" points in the lower basin reservoir simulation model. These hydrographs were then used as input to a UNET unsteady flow hydraulic model of the San Joaquin River. A review of the mainstem storm centerings found that the highest peak stages along the San Joaquin River within the study area are generated by the San Joaquin River at Vernalis storm centering. Therefore, hydraulic models for only one centering were evaluated in the feasibility study.

The sensitivity of downstream peak flows to upstream levee failures was conducted to determine if it would have a significant impact the evaluation of flood risk. The model was run for three different upstream levee failure scenarios.

- Infinite levee with no overtopping (Infinite). This is considered the extreme high estimate of peak flow and stage related to levee assumptions because no floodplain storage is allowed. All flow is confined to the leveed channel.

- Overtopping without Failure (No Fail). This model assumed all levees would overtop but would not fail. This may not be the most likely condition because some levees would likely fail prior to overtopping (probability of failure indicated by the fragility curve).

- With levee failure condition (With Fail). This model assumed all levees would fail at the 50% fragility point. This may not be the most likely condition because not all levees would fail at the 50% fragility point during the same flood.

A comparison of peak flows for the different levee overtopping assumptions is described in Table 12. The comp study models were only run for floods larger than 10% ACE.

Table 12
Sensitivity of Upstream Levee Failures, San Joaquin River near Vernalis
Regulated Conditions

	Peak DIscharge by ACE (CFS)						
Levee Scenario	**50% ACE**	**10% ACE**	**4% ACE**	**2% ACE**	**1% ACE**	**0.5% ACE**	**0.2% ACE**
Infinite Levee	NA	36900	47000	58400	90800	145500	233700
No Failure	NA	35100	42300	47700	78200	144500	224100
With Failure	NA	32900	43000	50300	77300	113300	166600
Source: 2002 Sacramento-San Joaquin Comprehensive Study UNET model results.							

The peak flow of infinite height assumption was found to always be greater for a given ACE event. The greatest difference between infinite height and no fail scenarios occurred at the 2% (1/50) ACE to 1% (1/100) ACE event which is probably around the flood magnitude that most system levees are overtopped. The No-Fail and With-Fail conditions are similar for floods smaller than 1% (1/100) ACE. The No-fail is larger than the with-fail condition for floods larger than 1% (1/100) ACE. The most likely condition is probably between the no-fail and with-fail conditions. The with-failure scenario also describes the relatively small influence that upstream transitory storage would have on reducing peak flows within the study area for floods as large as a 1% (1/100) ACE.

The overtopping with no failure scenario for areas outside the project area was adopted as the most likely hydraulic condition for this study to support the risk analysis. The probability of overtopping levee failure within the study area is accounted for in the FDA model using a fragility curve that assumes 100% failure probability at the levee crest. This assumption helps make a breach probability more statistically independent rather than dependent on each other and is consistent with historical observations that the probability of a breach does not appear to be highly dependent on other breaches occurring. There is no specific guidance on how to apply overtopping assumptions to system wide risk analysis. However, the approach taken is consistent with EM 1110-2-1619. The overtopping without failure assumption for areas outside the project area is also consistent with the DWR Urban Levee Design Criteria and FEMA mapping approaches.

A table of adopted regulated peak flows for this study is provided in Table 13. Due to upstream conditions, hydrographs for channel and right overbanks are required for events greater than a 1% (1/100) ACE event. A period of record of 82-yrs should be utilized in performance analysis to account for uncertainty in estimating the unregulated flow at Vernalis. A plot of the resulting flood frequency estimates and historical regulated flows is provided as Plate 18.

Table 13
Flood Frequency Flow Estimates, San Joaquin River near Vernalis
Regulated Conditions

	Peak Discharge by ACE (CFS)						
Peak Flow	**50% ACE**	**10% ACE**	**4% ACE**	**2% ACE**	**1% ACE**	**0.5% ACE**	**0.2% ACE**
Channel	6400	35100	42300	47700	78200	124600	165200
Right Overbank	0	0	0	0	0	20400	60500
Total	6400	35100	42300	47700	78200	144500	224100
Note: Time of peak channel flow is different than time of peak overbank flow. As a result, the peak total flow is not equal to the sum of the channel peak flow and overbank peak flow.							

The California Department of Water Resources is currently conducting a study of Central Valley Hydrology. The Central Valley Hydrology Study (CVHS) will provide more recent hydrologic frequency estimates throughout the study area. However, the results were not finalized at the time of this study. The draft flood frequency estimates from the CVHS study were compared to the comp study estimates and found to be similar.

b. Calaveras River and Mormon Slough. The upstream hydraulic model boundary for and Calaveras River and Mormon Slough is the USACE stream gage Mormon Slough at Bellota. The drainage area at the gage is 470 square miles. Hydrologic analysis is described in the hydrology appendix dated April 2014. Flood frequency curves and a suite of 10-day hydrographs were developed for the Mormon Slough at Bellota gage. The unregulated frequency analysis was performed with PeakfqSA software which uses the Expected Moments Algorithm (EMA) and Multiple Grubbs Beck outlier test. The method is approved for use by HQ USACE. The period of record analyzed is 104 years from 1907 to 2010. Unregulated flow frequency statistics for the Mormon Slough at Bellota Gage are provided in Table 14. Unregulated discharges by frequency and duration are provided in Table 15.

Table 14
Rain Flood Frequency Statistics, Mormon Slough at Bellota
Unregulated Conditions

Flood Duration	Adopted Log Mean	Adopted Log Standard Deviation	Adopted Log Skew	Record (Years)	
				Years Evaluated	Years Used for Statistics
1-Day	3.775	0.482	-0.810	1907 - 2010	104 (1/)
3-Day	3.608	0.475	-0.753	1907 - 2010	104
7-Day	3.417	0.464	-0.666	1907 - 2010	104
15-Day	3.240	0.461	-0.671	1907 - 2010	104
30-Day	3.079	0.448	-0.668	1907 - 2010	104
(1/) To account for local inflow uncertainty, 52 year Equivalent Record adopted for use in FDA analysis					

Table 15
Flood Frequency, Mormon Slough at Bellota
Unregulated Conditions

Flood Duration	Duration Average Discharge by ACE (CFS)						
	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
1-Day	6900	21700	29700	35300	40500	45400	51300
3-Day	4600	14600	20200	24200	28000	31600	36100
7-Day	2900	9300	13000	15800	18500	21100	24500
15-Day	2000	6100	8600	10300	12100	13800	16000
30-Day	1300	4100	5700	6800	7900	9000	10400

The analysis involved routing scaled versions of four large historic flood events (reservoir inflow plus local flow hydrographs) through an HEC-ResSim reservoir routing model. Four unregulated to regulated transforms were derived and then averaged to produce a final adopted peak regulated flow frequency curve. Selected regulated hydrographs at Bellota based on the 1997 flood pattern and matching the regulated peak flow frequency curve were adopted for input into HEC-RAS model for modeling specific frequency events at Bellota. A rainfall runoff model was used to derive concurrent local flow hydrographs as internal boundary conditions in the HEC-RAS hydraulic model reaches downstream of Mormon Slough at Bellota. A table of adopted regulated peak flows for this study is provided in Table 16. Although the frequency analysis utilized 104 years of record, an equivalent period of record of 52-yrs should be utilized in performance analysis to account for uncertainty in estimating the ungaged unregulated flow between New Hogan Dam and Bellota. A plot of the resulting flood frequency estimates and historical regulated flows is provided as Plate 19.

Table 16
Flood Frequency, Mormon Slough at Bellota
Regulated Conditions

	Duration Average Discharge by ACE (CFS)						
	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
Peak Flow	3520	9530	10640	12500	12500	12500	16000

d. Delta Stage-Frequency. A stage frequency analysis was conducted at four stage gages in the Sacramento-San Joaquin Delta that serve as downstream boundary conditions in the hydraulic models. The stage-frequency analysis was conducted for DWR stream gages; Old River at Clifton Court Ferry (B95340), Middle River at Bowden Highway (B95500), San Joaquin River at Ringe Pump (B95620), and Stockton Ship Channel at Burns Cutoff (B95660) . Stage-frequency estimates were developed for future sea level conditions including 2010 and 2070. The frequency analysis is described in detail in the USACE Memorandum for File, Delta Stage-Frequency Analysis for Alternative Comparisons, 9 May 2014 (USACE, 2014A). The stage frequency curves are provided as Plate 20 and Tables 17 and 18. A map of the study area showing gage locations is presented in Plate 21.

The stage frequency analysis was based on stage data from the period from 1953 to 2009. Historical peak stages would have been higher under existing (2010) sea level conditions. Historical stage data were adjusted to 2010 sea level conditions for use in the frequency analysis. Each data set was adjusted by increasing historical recorded elevations to 2010 conditions using the eustatic rate of sea level rise of 0.0056 ft/yr (1.7mm/yr). The rate of eustatic sea level rise was obtained from EC 1165-2-212 and agrees with the reported value in NOAA, 2013 as the estimated rate of sea level rise over the 20th century.

Graphical stage-frequency curves were developed for each gage by plotting the historical stage records using Weibul plotting positions. Extrapolation of the stage frequency curves from 2% ACE to 0.2% ACE events was based on hydraulic model simulations of the San Joaquin River system. For larger flood events the stage-discharge relationship at each gage was based on DSM2 model results presented in the March 2002 report “Sacramento and San Joaquin River Basins Comprehensive Study, Existing Hydrodynamic Conditions in the Delta during Floods”. These relationships between stage and flow at each gage site are currently the best available analysis of hydraulic conditions in the delta for extreme flood events. While suitable for economic analysis, estimates should be refined for design purposes.

Future Sea level Rise was computed following the method outlined in EC 1165-2-212 for three scenarios. Curve I is based on the historical rate of sea level rise. Curve II reflects an intermediate estimate of the future rate of sea level rise. Curve III reflects a high estimate of the future rate of sea level rise. The rates are provided in Table 19. The Curve II rates were used to estimate future increases in sea level over the period 2010 through 2070. The rates provided for Curve I and Curve III are provided to describe the sensitivity of future sea level estimates to this assumption. Future sea level rise was assumed to impact all flood frequencies the same amount because the Delta consists of a network of channels that would have similar hydraulic characteristics for higher sea level conditions.

Table 17

Mean Stage estimates by Annual Chance of Exceedance, No Action Alternative 2010 Sea Level Conditions

ACE	Mean Stage (Feet-NAVD88)			
	Old River at Clifton Court Ferry (B95340)	Middle River at Borden Hwy (B95500)	Stockton Ship Channel at Burns Cutoff (B95660)	San Joaquin River at Ringe Pump (B95620)
0.002 (1/500)	13.08*	11.20*	13.01*	12.91*
0.005 (1/200)	12.12*	9.90*	12.12*	12.02*
0.010 (1/100)	11.44*	9.80*	10.10*	10.00*
0.020 (1/50)	9.95	9.57	9.90	9.80
0.040 (1/25)	9.75	9.50	9.70	9.60
0.100 (1/10)	9.35	9.10	9.30	9.20
0.200 (1/5)	8.70	8.55	8.70	8.60
0.300 (1/3)	7.70	7.80	8.15	8.05
0.500 (1/2)	7.15	7.25	7.70	7.60
0.950 (1/1.05)	6.35	6.45	6.70	6.60

* Stage estimates for events larger than 0.02 (1/50) ACE are based on hydraulic model extrapolation. While suitable for economic analysis, estimates should be refined for design purposes.

Future Sea Level based EC 1165-2-212 Curve II. Curve I and III estimates can be computed using values in Table 19.

Table 18
Mean Stage estimates by Annual Chance of Exceedance, No Action Alternative 2070 Sea Level Conditions

ACE	Mean Stage (Feet-NAVD88)			
	Old River at Clifton Court Ferry (B95340)	Middle River at Borden Hwy (B95500)	Stockton Ship Channel at Burns Cutoff (B95660)	San Joaquin River at Ringe Pump (B95620)
0.002 (1/500)	14.74*	12.86*	14.67*	14.57*
0.005 (1/200)	13.78*	11.56*	13.78*	13.68*
0.010 (1/100)	13.10*	11.46*	11.76*	11.66*
0.020 (1/50)	11.61	11.23	11.56	11.46
0.040 (1/25)	11.41	11.16	11.36	11.26
0.100 (1/10)	11.01	10.76	10.96	10.86
0.200 (1/5)	10.36	10.21	10.36	10.26
0.300 (1/3)	9.36	9.46	9.81	9.71
0.500 (1/2)	8.81	8.91	9.36	9.26
0.950 (1/1.05)	8.01	8.11	8.36	8.26

* Stage estimates for events larger than 0.020 (1/50) ACE are based on hydraulic model extrapolation. While suitable for economic analysis, estimates should be refined for design purposes.

Future Sea Level based EC 1165-2-212 Curve II. Curve I and III estimates can be computed using values in Table 19.

Table 19
Sea Level Rise from 2010 Conditions

Year	Sea Level Rise from 2010 Conditions (Feet)		
	Curve I (Sensitivity)	Curve II (Adopted)	Curve III (Sensitivity)
2010	0.00	0.00	0.00
2015	0.05	0.07	0.10
2020	0.10	0.16	0.23
2025	0.15	0.26	0.37
2030	0.21	0.37	0.53
2035	0.28	0.49	0.70
2040	0.34	0.62	0.90
2045	0.42	0.77	1.12
2050	0.49	0.92	1.35
2055	0.58	1.09	1.60
2060	0.66	1.27	1.87
2065	0.75	1.46	2.16
2070	0.85	1.66	2.47

Rate of Sea Lever Rise based on EC 1165-2-212

e. Interior Drainage. An interior drainage analysis was performed by Peterson-Brustad Incorporated (PBI) for Bear Creek, Mosher Creek, and French Camp Slough sub-basins impacting the study area. A storm centered over the urban area of Stockton was utilized for the analysis. The interior drainage analysis evaluated rainfall runoff and flood depths for 50% (1/2) ACE through 0.2% (1/500) ACE flood events. Storm events with 72-hour durations were evaluated. The analysis utilized an HEC-HMS model to compute sub basin runoff and a FLO-2D two dimensional hydraulic model to route the runoff through the study area. The results indicated that residual damages from interior drainage would not influence alternative selection and would not meet the 800cfs rule. In addition, the analysis indicated that damages from interior drainage are negligible in comparison to flooding from the principle sources of flooding described in this report. Therefore, interior drainage was not examined in detail for this study.

4.3 Hydraulic Models

Four separate hydraulic models, adapted from existing hydraulic models, were utilized to evaluate the no action plan for this study. Water surface profiles for the San Joaquin River were computed using an HEC-RAS unsteady one-dimensional flow model of the San Joaquin River system. The model extents are shown on Plate 21. Water surface profiles for Calaveras River and Mormon Slough were computed using an HEC-RAS unsteady flow model of the system. The model extents are shown on Plate 22.

Flooding was only modeled for breach locations impacting the economic impact areas shown in Plate 4. The selection of the breach locations was based on analysis conducted during plan formulation screening. The breach locations were selected to single out the primary sources of comingled flooding within the study area. Flood risk to areas outside these economic impact areas was found unlikely to support federal interest. The selection of the study area is described in the Feasibility Study report. Levee breach simulations for the area North of French Camp Slough were conducted using the North FLO-2D model shown on Plate 23. Levee breach simulations for the area south of French Camp Slough were conducted using the south FLO-2D model and are shown on Plate 24.

The computer model HEC-RAS calculates steady or unsteady gradually varied flow in natural and manmade channels by performing step-backwater calculations of the 1-D flow energy equation through a series of input geometric cross-sections with empirically defined hydraulic roughness coefficients. The computer model FLO-2D is a 2-dimensional, dynamic flood routing model that simulates movement of water across the ground surface while reporting volume conservation. It numerically routes flood hydrographs over a system of grid elements, and predicts the area of inundation and flood wave attenuation.

Without project conditions were evaluated using an uncoupled 1-d and 2-d modeling approach that has been standard procedure on multiple studies within the Sacramento District. River stages and profiles and breaches were simulated using an HEC-RAS model because RAS incorporates more detailed hydraulic capabilities for channel flow and breaches. The breach outflow hydrographs were then transferred to a 2-dimensional FLO-2D model of the floodplain. The FLO-2D model has more detailed capabilities than HEC-RAS for simulating the distribution of the breach hydrographs on the floodplain. This process leverages the most robust capabilities of both models.

a. San Joaquin River. Water surface profiles and breaches for the San Joaquin River were computed using an HEC-RAS unsteady one-dimensional flow model of the San Joaquin River system. The origin of the model was the HEC-UNET model developed as part of the 2002 comp study. The model was updated to HEC-RAS by the California Department of Water Resources for use in Task Order 120 (TO120) of the Central Valley Flood Protection Plan (CVFPP). The model was updated to address the needs of the feasibility study. The primary updates were to extend the model downstream to three stage gages in the Sacramento San Joaquin Delta and truncate the upstream end of the model at the Vernalis gage. A map of the HEC-RAS hydraulic model domain is provided as Plate 21. A detailed description of the changes made to the model is provided in the Technical Memorandum, San Joaquin River Main Stem HEC-RAS model setup by Peterson Brustad Incorporated, 13 September 2013 (PBI, 2013A).

(1) Cross Sections. The model contains a total of 530 cross sections. The cross sections are spaced at roughly ¼-mile intervals along the river reaches. Cross section geometry data were obtained from the 2002 Sacramento-San Joaquin Comprehensive Study and updated to the NAVD88 datum using conversion values in the NGS Vertcon computer program.

(2) Storage Areas. The model contains a total of 31 storage areas throughout the domain.

(3) Bridges and Inline Structures. The model contains a total of 25 bridges, 1 inline structure and 1 major weir diversion (Paradise Dam).

(4) Lateral Structures (Levees). The HEC-RAS model utilizes the lateral weir option to simulate overtopping of the levee crest. The structures were manually coded into each HEC-RAS model based upon Top of Levee (TOL) elevation data from the USACE National Levee Database (NLDB) survey data. The lateral structure outflow is linked to the storage areas described above.

(5) Blocked Obstructions. Blocked obstructions were used throughout the model to eliminate the cross section area on the landward side of the levee. The landward areas are modeled as storage areas and lateral weirs along the crest of the levee control the flow over and into and out of the storage areas. The blocked obstructions are needed because the cross sections extend approximately 100 feet landward of the levee and this is not a conveyance area under this approach. The levee card is not suitable in this case because the conveyance area on the landward side of the cross section would become conveyance area once overtopped. The heights of the blocked obstructions were made sufficiently high to insure the levee overtopping was consistent with the lateral structure levee approach described above.

(6) Ineffective Flow Areas. Ineffective flow areas were incorporated into the model to simulate areas where water is stored, but is not an active conveyance area.

(7) Manning’s Roughness Values. Manning’s n-values provided in the source model by DWR were adopted for this study. The model calibration is described in the DWR

documentation described above. Values were selected based on model calibration to high water marks collected during the March 1995 event. Boundary condition inflows for the model calibration were based on DWR and USGS stream gage records. Manning's roughness values range from 0.035 to 0.58 in the main channel and 0.042 to 0.110 in the overbanks.

(8) Upstream Boundary Conditions. Upstream boundary conditions are a set of regulated flow hydrographs for the Channel and Right Overbank at Vernalis. The channel and right overbank flow split were obtained from the 2002 Sacramento-San Joaquin Comprehensive Study UNET model.

(9) Downstream Boundary Conditions. The model includes three downstream stage-discharge rating boundary conditions; 1) Old River at Clifton Court Ferry 2) Middle River at Bowden Bridge, and 3) Stockton Deep Water Ship Channel at Burns Cutoff . The stage-discharge rating curves were developed through an initial set of model runs. For each ACE flow event a constant stage with the same ACE stage was set at each of the downstream boundary conditions. The system model was then run to determine the peak computed flow at each downstream boundary for the ACE event. The resulting peak stage and peak flow formed an ordinate of the final stage-discharge curve. This process was repeated for 50% ACE through 2% ACE events.

For larger flood events the stage-discharge relationship at each gage was based on DSM2 model results presented in the March 2002 report "Sacramento and San Joaquin River Basins Comprehensive Study, Existing Hydrodynamic Conditions in the Delta during Floods". These relationships between stage and flow at each gage site are currently the best available analysis of hydraulic conditions in the delta for extreme flood events. The resulting combined stage-discharge relationships define the downstream boundary conditions of the hydraulic model.

The development of the stage-frequency curves is described in the hydrology section above. Models were developed assuming 2010 and 2070 sea level conditions at the downstream boundary condition.

(10) Model Calibration. The model was calibrated to the March 1995 flood event. Details on the model calibration are provided in DWR, 2009.

(11) Stage Uncertainty. The total SD of stage uncertainty was computed at the four index points along the San Joaquin River. A SD of 1.5 feet is recommended for all reaches of the San Joaquin River.

Stage uncertainty was estimated following methods described in EM-1110-2-1619. The total stage uncertainty was estimated from natural and model uncertainty. A detailed description of the stage uncertainty analysis is provided in the 13 September 2013 Technical Memorandum San Joaquin River Main Stem HEC-RAS modeling by Peterson Brustad Inc. (PBI, 2013A). The standard deviation (SD) of total stage uncertainty was calculated using the following equations modified from EM1110-2-1619.

$$\mathrm{SD}_{\mathrm{total}} = \sqrt{\mathrm{SD}_{\mathrm{natural}}^2 + \mathrm{SD}_{\mathrm{model}}^2}$$

The natural uncertainty, *SD natural*, was computed using the equation provided in EM-1110-2-1619. The equation is based on streambed type, drainage area, maximum expected stage range, and 1% ACE discharge. The model uncertainty, *SD model*, was estimated using Table 5-2 of EM 1110-2-1619. Because several sections of the Main Stem HEC-RAS model have not been calibrated, Manning's n reliability was judged to be "Poor". Topography for the model is relatively accurate and is primarily based on Comp Study surveys and CVFED LiDAR and bathymetry data. With these parameters, the minimum *SD model* value was estimated at 1.3 feet.

b. Calaveras River and Mormon Slough. Water surface profiles for Calaveras River and Mormon Slough system were computed using an existing draft version of an HEC-RAS steady one-dimensional flow model. The draft model was developed under the California Department of Water Resources (DWR) Central Valley Floodplain Evaluation and Delineation (CVFED) program. The model was reviewed and modified for the Feasibility Study by Peterson Brustad Incorporated (PBI). Development and review of the model is described in the PBI Technical Memorandum "Review and Update of the CVFED Calaveras River HEC-RAS Model, 9 September 2013 (PBI, 2013B). A map of the HEC-RAS hydraulic model domain showing cross sections and hydrograph boundary locations is provided as Plate 22. The hydraulic model extends from Belota to the San Joaquin River.

(1) Cross Sections. The model contains 425 cross sections with an average spacing of 500 feet. Cross section geometry data were obtained from the LiDAR data acquired by the State of California for their Central Valley Floodplain Evaluation and Delineation (CVFED) program. The data were collected over several weeks between March 17, 2008 and April 4, 2008. The underwater portion of each cross section was adjusted to reflect recent NAVD88 ground surveyed bathymetric cross section data obtained by the State of California Department of Water Resources in 2010.

(2) Storage Areas. The model includes 14 storage areas to account for overland flooding. Storage areas were not defined for the entire study area because overbank flooding is transferred to a FLO-2D model of the floodplain area.

(3) Bridges and Inline Structures. The model contains 62 Bridges and 9 inline structures coded into the model from field surveys and sketches.

(4) Lateral Structures (Levees). The HEC-RAS model utilizes the lateral weir option to simulate overtopping of the levee crest. The structures were manually coded into each HEC-RAS based upon Top of Levee (TOL) elevation data from the USACE National Levee Database (NLDB) survey data. The lateral structure outflow is linked to the storage areas described above.

(5) Levees. The levee crest elevation was specified for each cross section. The top of levee elevation was obtained from the NAVD88 National Levee Database (NLDB) ground survey conducted in 2007-2008.

(6) Blocked Obstructions. Blocked obstructions were used throughout the model to eliminate the cross section area on the landward side of the levee. The landward areas are modeled as storage areas and lateral weirs along the crest of the levee control the flow over and into and out of the storage areas. The blocked obstructions are needed because the cross sections extend approximately 100 feet landward of the levee and this is not a conveyance area under this approach. The levee card is not suitable in this case because the conveyance area on the landward side of the cross section would become conveyance area once overtopped. The heights of the blocked obstructions were made sufficiently high to contain a 0.2% (1/500) ACE flood event.

(7) Ineffective Flow Areas. Ineffective flow areas were incorporated into the model to simulate areas where water is stored, but is not active conveyance area.

(8) Manning's Roughness Values. Manning's roughness values range from 0.030 to 0.35 in the main channel and 0.035 to 0.045 in the overbanks. The roughness values were based on limited calibration to high water observations made during a high-water event in 6 April 2006. High water mark staking was not available for the event. The calibration was based on photographs of the high water and anecdotal evidence.

(9) Upstream Boundary Conditions. The primary upstream boundary condition is the regulated flow at the San Joaquin River at Belota gage. Development of the inflow hydrographs is summarized in the hydrology section above. The model also includes inflows from localized drainage at internal boundary conditions throughout the model.

(10) Downstream Boundary Conditions. The downstream boundary condition was the stage-frequency relationship at the Stockton Deep Water Ship Channel at Burns Cutoff. The development of the boundary conditions is described in the 15 August 2013 technical memorandum, Delta Stage-Frequency Analysis for Alternative Comparisons by CESPK-ED-HA. Models were developed assuming 2010 and 2070 sea level conditions at the downstream boundary condition.

(11) Model Calibration. As described above, the model calibration to the 6 April 2006 event was limited by available information.

(12) Stage Uncertainty. The total SD of stage uncertainty was computed at seven index points along Calaveras River and Mormon Slough. A SD of 0.9 feet is to be used for all reaches of the Calaveras River and Mormon Slough system.

Stage uncertainty was estimated following methods described in EM-1110-2-1619. The total stage uncertainty was estimated from natural and model uncertainty. A detailed described in the PBI Technical Memorandum "Review and Update of the CVFED Calaveras River HEC-RAS Model, 9 September 2013 (PBI, 2013B). The standard deviation (SD) of total stage uncertainty was calculated using the following equations modified from EM1110-2-1619.

$$\mathrm{SD}_{\mathrm{total}} = \sqrt{\mathrm{SD}_{\mathrm{natural}}^2 + \mathrm{SD}_{\mathrm{model}}^2}$$

The natural uncertainty, *SD natural*, was computed using the equation provided in EM-1110-2-1619. The equation is based on streambed type, drainage area, maximum expected stage range, and 1% ACE discharge. The model uncertainty, *SD model*, was estimated using Table 5-2 of EM 1110-2-1619. The model calibration was estimated to result in a "fair" reliability of Manning's Roughness values. Topography for the model is relatively accurate and is primarily based on Comp Study surveys and CVFED LiDAR and bathymetry data. With these parameters, the minimum *SD model* value was estimated at 0.7 feet.

e. North FLO-2D Model. An existing FLO-2D model was utilized to evaluate water surface elevations resulting from levee breaches within the study area. The FLO-2D model was developed by HDR, Inc. as part of the Department of Water Resources' (DWR) Central Valley Floodplain Evaluation and Delineation (CVFED) program. The model underwent extensive quality control review by DWR and USACE. This model was used in the Feasibility Study to analyze levee breach scenarios at each of the 7 LSJRFS index points along the Calaveras River and Stockton Diverting Canal. A detailed description of the model is provided in the Technical Memorandum, San Joaquin Area Flood Control Agency, Two-Dimensional (FLO-2D) Hydraulic Model of the Lower San Joaquin River System. 3 December 2013. A map of the model domain is provided in Plate 23.

(1) Computational Domain**.** The valid computational domain is defined as the Lower San Joaquin Basin Feasibility study area. The model's domain extends beyond the valid computational domain in order to establish model boundary conditions. All results outside the valid domain were truncated from the results.

(2) Grid Elements**.** A 250-ft grid size was selected in order to keep the number of grid elements down to a workable number and to avoid long model run times. Model geometry was based on LiDAR data acquired by the State of California for their Central Valley Floodplain Evaluation and Delineation (CVFED) program. The data were collected over several weeks between March 17, 2008 and April 4, 2008.

(3) Channel Elements**.** The model includes channel elements for Bear Creek and its tributaries, Fivemile Slough, Mosher Slough, Calaveras River and Mormon Slough, Stockton Deep Water Ship Channel, and French Camp slough and its tributaries.

(4) Floodplain Roughness and Reduction Factors. Overland n-values and area reduction factors (ARF) were developed for a variety of different land uses. Values ranged from 0.04 to 0.11 within urban areas and 0.04 to 0.25 for non-urban areas. The model includes Area Reduction Factors (ARFs) to account for the reduction in storage associated with buildings. The model also includes Width Reduction Factors (WRFs) to account for the reduction in conveyance areas associated with buildings and other structures.

(5) Levees and Embankments. Levees and embankments are included in the model as FLO-2D levee features. However, channels with levees were modeled entirely as channel sections that included their levees as part of the channel.

(6) Hydraulic Structures. Hydraulic structures within the floodplain were coded into the FLO-2D model by adjusting the geometry or utilizing stage-discharge rating curves
.

(7) Pump Stations. The model does not include interior pump stations.

(8) Boundary Condition Inflows. The inflow hydrographs for the FLO-2D model consist of levee overtopping and breach hydrographs obtained from HEC-RAS model simulations.

(9) Boundary Condition Outflows. The purpose of the FLO-2D model is to simulate the movement of breach floodwaters within the study area on the interior side of levee system. Outflow elements were specified along the edge of the model boundary.

(10) Stage Uncertainty. Stage uncertainty was not computed for the FLO-2D model results. The FDA model only accounts for uncertainty in the channel stage-discharge relationship. The channel stage-discharge uncertainty is described in the HEC-RAS model description above.

e. South FLO-2D Model. An existing FLO-2D model was utilized to evaluate water surface elevations resulting from levee breaches within the study area. The FLO-2D model was developed by HDR, Inc. as part of the Department of Water Resources' (DWR) Central Valley Floodplain Evaluation and Delineation (CVFED) program. The model underwent extensive quality control review by DWR and USACE. This model was used in the Feasibility Study to analyze levee breach scenarios at each of the 4 LSJRFS index points along the Lower San Joaquin River. A detailed description of the model is provided in the Technical Memorandum, Lower San Joaquin River and Tributaries Two-Dimensional (FLO-2D) Hydraulic Model of the Lower San Joaquin River System. 20 November 2013. A map of the model domain is provided in Plate 24.

(1) Computational Domain**.** The valid computational domain is defined as the Lower San Joaquin Basin Feasibility study area. The model's domain extends beyond the valid computational domain in order to establish model boundary conditions. All results outside the valid domain were truncated from the results.

(2) Grid Elements**.** A 400-ft grid size was selected in order to keep the number of grid elements down to a workable number and to avoid long model run times. Model geometry was based on LiDAR data acquired by the State of California for their Central Valley Floodplain Evaluation and Delineation (CVFED) program. The data were collected over several weeks between March 17, 2008 and April 4, 2008.

(3) Channel Elements**.** The model includes channel elements for the San Joaquin River and tributaries.

(4) Floodplain Roughness and Reduction Factors. Overland n-values and area reduction factors (ARF) were developed for a variety of different land uses. Values ranged from 0.04 to 0.20 for non-urban areas. The model includes Area Reduction Factors (ARFs) to account for the

reduction in storage associated with buildings. The model also includes Width Reduction Factors (WRFs) to account for the reduction in conveyance areas associated with buildings.

(5) Levees and Embankments. Levees and embankments are included in the model as FLO-2D levee features. However, the levees along the San Joaquin River were modeled entirely as channel sections that included their levees as part of the channel.

(6) Hydraulic Structures. Hydraulic structures within the floodplain were coded into the FLO-2D model by adjusting the geometry or utilizing stage-discharge rating curves

(7) Pump Stations. The model does not include interior pump stations.

(8) Boundary Condition Inflows. The inflow hydrographs for the FLO-2D model consist of levee overtopping and breach hydrographs obtained from HEC-RAS model simulations.

(9) Boundary Condition Outflows. The purpose of the FLO-2D model is to simulate the movement of breach floodwaters within the study area on the interior side of levee system. Outflow elements were specified along the edge of the model boundary.

(10) Stage Uncertainty. Stage uncertainty was not computed for the FLO-2D model results. The FDA model only accounts for uncertainty in the channel stage-discharge relationship. The channel stage-discharge uncertainty is described in the HEC-RAS model description above.

4.4 Hydraulic Model Results.

The hydraulic models described above were utilized to compute water surface profiles and breach simulations. Water surface profiles and breach simulations were performed for 50% (1/2) ACE, 10% (1/10) ACE, 4% (1/25) ACE, 2% (1/50) ACE, 1% (1/100) ACE, 0.5% (1/200) ACE, and 0.2% (1/500) events.

a. Water surface profiles. Computed water surface profiles for 2010 conditions are presented in Plates 25 for San Joaquin River, Plate 26 for Lower Calaveras River, Plate 27 for Upper Calaveras River, and Plate 27 for Mormon Slough. Computed water surface profiles for 2070 conditions are presented in Plates 29 for San Joaquin River, Plate 30 for Lower Calaveras River. The 2010 and 2070 profiles are identical for the other reaches. Stage-Discharge-Frequency plots at the index points within and outside the study area are shown in Plate31A through 31N and 32A through 32E respectively. The plots include stage estimates for 2010 and 2070 sea level conditions. The Stage-Discharge-Frequency plots also show with project conditions described later in this report.

b. Levee Breach Scenarios. Levee breaches are used to define the inundation if a breach were to occur. Breach simulations were conducted using two methods. A two dimensional method was used where the flood inundation is characterized as shallow unconfined type flooding. A simplified one dimensional level pool method was used for breach locations where the flooded area would equalize to a level water surface elevation. The breach simulation locations and formation parameters are shown on Plate 4 and Table 20.

(1) Two Dimensional Method: This method involved an uncoupled simulation using the one-dimensional HEC-RAS models and FLO2D models described above. A major assumption in this approach is the floodplain flows are not largely influenced by channel hydraulics except at the breach. Therefore, the uncoupled model approach is sufficiently accurate. The levee breach was simulated in a HEC-RAS hydraulic model of the system. The resulting breach hydrograph served as input to a FLO-2D model used to compute the inundation.

Breach formation parameters such as width and time to develop were estimated following the procedures described in the August 2013 Sacramento District Hydraulic Design report "Development of Levee Breach Parameters for HEC-RAS Application". The resulting inundation maps are hypothetical simulations of levee failures and do not represent the probability of occurrence. Breach simulations performed using the two dimensional method are shown on Plates 33A through 33J.

(2) One Dimensional Level Pool Method: This method was utilized for the Delta breach locations where the volume of the inundated area was relatively small with respect to the flow or stage hydrograph. The peak stage in the channel of the HEC-RAS model was assumed to define a level pool. The level pool was mapped using the FLO-2D floodplain elevation elements and computing the depth below the level pool for each grid element. This approach was used for breach simulations at index points D-BS, D3, D4, and D5 which are shown on Plates 34A through 34D.

Table 20
Levee Breach Simulation Parameters

Flood Source	Breach Location	Levee Height at Breach Location (Feet)	Breach Width (Feet)	Time to Develop full Breach (Minutes)	Economic Impact Area
San Joaquin River	LRTB	1/	1/	1/	RD17
	LR4	17.1	190	27	RD17
	LR3	18.8	210	29	RD17
	LR2	16.5	180	27	RD17
	LR1	16.8	190	27	RD17
French Camp Slough	FR1	14.0	155	25	CS-02
	FL1	12.2	1/	1/	RD17
Stockton Diverting Canal	SL1	10.7	118	22	CS-01,CS03
	SL2	10.7	118	22	CS-01,CS-02,CS-03
Calaveras River	CR2	8.0	88	19	NS-04, NS-03
	Cl2	8.5	94	19	CS-01,CS-02,CS-03
Delta Front	D3	11.2	2/	2/	NS-02
	D4	13.5	2/	2/	CS-01
	D5	13.4	2/	2/	NS-03
	D-BS	14.5	2/	2/	NS-03
1/ A breach at LR4 was used to simulate a breach at LRTB 2/ Delta breaches assumed level pool flooding.					

d. Natural Floodplains. Natural floodplains were developed to address planning requirements of ER 1165-2-26. The natural floodplains were developed by plotting the maximum inundation depth from all simulated breaches for a given ACE event. The inundation area represents the maximum extent of areas with potential risk of being flooded from the primary flood sources described in this study. The floodplains are provided in Plates 35 through 42. These floodplains include the effects of unnatural features in the floodplain (bridges, berms, roadways, levees). Therefore, they do not represent the actual “natural conditions”.

4.5 Wind Wave Analysis.

An analysis of wind wave run-up, wind setup, overtopping discharge, and wind wave erosion was conducted for levee reaches within the study area. Previous analysis for the Sutter Basin Feasibility study found that wind wave runup and setup were largely independent of water surface in the top 2/3 of the levee height. Therefore, wind wave runup and setup were computed assuming the top of levee stage. An assessment of stable rock diameter was also conducted to evaluate the potential for wind wave erosion. Estimated stable rock sizes are provided in Table 21. Results for wind wave run up and setup up for a hypothetical water level at the levee crest are summarized in Table 22. The results of the wind wave analysis are presented in Table 22. The complete analysis is described in the Technical Memorandum “Wind Wave Analysis for LSJRFS Alternative Comparisons”, 14 February 2014.

Wind wave runup and setup were evaluated for five wind speed scenarios over a range of 95% (1/1.1) ACE to 1.3% (1/76) ACE wind speeds. The wind analyses were based on 80 years of record at the Sacramento Executive Airport wind gage. This gage is only 40 miles north of the study area is a reasonable indicator of wind frequencies for feasibility level plan comparisons. An evaluation of closer wind stations should be considered during final design.

The distance between top of levee and mean water surface where 0.05cfs of overtopping would occur was estimated for each wind scenario. This distance is assumed to be the point at which levee failure is likely due to overtopping from the given wind scenario. The overtopping discharge was based on EC 1110-2-6067 which specifies a maximum acceptable wave overtopping discharge of 0.1 cfs/ft for well maintained unarmored earthen levee and 0.01 cfs/ft for lesser quality levees.

Analysis was performed for two representative levee reaches within the study area. Wind wave analyses were not conducted for Calaveras River, Mosher Slough, Stockton Diverting Canal, and Smith Canal because fetch lengths were less than 500 feet and not considered long enough for wind waves to be a significant performance consideration in this study. The names of the typical sites described below are based on cost estimating reach number designations described in Plates 9A through 9D.

a. San Joaquin River Main stem. This location is considered to be representative of all San Joaquin River, Stockton Deep Water Ship Channel, and French Camp Slough levee reaches considered in the alternatives. Run-up estimates assumed the levee slope was grass lined.

b. RD17 Tieback Levee. This location is representative of the Tieback levee at the upstream reach of RD17. The wind wave runup conditions assume a levee failure has occurred along the San Joaquin River and has inundated the area upstream of the RD17 tieback levee. Run-up estimates assumed the levee slope was grass lined.

Table 21: Estimated Stable Rock Revetment Sizes

Reach (Representative Wind Wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Hs Significant Wave Height (Feet)	H_{10} 10% Wave Height (Feet)	Stable Rock Revetment Size	
							Median Weight (lbs)	Median Diameter (Feet)
San Joaquin River Main Stem (SJR_160_R)	1.3%	69	1900 ft	18.0 ft	1.3 ft	1.7 ft	25 lbs	0.6 ft
	5%	47			0.9 ft	1.1 ft	8 lbs	0.4 ft
	20%	33			0.6 ft	0.8 ft	3 lbs	0.3 ft
	50%	14			0.3 ft	0.4 ft	0.3 lbs	0.1 ft
	95%	5			0.1 ft	0.1 ft	0.01 lbs	0.04 ft
RD17 Tieback (SJR_200_R)	1.3%	69	24300 ft	14.0 ft	3.9 ft	5.0 ft	680 lbs	1.7 ft
	5%	47			2.6 ft	3.3 ft	200 lbs	1.1 ft
	20%	33			1.7 ft	2.2 ft	56 lbs	0.7 ft
	50%	14			0.6 ft	0.8 ft	3 lbs	0.3 ft
	95%	5			0.2 ft	0.3 ft	0.1 lbs	0.09 ft

Notes:

* Wave Runup calculated using EurOtop method
**Stable Rock Size based on Hudson Method.

Table 22: Summary of Wind Wave Run-Up and Set Up, Alternative 1

Reach (Representative Wind Wave Reaches and Cover)	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) (Grass Lined)	1.3%	69	1900 ft	18.0 ft	2.36 ft	0.07 ft	1.0 ft
	5%	47			1.72 ft	0.03 ft	0.6 ft
	20%	33			1.28 ft	0.02 ft	0.3 ft
	50%	14			0.63 ft	0.0 ft	0.1 ft
	95%	5			0.26 ft	0.0 ft	0.0 ft
RD17 Tieback (SJR_200_R) (Grass Lined)	1.3%	69	24300 ft	14.0 ft	9.5 ft	1.1 ft	7.2 ft
	5%	47			6.4 ft	0.4 ft	4.1 ft
	20%	33			4.4 ft	0.2 ft	2.3 ft
	50%	14			1.7 ft	0.0 ft	0.5 ft
	95%	5			0.5 ft	0.0 ft	0.0 ft

Notes:

* Wave Runup calculated using EurOtop method

**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

4.6 Sedimentation and Channel Stability

Sedimentation was not studied in detail. The levee fragility related to erosion is incorporated into the fragility curves used to evaluate engineering performance of the no action plan.

4.7 Performance and Flood Risk

Performance is described by the Annual Exceedance Probability and the assurance of preventing damages from a range of flood frequencies. Flood risk is defined as the probability of a flood event occurring and the consequences of occurrence. Performance and Flood Risk were assessed using the USACE FDA model version 1.2.5a (USACE, 2010). The FDA model combines flow-frequency, stage-discharge, geotechnical fragility, and stage-damage relationships to estimate damages. Uncertainty in each relationship is incorporated by assigning uncertainty estimates and applying a Monte Carlo type approach to combine the results.

Flow-frequency, stage discharge, and geotechnical frequency relationships reflect the exterior (probability) portion of the flood risk calculations. Inundation depth and stage-damage relationships reflect the interior (consequence) portion of the flood risk calculations.

For the probability portion of the risk calculations, the hydraulic model assumptions are based on flows contained to the channel (allowed to overtop without failure). This assumption makes the breach probability statistically independent rather than dependent on another breach occurring (or not occurring). This is consistent with historical observations that indicate the probability of a breach does not appear to be highly dependent on other breaches occurring. There is no specific guidance on how to apply overtopping assumptions to system wide risk analysis and the approach is consistent with USACE risk and uncertainty guidance in EM 1110-2-1619. A sensitivity analysis to this assumption is provided in the Hydrology Section.

For the consequence portion of the risk calculations, the hydraulic model assumptions are based on levee breach failure or simply the depth for natural overbank (non-levee) conditions.

The risk assessment approach included an evaluation of potential flood sources with respect to geotechnical fragility, channel hydrology, channel hydraulics, and potential inundation patterns of a levee breach or natural overbank (non-levee). Fifteen index points were identified to reflect the reach characteristics within the study area. Within each reach a representative geotechnical fragility curve was developed. At the geotechnical curve location a stage-discharge relationship was developed using the system based hydraulic models described above. Selection of the geotechnical reaches is described in detail in the geotechnical analysis report.

a. Performance. Performance is described by Annual Exceedance Probability (AEP), assurance of passing a given Annual Chance Exceedance (ACE) hydrologic event, and Long Term Risk. AEP describes the probability of the design being exceeded over the full range of flood events and their uncertainties. The reliability of Flood Risk Management (FRM) features within the study area is expressed as an assurance level (conditional non-exceedance probability) for a given median ACE hydrologic event. The Long Term Risk describes the probability of being flooded over a given period of time (For example, 10, 30, or 50 years). The performance varies over levee reaches due to variations in geotechnical fragility, hydrology, and hydraulic characteristics and their uncertainties.

Performance was computed for the 15 index points within the study area using the HEC-FDA computer program. The index points are shown on Plate 3. Performance was calculated at the representative geotechnical fragility curve location and assumed to represent the performance at the breach location. Performance was calculated with the HEC-FDA program using an unregulated flow-frequency curve, unregulated to regulated transform, stage-discharge relationships, and geotechnical fragility curves. Uncertainty in each relationship was incorporated in the FDA model. The probability of failure due to wind wave runup and setup was not included in the performance calculations because it found to be relatively small compared to the other modes of failure and would have no influence on plan selection. The fragility curves are provided in Attachment A. FDA input assumptions are described in Table 23.

Flow-frequency curves were based on the analytical statistics computed for unregulated conditions. Uncertainty in the flow-frequency curve is based on the period of record described in the hydrology section above. The nearest upstream analytical curve statistics were utilized in combination with an unregulated-regulated transform. The unregulated flow in the transform is computed directly from the flow frequency statistics. The regulated flow used in the transform was obtained from the hydraulic model at the index location. The transforms are used to translate the uncertainty in flow frequency estimates to the regulated condition.

The geotechnical fragility curves were based on geotechnical analysis and are presented in the geotechnical appendix and provided as Attachment A to this report. The curves are assumed to have a 100% probability of failure at the levee crest. The crest elevation was modified in the FDA model to represent the Hydraulic Top of Levee (HTOL). The hydraulic top of levee at the index point is defined as the elevation corresponding to the first point of overtopping within the reach. The HTOL is lower than the actual top of levee at index points with high localized crest elevations. The probability of failure due to wind wave runup and setup was not included in the geotechnical fragility curve because it was found to be relatively small compared to the other modes of failure and would have no influence on plan selection.

Stage discharge relationships used in the analysis are described in Plates 31A through 31N. The uncertainty in the stage discharge curves was calculated using methods described in EM 1110-2-1619, Risk Analysis for Flood Damage Reduction Studies.

Table 23
FDA Input for San Joaquin River Performance Calculations
Alternative 1 - No Action

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	1/	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR4	33.9	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR3	31.0	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR2	27.8	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR1	25.0	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1	15.9 (b)	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	FL1	21.4	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	39.2	No Action	No Action	MS at Bellota	EPR = 52 yrs
	SL2	44.6	No Action	No Action	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	29.7	No Action	No Action	MS at Bellota	EPR = 52 yrs
	Cl2	31.4	No Action	No Action	MS at Bellota	EPR = 52 yrs
Delta Front	D3	13.2	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	D4	18.8	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	D5	17.5	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	D-BS	18.0	No Action	No Action	SJR nr Vernalis	EPR = 82yrs

1/ Parameters at LR4 used to estimate performance of LRTB
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Flood Depths. Maps showing composite floodplains were developed to demonstrate FRM assurance relative to a standard assurance criterion. The maps show inundation from any flood source that would not meet a risk and uncertainty based assurance criterion. The assurance criterion was based on the NFIP levee system analysis criteria described in EC 1110-2-6067 and was adopted for use in describing the performance of all ACE events. This criterion is described as “Option 2” in the DWR Urban Levee Design Criteria. The assurance criterion utilized for this study does not account for wind wave overtopping.

- For assurance less than 90% the levee does not pass criteria
- For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria.
- For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria.

The composite floodplains are provided in Plates 43 through 50. Table 24 provides performance values at simulated breach locations for 2010 conditions. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation project performance.

Table 24
Performance at Simulated Levee Breach Locations, Alternative 1
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0117	0.0110	0.2973	0.4446	0.9999	0.9984	0.9918	0.8749	0.5090	0.1908	0.0384
LR4	0.0073	0.0706	0.1971	0.3064	0.9999	0.9731	0.9525	0.9241	0.8826	0.8423	0.8095
LR3	0.0095	0.0913	0.2496	0.3803	0.9999	0.9761	0.9394	0.8998	0.7938	0.6627	0.5650
LR2	0.0211	0.1923	0.4731	0.6563	0.9999	0.9289	0.8683	0.7922	0.6831	0.5788	0.5161
LR1	0.0126	0.1188	0.3158	0.4688	0.9999	0.9610	0.9400	0.8830	0.7439	0.5772	0.4620
French Camp Slough											
FR1	0.0270	0.2393	0.5596	0.7451	0.9999	0.9490	0.9121	0.8065	0.4864	0.4394	0.0158
FL1	0.0132	0.1245	0.3290	0.4857	0.9999	0.9629	0.9460	0.9208	0.8269	0.6239	0.3857
Stockton Diverting Canal											
SL1	0.0105	0.1003	0.2717	0.4104	0.9999	0.9666	0.9633	0.9509	0.9306	0.9088	0.8900
SL2	0.0153	0.1428	0.3701	0.5372	0.9999	0.9543	0.9220	0.8951	0.8595	0.8090	0.7724
Calaveras River											
CR2	0.0094	0.0903	0.2471	0.3769	0.9999	0.9752	0.9356	0.9011	0.8563	0.8006	0.7440
CL2	.01680	0.1562	0.3991	0.5721	0.9999	0.9566	0.9410	0.9174	0.8881	0.8576	0.8292
Delta Front											
D3	0.1519	0.8074	0.9929	0.9997	0.8276	0.7477	0.7230	0.7021	0.6330	0.4968	0.3859
D4	0.0646	0.4872	0.8652	0.9645	0.9460	0.8776	0.8283	0.7876	0.7291	0.6462	0.5608
D5	0.1197	0.7206	0.9782	0.9983	0.8758	0.7806	0.7593	0.7426	0.7206	0.6890	0.6545
D-BS	0.1521	0.8079	0.9929	0.9997	0.8720	0.8005	0.7712	0.7522	0.7085	0.6381	0.5848
Cell shaded if assurance is less than criteria.											

c. Flood Velocities. Flood velocities are an indicator of life safety risk. If a levee breach were to occur, inundation velocities and depths within the study area would vary by proximity to a breach, breach location, and magnitude of flood event. The velocity field for a levee breach can be characterized as highest near the breach due to the rapidly varying flow conditions. The remaining area would have lower velocities associated with the slope of the topography and floodplain roughness. For evaluation of life loss consequence the study area can be divided into a breach zone, zone with rapidly rising water, and a remaining zone (Yonkman, 2008). Simulations of levee breaches at the peak stage of a 1% ACE event were used to evaluate characteristics of each zone.

(1) Breach zone. The breach zone is characterized by destruction of buildings and the highest life safety consequence. Yonkman describes this area as having velocities greater than 6 feet per second and the product of depth and velocity greater than 22 ft^2 per second. For the Lower San Joaquin Feasibility study, the limit of this zone is estimated to range from 250 feet to 7,600 feet from the breach location. The results indicate a breach zone of approximately 250 feet for the Calaveras River, Mormon Slough, and upper reaches of French Camp slough. The breach

zone for Lower San Joaquin River, Delta, and Lower French Camp Slough could be as much as 7600 feet. This was based on the evaluation of the maximum velocity and maximum depths in breach simulations. The characteristics of simulated breaches are shown Table 25.

(2) Zone with rapidly rising water. This zone is characterized by rapidly changing velocity and depth. Model results indicate velocities of less than 3 feet per second within a few thousand feet from the levee for most breach simulations. Within this zone, the product of depth and velocity would be greatest adjacent to the Delta Front and San Joaquin River levees and would be the highest life safety concern within this zone.

(3) Remaining zone. This zone is characterized by slower onset of flooding. The majority of the study area is defined as the remaining zone. Models of breaches indicate velocities of less than 2fps for the remaining portion of the inundation area. Higher velocities are indicated where flows overtop linear features. Additional locations with higher velocities may occur. However, they would be localized and uncertain.

Table 25
Levee Breach Simulations, 1% (1/100) ACE

Economic Impact Area	Breach ID	Grid Element	Breach Width (Feet)	Time to Develop full Breach (Minutes)	Breach Initiation Time (Hour)	Peak Breach Outflow (1% ACE) (cfs)	Maximum Grid Element Depth at Breach (1% ACE) (Feet)	Estimated Radial extent of Breach Zone (1% ACE) (Feet)
North Stockton	CR2	70712	88	19	308	1250	2.0	250
	CR1	74635	79	18	309	1060	1.8	250
Central Stockton	SL2	85232	118	22	311	3130	3.0	250
	SL1	77803	118	22	310	900	1.5	250
	CL2	72302	94	19	271	610	1.7	250
	CL1	78512	95	19	311	880	1.2	250
	FR1	114492	155	25	123	4500	7.4	250
RD17	LR1	2343	190	27	129	7800	10.3	400
	LR2	6064	180	27	133	6400	13.3	1600
	LR3	9580	210	29	135	11,700	9.7	400
	LR4	14469	190	27	133	10,200	11.5	7600
	FL1	1/	1/	1/	1/	1/	1/	1/
1/ The LR1 breach simulations were used because FL1 was found to be similar.								

d. Flood Warning Time. Flood warning time varies throughout the area and is dependent on the source and type of flood event. The principle sources of flood warnings are advisories by the National Weather Service (NWS) and river stage forecasts by the California Nevada River Forecast Center (CNRFC). The flood warning time would likely be greater for an overtopping related breach than a geotechnical failure type breach.

Flood warnings/small river and stream flood warnings are issued by the NWS when flooding of main stem rivers is occurring or imminent (CNRFC, 2013). Main stem river flooding refers to flooding of gauged and forecasted rivers (CNRFC, 2013). The product can also be used to issue Small River and Stream Flood Warnings for smaller rivers/streams which do not have forecast points.

Flash Flood Warnings are issued when flooding is reported; when precipitation capable of causing flooding is observed by radar and/or satellite; when observed rainfall exceeds flash flood guidance or criteria known to cause flooding; or when a dam or levee failure has occurred or is imminent (CNRFC, 2013). A flash flood is defined as a flood caused by heavy or excessive rainfall in a short period of time, and occurring generally within 6 hours of the causative event (CNRFC, 2013).

In addition to the advisories described above, the NWS in coordination with the California Department of Water Resources issues forecasts and guidance for river flows through the CNRFC. In general, river forecasts are based on modeled runoff from observed precipitation, snowmelt estimates, and reservoir operations. The forecast length varies depending on the location. River guidance is based on modeled runoff from forecasted precipitation, snowmelt estimates, and reservoir operations. The forecasts and guidance are issued for a forecast site in a graphical format that compares the future river stage to a monitor stage, flood stage, and danger stage. The combined forecast and guidance are made 5 days into the future.

Flooding from interior drainage sources within the study area is likely to be the result of localized concentrated rainfall. It is assumed these floods would be preceded by a general flood watch issued by the NWS 12 to 24 hours in advance and a flash flood warning 6 hours in advance of the localized flooding.

Flooding from a levee overtopping event along the San Joaquin River would result from a large regional storm event in the San Joaquin River Watershed. CNRFC river flood forecast points on the San Joaquin River are located at Vernalis and Mossdale. It is assumed that an overtopping flood would be preceded by a flood warning and river guidance issued by the NWS and CNRFC five days in advance. A more accurate warning of potential levee overtopping, based on river forecasts, would likely be made 48 hours in advance. This estimate was based on a review of the flood guidance plots for December 2005-January2006 flood which indicate the forecasted peak flow was similar to the observed flow approximately 48 hours prior.

Flooding from a levee overtopping event along the Calaveras River, Stockton Diverting Canal, or Mormon Slough, would result from a large regional storm event in the Calaveras River watershed. There are no CNRFC forecast points in the Calaveras River watershed. It is assumed these floods would be preceded by a flood warning by the NWS and CNRFC five days in advance. Forecasted releases from New Hogan Dam would likely be posted to the California Data Exchange Center and the Sacramento Districts Website. However, there is no standard operating procedure or requirements to make these forecasts available to the public.

It is estimated that flooding from a geotechnical levee breach would have little to no advance warning (less than 1 hour) and the floodwave would rapidly inundate the adjacent areas.

4.8 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system. There is no induced flooding for the no-action plan. However, a description of flood depth, duration, and frequency, are provided below for comparison with the other plans.

a. Flood Depth.

Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Four index points were selected outside the study area to demonstrate the potential change in flood depths outside the study area. Middle River at Borden Highway index point is located at a recording stage gage and was selected to represent potential changes to the stage of middle River downstream of the study area. Old River at Clifton Court Ferry index point is located at a recording stage gage and was selected to represent potential changes to the stage of Old River downstream of the study area. Paradise Cut at Paradise Road index point was selected to represent potential changes to stage in Paradise Cut adjacent to the planned River Islands development. The Stockton Deep Water Ship Channel (SDWSC) at Burns Cutoff index point is located at a recording stage gage and was selected to represent potential changes to the stage of San Joaquin River downstream of the study area.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration.

The duration of a high flood stages depends on storm duration, antecedent watershed conditions, and antecedent reservoir storage. The duration of high stages along the delta front and San Joaquin River would likely be one week. The duration of high stages along the Calaveras River would likely be several days. The duration of high stages from interior runoff would likely be less than 1 day.

c. Frequency.

The change in flood frequency is described by changes in Annual Exceedance Probability (AEP) and Assurance. The change in stage and flow frequency at index points is provided in Plates 31 and 32. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

The performance values associated with hydrologic and hydraulic parameters are provided in Table 26. For purposes of evaluating induced flooding the risk analysis is limited to hydrologic and hydraulic parameters and their uncertainties. This approach is consistent with Section 3.b

(2) of the memorandum "Clarification Guidance on the Policy and Procedural Guidance for the Approval of Modifications and Alterations of Corps of Engineers Projects" (USACE, 2008).

Table 26
2010 Performance at Selected Locations, Alternative 1
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency					
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0113	0.1075	0.2892	0.4338	0.9999	0.9999	0.9957	0.8808	0.5134	0.1915	0.0374
LR4	0.0001	0.0007	0.0022	0.0037	0.9999	0.9999	0.9999	0.9998	0.9975	0.9858	0.9693
LR3	0.0000	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9992	0.9982
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9998	0.9986	0.9972
LR1	0.0005	0.0050	0.0148	0.0245	0.9999	0.9999	0.9999	0.9990	0.9838	0.9251	0.8565
French Camp Slough											
FR1	0.0109	0.1036	0.2796	0.4211	0.9999	0.9997	0.9929	0.9027	0.5550	0.1876	0.0183
FL1	0.0031	0.0306	0.0889	0.1437	0.9999	0.9999	0.9999	0.9964	0.9407	0.7268	0.4865
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0028	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9829
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0029	0.0288	0.0839	0.1358	0.9999	0.9982	0.9931	0.9814	0.9172	0.7624	0.6203
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0010	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9994	0.9994	0.9994	0.9994
Paradise Cut at I-5 F-PCI5	0.0014	0.0140	0.0415	0.0682	0.9999	0.9995	0.9995	0.9995	0.9995	0.9952	0.9779
Paradise Cut at Paradise Rd. F-PCPR	0.0017	0.0167	0.0492	0.0807	0.9999	0.9995	0.9995	0.9995	0.9995	0.9867	0.8641
SDWSC blw Burns Cutoff F-B95660	0.0002	0.0016	0.0049	0.0081	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995
Assurance estimates account for stage uncertainty, hydrologic uncertainty only. SDWSC- Stockton Deep Water Ship Channel											

4.9 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at

downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 27. Composite floodplain maps were not developed for 2070 conditions.

Table 27
Performance at Simulated Levee Breach Locations, Alternative 1, 2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0118	0.1122	0.3002	0.4483	0.9999	0.9984	0.9912	0.8707	0.5026	0.4440	0.5153
LR4	0.0075	0.0726	0.2023	0.3139	0.9999	0.9725	0.9509	0.9228	0.8819	0.8454	0.8093
LR3	0.0101	0.0968	0.2632	0.3990	0.9999	0.9715	0.9362	0.8962	0.7875	0.6712	0.5652
LR2	0.0257	0.2295	0.5426	0.7285	0.9999	0.9153	0.8415	0.7718	0.6711	0.5826	0.5153
LR1	0.0141	0.1326	0.3475	0.5091	0.9999	0.9567	0.9334	0.8764	0.7412	0.5910	0.4616
French Camp Slough											
FR1	0.0415	0.3458	0.7200	0.8801	0.9098	0.9098	0.8425	0.7033	0.3926	0.4394	0.0111
FL1	0.0202	0.1849	0.4586	0.6403	0.9999	0.9443	0.9244	0.9005	0.8055	0.5999	0.3647
Stockton Diverting Canal											
SL1	0.0105	0.1003	0.2717	0.4104	0.9999	0.9666	0.9633	0.9509	0.9306	0.9088	0.8900
SL2	0.0153	0.1428	0.3701	0.5372	0.9999	0.9543	0.9220	0.8951	0.8595	0.8090	0.7724
Calaveras River											
CR2	0.0094	0.0903	0.2471	0.3769	0.9999	0.9752	0.9356	0.9011	0.8563	0.8006	0.7440
CL2	0.0168	0.1562	0.3991	0.5721	0.9999	0.9098	0.8425	0.7033	0.3926	0.1268	0.0111
Delta Front											
D3	0.2091	0.9043	0.9991	0.9999	0.7935	0.6418	0.5907	0.5516	0.4483	0.2832	0.1665
D4	0.0962	0.6361	0.9518	0.9936	0.9199	0.8140	0.7601	0.7164	0.6577	0.5820	0.5067
D5	0.1582	0.8214	0.9943	0.9998	0.8232	0.7473	0.7262	0.7097	0.6851	0.6431	0.5926
D-BS	0.1890	0.8769	0.9981	0.9999	0.8490	0.7013	0.6723	0.6544	0.6076	0.4655	0.4655

4.10 California State Urban Levee Design Criteria

Although the California State Urban Levee Design Criteria (ULDC) is not a federal objective of the study, it is a local sponsor objective. Two options are offered in the ULDC requirements for determining if a levee meets the urban and urbanizing area levee system design. The freeboard option (option 1) requires 3 feet of freeboard above the mean 0.5% (1/200) ACE flood event. The risk and uncertainty option (option 2) allows for a lesser amount of freeboard (2 feet) if a high level of assurance (95%) can be demonstrated. The hydraulic performance of the no-action alternative relative to the ULDC requirements for 2070 conditions is provided in Table 28. The ULDC also requires minimum geotechnical design requirements. However, these are not accounted for in the assessment conducted for in the hydraulic analysis.

Table 28
Alternative 1 Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind Wave Run up and Setup (FT)	Minimum ULDC Required Freeboard	Mean 0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	33.9	10.6	10.6	30.0	3.9	99%
	LR4	RD17	33.9	2.4	3.0	30.0	3.9	99%
	LR3	RD17	31.0	2.4	3.0	25.6	5.4	99%
	LR2	RD17	27.8	2.4	3.0	23.0	4.8	99%
	LR1	RD17	25.0	2.4	3.0	22.6	2.4	92%
French Camp Slough	FR1	CS-02	21.8	<3.0	3.0	20.4	1.4	15%
	FL1	RD17	21.4	<3.0	3.0	20.4	1.0	70%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	30.3	8.1	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.8	4.8	99%
Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	26.5	3.2	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.5	4.9	99%
Delta Front	D3	NS-02	13.2	<3.0	3.0	13.6	0.4	45%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
LRTB assurance based on LR4 index point

5.0 ALTERNATIVE 7A

Alternative 7A provides flood risk reduction benefits to portions of North and Central Stockton economic impact areas. The alternative includes new delta front levee segments, Fix-in-Place levee segments along the Delta front and San Joaquin River, a closure structure at Fourteenmile Slough, and a closure structure at Smith Canal. A summary of the design features associated with Alternative 7A are described below and shown on Plate 51.

5.1 Hydraulic Design Summary

a. General Design. All project features would be designed to meet current USACE design requirements. This alternative would combine the fix-in-place measures of cutoff wall, seismic deep soil mixing, seepage berm, and levee geometry improvements. Descriptions of these improvements are provided in the feasibility study report.

The performance analysis described below assumes the geotechnical performance of the project features would have negligible probability of failure below the design top of levee. It was assumed all levee features would fail completely if overtopped.

b. Levee Design Height. This project would include levee improvements as shown on Plate 51. The levee height would be based on the authorized design profile, the existing profile, or increased height to achieve the DWR ULDC requirements for 2070 sea level conditions, whichever is higher. The models used to define the improvements assumed the levees in RD17 also met ULDC requirements. However improvements to the RD17 levees are not included in Alternative 8A and were not included in models used to assess the project performance. The height required to meet ULDC requirements was computed using the HEC-RAS models modified from the no action condition.

c. New Levees. Alternative 7A would extend the levee along the right bank of French Camp Slough upstream to the UPRR rail yard. The design height of new levees is described above.

d. Upstream Reservoir Operation. Alternative 7A does not include any modifications to upstream reservoirs. The hydraulic analysis assumes all upstream reservoirs are operated the same as no-action conditions.

e. Interior Drainage Facilities. Alternative 7A does not include any modifications to interior drainage facilities.

f. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions. Additional operation and maintenance would be required at the Smith Canal and Fourteenmile Slough Closure Structures. It is estimated that vegetation maintenance within 20 feet of the levee toe would have little to no impact on the hydraulic estimates.

g. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (*Design of I-Walls*, 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority. The RD17 and French Camp slough tie back levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile upstream of the project rather than being overtopped within the study area. Flow would outflank the levee only during the peak of the event and would reduce the flow and stage along the levee reaches. The outflanking would occur slowly and allow more evacuation time.

As described above, this alternative would extend the levee along the right bank of French Camp Slough further upstream to meet the ULDC requirements. However, the natural ground upstream of the levee would remain lower than the proposed levee extension to maintain levee superiority.

h. Erosion Protection. Rock revetment erosion protection would be placed along the proposed delta front levees with long fetches. The results of wind wave analysis conducted for Alternative 7A are presented below.

i. Diversion structures. Alternative 7A does not include any additional diversion structures beyond the no action alternative.

j. Closure Structures.

(1) Smith Canal Closure Structure. A gate type closure structure would be constructed on Smith Canal to provide flood risk reduction from high stages in the Sacramento and San Joaquin Delta. The proposed closure structures would consist of a fixed sheet pile wall structure with an opening gate structure to allow for navigation. The opening portion of the closure structure would be a 50' wide miter gate structure. As needed, a sheet pile floodwall would be constructed adjacent to the control structures to tie the structures into the adjacent levee or high ground areas

The structure would be closed during peak flood events when the stage reached approximately 8.0 feet NAVD88 or in the event of a levee breach along Smith Canal. The closure structures would prevent the extremely large volume of floodwaters in the Delta from flowing to the breach opening. As a result, the volume of floodwaters from a breach would be restricted to only the volume held in the canal.

(2) Fourteenmile Closure Structure. A gate type closure structure would also be constructed on Fourteenmile Canal to provide flood risk reduction from high stages in the Sacramento and San Joaquin Delta. The structure design is similar to the Smith Canal closure structure.

5.2 Hydrology.

The hydrology associated with Alternative 7A is identical to Alternative 1 (no-action conditions).

5.3 Hydraulic Models and Results

Hydraulic models associated with Alternative 7A were modified to reflect increased levee height required to meet the DWR ULDC requirements for 2070 sea level conditions. Height increases were limited to only the levees providing FRM to the study area and assume the upstream levees in RD17 were also improved to meet the ULDC requirements. However improvements to the RD17 levees are not included in Alternative 7A and were not included in models used to assess the project performance. Stage and Flow frequency curves are provided in Plates 31A through 31N and Plates 32A through 32E.

5.4 Wind Wave Analysis

Additional Wind Wave analysis was performed for the proposed delta front levee segments. The analysis was performed following the methods described in the no action plan. An assessment of stable rock diameter was also conducted to evaluate the potential for wind wave erosion. The results of the wind wave analysis are presented in Tables 29 and 30.

a. Delta Front – Shima Tract. This location is representative of Shima Tract reaches ST_10_R through ST_30_R, Fourteenmile slough reach FM_60_L, and Five mile Slough reach FS_10R. The wind wave runup estimates assume a levee failure has occurred outside the proposed project reaches and Shima Tract has completely flooded. Based on the results of the wind wave erosion analysis provided in Table 29, 1-foot median diameter rock revetment was specified along these levee segments.

b. Delta Front – Fourteenmile Slough. This location is representative of Fourteenmile Slough reaches FM_30_L and FM_40_L and Ten Mile Slough reach TS_30L. The wind wave runup conditions assume a levee failure has occurred outside the proposed project reaches and Wright-Elmwood Tract has completely flooded. Based on the results of the wind wave erosion analysis presented in Table 29, 1-foot median diameter rock revetment was specified along these levee segments.

Table 29: Stable Rock Revetment Sizes, Proposed Delta Front Levees

Representative Wind Wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Hs Significant Wave Height (Feet)	H_{10} 10% Wave Height (Feet)	Stable Rock Revetment Size	
							Median Weight (lbs)	Median Diameter (Feet)
Delta Front-Fourteenmile Slough FM_30_L	1.3%	54	9300 ft	17.0 ft	2.2 ft	2.8 ft	121.7 lbs	1.0 ft
	5%	36			1.7 ft	2.2 ft	56.1 lbs	0.7 ft
	20%	25			1.0 ft	1.3 ft	11.4 lbs	0.4 ft
	50%	10			0.4 ft	0.5 ft	0.7 lbs	0.2 ft
	95%	5			0.2 ft	0.3 ft	0.1 lbs	0.09 ft
Delta Front-Shima Tract ST_20_R	1.3%	54	10100 ft	14.0 ft	2.3 ft	2.9 ft	139 lbs	1.0 ft
	5%	36			1.5 ft	1.9 ft	38.6 lbs	0.7 ft
	20%	25			1.1 ft	1.4 ft	15.2 lbs	0.5 ft
	50%	10			0.4 ft	0.5 ft	0.7 lbs	0.2 ft
	95%	5			0.2 ft	0.3 ft	0.1 lbs	0.09 ft

Notes:

* Wave Runup calculated using EurOtop method
**Stable Rock Size based on Hudson Method.

Table 30: Wind Wave Run-Up and Set Up Results, Alternative 7A

Representative Wind Wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) Grass Lined	1.3%	69	1900 ft	18.0 ft	2.36 ft	0.07 ft	1.0 ft
	5%	47			1.72 ft	0.03 ft	0.6 ft
	20%	33			1.28 ft	0.02 ft	0.3 ft
	50%	14			0.63 ft	0.0 ft	0.1 ft
	95%	5			0.26 ft	0.0 ft	0.0 ft
Delta Front-Fourteen Mile Slough (FM_30_L) Rock Lined	1.3%	54	9300 ft	17.0 ft	2.7 ft	0.2 ft	1.6 ft
	5%	36			1.9 ft	0.1ft	1.0 ft
	20%	25			1.4 ft	0.0 ft	0.6 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
Delta Front-Shima Tract (ST_20_R) Rock Lined	1.3%	54	10100 ft	14.0 ft	2.8 ft	0.3 ft	1.8 ft
	5%	36			2.0 ft	0.1 ft	1.0 ft
	20%	25			1.5 ft	0.1 ft	0.7 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
RD17 Tieback SJR_200_R (Grass Lined)	1.3%	69	24300 ft	14.0 ft	9.5 ft	1.1 ft	7.2 ft
	5%	47			6.4 ft	0.4 ft	4.1 ft
	20%	33			4.4 ft	0.2 ft	2.3 ft
	50%	14			1.7 ft	0.0 ft	0.5 ft
	95%	5			0.5 ft	0.0 ft	0.0 ft

Notes:

* Wave Runup calculated using EurOtop method
**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

5.5 Sedimentation and Channel Stability

Sedimentation and channel stability associated with Alternative 7A is identical to Alternative 1 (no action conditions).

5.6 Performance and Flood Risk

Flood risk to portions of North and Central Stockton would be reduced by Alternative 7A. The performance and residual flood risk associated with this alternative was modeled by adjusting the FDA inputs for breach simulations within the study area.

a. Performance. Performance is described by Annual Exceedance Probability (AEP) and assurance of passing a given hydrologic event. Performance estimates were recomputed assuming no failure until overtopping for reaches improved in the alternative. This was modeled by changing the with-project fragility curves so they had no probability of failure until overtopped. The levee height for the FR1 breach location was modified to account for the extension of the French Camp Slough levee further upstream. The levee height at the D3 breach location was modified to account for levee height increases to meet the ULDC requirement (assuming RD17 levees were also improved to ULDC requirements). These increases were determined to be economically feasible based on incremental net benefit analysis conducted for the initial and focused array of alternatives. All other inputs to calculate performance were identical to Alternative 1, the no action condition. The FDA input assumptions are described in Table 31. The performance of the project at index points throughout the study area is provided in Table 32.

Table 31
FDA Input for San Joaquin River Performance Calculations
Alternative 7A

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR4	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR3	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR2	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR1	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1	***Raise to 18.5 (b)***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	FL1	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
	SL2	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
	Cl2	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
Delta Front	D3	***Raise to 14.9***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D4	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D5	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D-BS	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs

Changes from no action plan shown in bold italics.
(a) Parameters at LR4 used to estimate performance of LRTB
(b) Hydraulic top of levee represented by natural bank upstream of levee.
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Floodplains. Maps showing composite floodplains were developed to demonstrate FRM reliability for Alternative 7A. The composite floodplains are provided in Plates 52 to 59. Table 32 provides the assurance values used to determine if a simulated breach was included in the composite floodplain map. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation project performance.

c. Flood Velocities. Flood velocities for a levee beach would be identical to Alternative 1.

Table 32
Performance at Simulated Levee Breach Locations, Alternative 7A
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0117	0.0110	0.2973	0.4446	0.9999	0.9984	0.9918	0.8749	0.5090	0.1908	0.0384
LR4	0.0073	0.0706	0.1971	0.3064	0.9999	0.9731	0.9525	0.9241	0.8826	0.8423	0.8095
LR3	0.0095	0.0913	0.2496	0.3803	0.9999	0.9761	0.9394	0.8998	0.7938	0.6627	0.5650
LR2	0.0211	0.1923	0.4731	0.6563	0.9999	0.9289	0.8683	0.7922	0.6831	0.5788	0.5161
LR1	0.0126	0.1188	0.3158	0.4688	0.9999	0.9610	0.9400	0.8830	0.7439	0.5772	0.4620
French Camp Slough											
FR1	0.0073	0.0705	0.1969	0.3062	0.9999	0.9999	0.9999	0.9766	0.7718	0.3554	0.0785
FL1	0.0132	0.1245	0.3290	0.4857	0.9999	0.9629	0.9460	0.9208	0.8269	0.6032	0.3857
Stockton Diverting Canal											
SL1	0.0105	0.1003	0.2717	0.4104	0.9999	0.9666	0.9633	0.9509	0.9306	0.9088	0.8900
SL2	0.0153	0.1428	0.3701	0.5372	0.9999	0.9543	0.9220	0.8951	0.8595	0.8148	0.7724
Calaveras River											
CR2	0.0094	0.0903	0.2471	0.3769	0.9999	0.9752	0.9356	0.9011	0.8563	0.8006	0.7440
CL2	.01680	0.1562	0.3991	0.5721	0.9999	0.9566	0.9410	0.9174	0.8881	0.8576	0.8292
Delta Front											
D3	0.0003	0.0025	0.0076	0.0126	0.9999	0.9999	0.9998	0.9989	0.9896	0.9584	0.9226
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Cell shaded if assurance is less than criteria.											

d. Flood Warning Time. Alternative 7A will result in a significant increase in warning time to the population within North and Central Stockton because the probability of flooding from a geotechnical type failure (1-hour warning time) would be reduced and the warning time for overtopping type failures are significantly longer (24 to 36 hours). A description of flood warning time is provided in Alternative 1.

5.7 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system.

a. Flood Depth.

Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Alternative 7A includes fix in place levees, levee raises along the Delta Front, and an extension of French Camp slough levees upstream. Flood depths in the channel at all index points would be the same as the no action condition. Flood depths in Smith Canal and Fourteenmile slough are not described by index points and would be reduced to 8 feet NAVD88 by the proposed closure structures. It is unlikely that improvements along the delta front levees would increase water levels from delta sources. It is possible that the increased delta front levee height could result in increased flood depths in the floodplain if a levee failure occurred along the Calaveras River or Stockton Diverting Canal. However, the area would already be flooded by the upstream levee breach.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42 and are the same as the no-action condition. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration.

It is unlikely that improvements would change the duration of flooding throughout the system.

c. Frequency. The Delta Front raises and extension of French Camp slough levees upstream are unlikely to have hydraulic impacts that would impact flood frequency. The computed AEP and assurance values based on only the hydrology and hydraulic inputs are presented in Table 33. Changes to AEP and assurance values are presented in Table 34. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

Table 33
2010 Performance at Selected Locations, Alternative 7A
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0113	0.1075	0.2892	0.4338	0.9999	0.9999	0.9957	0.8808	0.5134	0.1915	0.0374
LR4	0.0001	0.0007	0.0022	0.0037	0.9999	0.9999	0.9999	0.9998	0.9975	0.9858	0.9693
LR3	0.0000	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9992	0.9982
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9998	0.9986	0.9972
LR1	0.0005	0.0050	0.0148	0.0245	0.9999	0.9999	0.9999	0.9990	0.9838	0.9251	0.8565
French Camp Slough											
FR1	0.0073	0.0705	0.1969	0.3062	0.9999	0.9999	0.9999	0.9766	0.7718	0.3554	0.0785
FL1	0.0031	0.0306	0.0889	0.1437	0.9999	0.9999	0.9999	0.9964	0.9407	0.7268	0.4865
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0028	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9829
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0003	0.0025	0.0076	0.0126	0.9999	0.9999	0.9998	0.9989	0.9896	0.9584	0.9226
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0010	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9994	0.9994	0.9994	0.9994
Paradise Cut at I-5 F-PCI5	0.0014	0.0140	0.0415	0.0682	0.9999	0.9995	0.9995	0.9995	0.9995	0.9952	0.9779
Paradise Cut at Paradise Rd. F-PCPR	0.0017	0.0167	0.0492	0.0807	0.9999	0.9995	0.9995	0.9995	0.9995	0.9867	0.8641
SDWSC blw Burns Cutoff F-B95660	0.0002	0.0016	0.0049	0.0081	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995
Assurance estimates account for stage uncertainty, hydrologic uncertainty only. SDWSC- Stockton Deep Water Ship Channel											

Table 34
2010 Change in Performance at Selected Locations, Alternative 7A Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Change in Annual Exceedance Probability (Expected)	Change in Long Term Risk			Change in Flood Risk Management Assurance by Event Flood Frequency					
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0	0	0	0	0	0	0	0	0	0	0
LR4	0	0	0	0	0	0	0	0	0	0	0
LR3	0	0	0	0	0	0	0	0	0	0	0
LR2	0	0	0	0	0	0	0	0	0	0	0
LR1	0	0	0	0	0	0	0	0	0	0	0
French Camp Slough											
FR1	-0.0036	-0.0331	-0.0827	-0.1149	0	0.0002	0.007	0.0739	0.2168	0.1678	0.0602
FL1	0	0	0	0	0	0	0	0	0	0	0
Stockton Diverting Canal											
SL1	0	0	0	0	0	0	0	0	0	0	0
SL2	0	0	0	0	0	0	0	0	0	0	0
Calaveras River											
CR2	0	0	0	0	0	0	0	0	0	0	0
CL2	0	0	0	0	0	0	0	0	0	0	0
Delta Front											
D3	-0.0026	-0.0263	-0.0763	-0.1232	0	0.0017	0.0067	0.0175	0.0724	0.196	0.3023
D4	0	0	0	0	0	0	0	0	0	0	0
D5	0	0	0	0	0	0	0	0	0	0	0
D-BS	0	0	0	0	0	0	0	0	0	0	0
Outside Study Area											
Middle R. at Borden Hwy	0	0	0	0	0	0	0	0	0	0	0
Old R. at Clifton Court Ferry	0	0	0	0	0	0	0	0	0	0	0
Paradise Cut at I-5	0	0	0	0	0	0	0	0	0	0	0
Paradise Cut at Paradise Rd.	0	0	0	0	0	0	0	0	0	0	0
SDWSC blw Burns Cutoff	0	0	0	0	0	0	0	0	0	0	0

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.

SDWSC- Stockton Deep Water Ship Channel

5.8 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 35. Composite floodplain maps were not developed for 2070 conditions.

Table 35
Performance at Simulated Levee Breach Locations, Alternative7A
2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0118	0.1122	0.3002	0.4483	0.9999	0.9984	0.9912	0.8707	0.5026	0.4440	0.5153
LR4	0.0075	0.0726	0.2023	0.3139	0.9999	0.9725	0.9509	0.9228	0.8819	0.8417	0.8093
LR3	0.0101	0.0968	0.2632	0.3990	0.9999	0.9715	0.9362	0.8962	0.7875	0.6593	0.5652
LR2	0.0257	0.2295	0.5426	0.7285	0.9999	0.9153	0.8415	0.7718	0.6711	0.5736	0.5153
LR1	0.0141	0.1326	0.3475	0.5091	0.9999	0.9567	0.9334	0.8764	0.7412	0.5757	0.4616
French Camp Slough											
FR1	0.0078	0.0753	0.2093	0.3238	0.9999	0.9999	0.9994	0.9679	0.7401	0.3260	0.0673
FL1	0.0202	0.1849	0.4586	0.6403	0.9999	0.9443	0.9244	0.9005	0.8055	0.5790	0.3647
Stockton Diverting Canal											
SL1	0.0105	0.1003	0.2717	0.4104	0.9999	0.9666	0.9633	0.9509	0.9306	0.9088	0.8900
SL2	0.0153	0.1428	0.3701	0.5372	0.9999	0.9543	0.9220	0.8951	0.8595	0.8148	0.7724
Calaveras River											
CR2	0.0094	0.0903	0.2471	0.3769	0.9999	0.9752	0.9356	0.9011	0.8563	0.8006	0.7440
CL2	0.0168	0.1562	0.3991	0.5721	0.9999	0.9566	0.9410	0.9174	0.8881	0.8576	0.8292
Delta Front											
D3	0.0021	0.0207	0.0608	0.9992	0.9999	0.9968	0.9919	0.9830	0.9331	0.8107	0.6974
D4	0.0001	0.0013	0.0040	0.0067	0.9999	0.9999	0.9999	0.9992	0.9952	0.9826	0.9642
D5	0.0005	0.0047	0.0139	0.0231	0.9999	0.9998	0.9992	0.9965	0.9831	0.9402	0.8794
D-BS	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9993	0.9969	0.9938

5.9 California State Urban Levee Design Criteria

The hydraulic performance of Alternative 7A relative to the ULDC requirements for 2070 conditions is provided in Table 36.

Table 36
Alternative 7A Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind Wave Run up and Setup (FT)	Minimum ULDC Required Freeboard	Mean 0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	33.9	10.6	10.6	30.0	3.9	99%
	LR4	RD17	33.9	2.4	3.0	30.0	3.9	99%
	LR3	RD17	31.0	2.4	3.0	25.6	5.4	99%
	LR2	RD17	27.8	2.4	3.0	23.0	4.8	99%
	LR1	RD17	25.0	2.4	3.0	22.6	2.4	93%
French Camp Slough	FR1	CS-02	21.8	<3.0	3.0	20.4	1.4	15%
	FL1	RD17	21.4	<3.0	3.0	20.4	1.0	70%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	30.3	8.1	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.8	4.8	99%
Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	26.5	3.2	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.5	4.9	99%
Delta Front	D3	NS-02	14.9	<3.0	3.0	13.6	1.3	81%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
LRTB assurance based on LR4 index point

6.0 ALTERNATIVE 7B

Alternative 7B is similar to 7A but includes additional levee fixes in RD17 and improvements to the RD17 tieback levee. A summary of the design features associated with Alternative 7B are described below and shown on Plate 60.

6.1 Hydraulic Design Summary

a. General Design. All project features would be designed to meet current USACE design requirements. This alternative would combine the fix-in-place measures of cutoff wall, seismic deep soil mixing, seepage berm, and levee geometry improvements. Descriptions of these improvements are provided in the feasibility study report.

The performance analysis described below assumes the geotechnical performance of the project features would have negligible probability of failure below the design top of levee. It was assumed all levee features would fail completely if overtopped.

b. Levee Design Height. This project would include levee improvements as shown on Plate 60. The levee height would be based on the authorized design profile, the existing profile, or increased height to achieve the DWR ULDC requirements for 2070 sea level conditions, whichever is higher. The height required to meet ULDC requirements was computed using the HEC-RAS models modified from the no action condition.

c. New Levees. Alternative 7B would extend and raise the RD17 tieback levee at Walthall Slough. The levee would be extended to where the natural ground elevation was equivalent to the 0.5% (1/200) ACE median water surface. The design height of new levees is described above. The extension of Duck Creek levees described in Alternative 7A would not be included in this alternative.

d. Upstream Reservoir Operation. Alternative 7B does not include any modifications to upstream reservoirs. The hydraulic analysis assumes all upstream reservoirs are operated the same way as the no-action alternative.

e. Interior Drainage Facilities. Alternative 7B does not include any modifications to interior drainage facilities.

f. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions. Additional operation and maintenance would be required at the Smith Canal and Fourteenmile Slough Closure Structures. It is estimated that vegetation maintenance within 20 feet of the levee toe would have little to no impact on the hydraulic estimates.

g. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (*Design of I-Walls,* 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority. The RD17 and French Camp slough tie back levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile upstream of the project rather than being overtopped within the study area. Flow would outflank the levee only during the peak of the event and would reduce the flow and stage along the levee reaches. The outflanking would occur slowly and allow more evacuation time.

h. Erosion Protection.

Erosion protection would be similar to Alternative 7A. However, additional rock revetment erosion protection would be placed along the RD17 tieback levee to address wind wave erosion. The results of wind wave analysis conducted for Alternative 7B are presented below.

i. Diversion structures. Alternative 7B does not include any additional diversion structures beyond the no action alternative.

j. Closure Structures.

(1) Smith Canal Closure Structure. The Smith Canal Closure Structure is identical to Alternative 7A.

(2) Fourteenmile Closure Structure. The Fourteenmile Closure Structure is identical to Alternative 7A.

6.2 Hydrology.

The hydrology associated with Alternative 7B is identical to Alternative 1 (no-action conditions).

6.3 Hydraulic Models and Results

Hydraulic models associated with Alternative 7B were modified to reflect increased levee height required to meet the DWR ULDC requirements for 2070 sea level conditions. Height increases were limited to only the levees providing FRM to the study area. Levees in RD17 were also improved to meet the ULDC requirements. Stage and Flow frequency curves are provided in Plates 31A through 31N and 32A through 32E.

6.4 Wind Wave Analysis

Additional Wind Wave analysis was performed for the RD17 tieback levee assuming a rock lined slope. The analysis was performed following the methods described in the no action plan. The wind wave estimates for Alternative 7B are provided in Table 37.

Table 37: Wind Wave Run-Up and Set Up Results, Alternative 7B

Representative Wind Wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) Grass Lined	1.3%	69	1900 ft	18.0 ft	2.36 ft	0.07 ft	1.0 ft
	5%	47			1.72 ft	0.03 ft	0.6 ft
	20%	33			1.28 ft	0.02 ft	0.3 ft
	50%	14			0.63 ft	0.0 ft	0.1 ft
	95%	5			0.26 ft	0.0 ft	0.0 ft
Delta Front-Fourteen Mile Slough (FM_30_L) Rock Lined	1.3%	54	9300 ft	17.0 ft	2.7 ft	0.2 ft	1.6 ft
	5%	36			1.9 ft	0.1ft	1.0 ft
	20%	25			1.4 ft	0.0 ft	0.6 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
Delta Front-Shima Tract (ST_20_R) Rock Lined	1.3%	54	10100 ft	14.0 ft	2.8 ft	0.3 ft	1.8 ft
	5%	36			2.0 ft	0.1 ft	1.0 ft
	20%	25			1.5 ft	0.1 ft	0.7 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
RD17 Tieback SJR_200_R (Rock Lined)	1.3%	69	24300 ft	14.0 ft	5.2 ft	1.1 ft	4.5 ft
	5%	47			3.5 ft	0.4 ft	2.4 ft
	20%	33			2.4 ft	0.2 ft	1.4 ft
	50%	14			0.9 ft	0.0 ft	0.3 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft

Notes:

* Wave Runup calculated using EurOtop method
**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

6.5 Sedimentation and Channel Stability

Sedimentation and channel stability associated with Alternative 7B is identical to Alternative 1 (no action conditions).

6.6 Performance and Flood Risk

Flood risk to portions of RD17, North Stockton, and Central Stockton would be reduced by Alternative 7B. The performance and residual flood risk associated with this alternative was modeled by adjusting the FDA inputs for breach simulations within the study area.

a. Performance. Performance is described by Annual Exceedance Probability (AEP) and assurance of passing a given hydrologic event. Performance estimates were recomputed assuming no failure until overtopping for reaches improved in the alternative. This was modeled by changing the the with-project fragility curves so they had no probability of failure until overtopped. The levee height at the D3 breach location was modified to account for levee height increases to meet the ULDC requirement (assuming RD17 levees were also improved to ULDC requirements). The levee height of the LRTB index point was modified to account for the extension of the tieback levee. These increases were determined to be economically feasible based on incremental net benefit analysis conducted for the initial and focused array of alternatives. All other inputs to calculate assurance were identical to Alternative 1, the no action condition.

The FDA input assumptions are described in Table 38. The performance of the project at index points throughout the study area is provided in Table 39.

Table 38
FDA Input for San Joaquin River Performance Calculations
Alternative 7B

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	***Raise to 34.9***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	LR4	***Raise to 34.9***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	LR3	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	LR2	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	LR1	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1	***Raise to 18.5 (b)***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	FL1	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
	SL2	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
	Cl2	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
Delta Front	D3	***Raise to 14.9***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D4	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D5	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D-BS	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs

Changes from no action plan shown in bold italics.
(a) Parameters at LR4 used to estimate performance of LRTB
(b) Hydraulic top of levee represented by natural bank upstream of levee.
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Floodplains. Maps showing composite floodplains were developed to demonstrate FRM reliability for Alternative 7B. The composite floodplains are provided in Plates 61 to 68. Table 38 provides the assurance values used to determine if a simulated breach was included in the composite floodplain map. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation project performance.

c. Flood Velocities. Flood velocities for a levee beach would be identical to Alternative 1.

Table 39
Assurance at Simulated Levee Breach Locations, Alternative 7B
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9382	0.8544
LR4	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9382	0.8544
LR3	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9999	0.9982	0.9906	0.9781
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9997	0.9989	0.9978
LR1	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9999	0.9987	0.9954	0.9917
French Camp Slough											
FR1	0.0070	0.0679	0.1901	0.2963	0.9999	0.9997	0.9935	0.9328	0.7353	0.4974	0.3465
FL1	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
Stockton Diverting Canal											
SL1	0.0105	0.1003	0.2717	0.4104	0.9999	0.9666	0.9633	0.9509	0.9306	0.9088	0.8900
SL2	0.0166	0.1540	0.3945	0.5666	0.9999	0.9496	0.9177	0.8895	0.8542	0.8090	0.7616
Calaveras River											
CR2	0.0094	0.0903	0.2471	0.3769	0.9999	0.9752	0.9356	0.9011	0.8563	0.8006	0.7440
CL2	.01680	0.1562	0.3991	0.5721	0.9999	0.9566	0.9410	0.9174	0.8881	0.8576	0.8292
Delta Front											
D3	0.0000	0.0003	0.0009	0.0014	0.9999	0.9999	0.9998	0.9996	0.9993	0.9990	0.9987
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0012	0.0019	0.9999	0.9999	0.9999	0.9999	0.9998	0.9997	0.9996
Cell shaded if assurance is less than criteria.											

d. Flood Warning Time. Alternative 7B will result in a significant increase in warning time to the population within RD17, North Stockton, and Central Stockton because the probability of flooding from a geotechnical type failure (1-hour warning time) would be reduced and the warning time for overtopping type failures are significantly longer (24 to 36 hours). A description of flood warning time is provided in Alternative 1.

6.7 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system.

a. Flood Depth.

Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Alternative 7B includes fix in place levees, levee raises along the Delta Front, upstream extension of French Camp slough levees, and upstream extension of the RD17 tieback levee. Flood depths in Smith Canal and Fourteenmile slough are not described by index points and would be reduced to 8 feet NAVD88 by the proposed closure structures.

It is unlikely that improvements along French Camp Slough would increase water levels. For these increases to occur a breach of the San Joaquin levee would have had to already occur and the area would already be flooded. Improvement to the RD17 tieback levee was found to increase stages for events larger than 1% ACE for index points along the San Joaquin River, Old River, Middle River, and Paradise cut. It is unlikely that improvements along the delta front levees would increase water levels from delta sources. It is possible that the increased delta front levee height could result in increased flood depths in the floodplain if a levee failure occurred along the Calaveras River or Stockton Diverting Canal. However, the area would already be flooded by the upstream levee breach.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42 and are the same as the no-action condition. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration.

It is unlikely that improvements would change the duration of flooding throughout the system.

c. Frequency. The Delta Front raises and extension of French Camp slough levees upstream are unlikely impact flood frequency. However, improvements to the RD17 tieback levee would impact stages for events more rare than 1% ACE. The computed AEP and assurance values based on only the hydrology and hydraulic inputs are presented in Table 40. Changes to AEP and assurance values are presented in Table 41. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

Table 40
2010 Performance at Selected Locations, Alternative 7B
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9934	0.8544
LR4	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR3	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9999	0.9982	0.9983	0.9781
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9997	0.9989	0.9978
LR1	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9990	0.9987	0.9951	0.9917
French Camp Slough											
FR1	0.0070	0.0679	0.1901	0.2963	0.9999	0.9997	0.9935	0.9328	0.7353	0.4974	0.3465
FL1	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0028	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9829
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0000	0.0003	0.0009	0.0014	0.9999	0.9999	0.9998	0.9996	0.9993	0.9990	0.9987
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0002	0.0023	0.0067	0.0112	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995
Paradise Cut at I-5 F-PCI5	0.0024	0.0240	0.0703	0.1143	0.9999	0.9995	0.9995	0.9995	0.9986	0.9952	0.5404
Paradise Cut at Paradise Rd. F-PCPR	0.0038	0.0376	0.1085	0.1743	0.9999	0.9995	0.9995	0.9995	0.9993	0.6660	0.1373
SDWSC blw Burns Cutoff F-B95660	0.0001	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.

SDWSC- Stockton Deep Water Ship Channel

Table 41
2010 Change in Performance at Selected Locations, Alternative 7B
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Change in Annual Exceedance Probability (Expected)	Change in Long Term Risk			Change in Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	-0.011	-0.1041	-0.2791	-0.417	0	0	0.0042	0.1187	0.4754	0.8019	0.817
LR4	0.0002	0.0027	0.0079	0.0131	0	0	0	-0.0003	-0.0087	-0.0527	-0.1149
LR3	0	0.0004	0.0013	0.0022	0	0	0	0	-0.0016	-0.0009	-0.0201
LR2	0	0	0	0	0	0	0	0	-1E-04	0.0003	0.0006
LR1	-0.0005	-0.0047	-0.0138	-0.0228	0	0	0	0	0.0149	0.07	0.1352
French Camp Slough											
FR1	-0.0039	-0.0357	-0.0895	-0.1248	0	0	0.0006	0.0301	0.1803	0.3098	0.3282
FL1	-0.0031	-0.0302	-0.0876	-0.1415	0	0	0	0.0035	0.0591	0.2728	0.5128
Stockton Diverting Canal											
SL1	0	0	0	0	0	0	0	0	0	0	0
SL2	0	0	0	0	0	0	0	0	0	0	0
Calaveras River											
CR2	0	0	0	0	0	0	0	0	0	0	0
CL2	0	0	0	0	0	0	0	0	0	0	0
Delta Front											
D3	-0.0029	-0.0285	-0.083	-0.1344	0	0.0017	0.0067	0.0182	0.0821	0.2366	0.3784
D4	0	0	0	0	0	0	0	0	0	0	0
D5	0	0	0	0	0	0	0	0	0	0	0
D-BS	0	0	0	0	0	0	0	0	0	0	0
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0	0	0	0	0	0	0	0	0	0	0
Old R. at Clifton Court Ferry F-B95340	-0.0008	0.0013	0.0037	0.0062	0	0	0	0.0001	0.0001	0.0001	0.0001
Paradise Cut at I-5 F-PCI5	0.001	0.01	0.0288	0.0461	0	0	0	0	-0.0009	0	-0.4375
Paradise Cut at Paradise Rd. F-PCPR	0.0021	0.0209	0.0593	0.0936	0	0	0	0	-0.0002	-0.3207	-0.7268
SDWSC blw Burns Cutoff F-B95660	-0.0001	-0.0006	-0.0019	-0.0031	0	0	0	0	0	0	0

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.

SDWSC- Stockton Deep Water Ship Channel

6.8 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 42. Composite floodplain maps were not developed for 2070 conditions.

Table 42
Performance at Simulated Levee Breach Locations, Alternative7B
2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9999	0.9998	0.9976	0.9934	0.9909
LR4	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9999	0.9998	0.9976	0.9934	0.9909
LR3	0.0000	0.0002	0.0005	0.0008	0.9999	0.9999	0.9999	0.9999	0.9995	0.9983	0.9976
LR2	0.0000	0.0005	0.0014	0.0024	0.9999	0.9999	0.9999	0.9999	0.9996	0.9993	0.9991
LR1	0.0013	0.0128	0.0380	0.0626	0.9999	0.9999	0.9999	0.9958	0.9554	0.8735	0.8231
French Camp Slough											
FR1	0.0120	0.1137	0.3037	0.4530	0.9999	0.9938	0.9549	0.8333	0.5886	0.3619	0.2332
FL1	0.0000	0.0001	0.0003	0.0010	0.9999	0.9999	0.9999	0.9999	0.9997	0.9987	0.9987
Stockton Diverting Canal											
SL1	0.0105	0.1003	0.2717	0.4104	0.9999	0.9666	0.9633	0.9509	0.9306	0.9088	0.8900
SL2	0.0153	0.1428	0.3701	0.5372	0.9999	0.9543	0.9220	0.8951	0.8595	0.8090	0.7724
Calaveras River											
CR2	0.0094	0.0903	0.2471	0.3769	0.9999	0.9752	0.9356	0.9011	0.8563	0.8006	0.7440
CL2	0.0168	0.1562	0.3991	0.5721	0.9999	0.9566	0.9410	0.9174	0.8881	0.8576	0.8292
Delta Front											
D3	0.0001	0.0099	0.0294	0.0485	0.9999	0.9967	0.9917	0.9873	0.9824	0.9777	0.9742
D4	0.0001	0.0013	0.0040	0.0067	0.9999	0.9999	0.9999	0.9992	0.9952	0.9826	0.9642
D5	0.0005	0.0047	0.0139	0.0231	0.9999	0.9998	0.9992	0.9965	0.9831	0.9402	0.8794
D-BS	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9993	0.9969	0.9938

6.9 California State Urban Levee Design Criteria

The hydraulic performance of alternative 7B relative to the ULDC requirements for 2070 conditions is provided in Table 43.

Table 43
Alternative 7B Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind Wave Run up and Setup (FT)	Minimum ULDC Required Freeboard	Mean 0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	38.2	6.3	6.3	31.9	6.3	99%
	LR4	RD17	34.9	2.4	3.0	31.9	3.0	99%
	LR3	RD17	31.0	2.4	3.0	27.0	4.0	99%
	LR2	RD17	27.8	2.4	3.0	22.7	5.1	99%
	LR1	RD17	25.0	2.4	3.0	20.8	4.2	87%
French Camp Slough	FR1	CS-02	21.8	<3.0	3.0	16.8	5.0	36%
	FL1	RD17	21.4	<3.0	3.0	16.8	4.6	99%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	30.3	8.1	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.8	4.8	99%
Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	26.5	3.2	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.5	4.9	99%
Delta Front	D3	NS-02	14.9	<3.0	3.0	11.9	3.0	98%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
LRTB assurance based on LR4 index point

7.0 ALTERNATIVE 8A

Alternative 8A provides flood risk reduction benefits to portions of North and Central Stockton economic impact areas. The alternative includes new delta front levee segments, Fix-in-Place levee segments along the Delta front and San Joaquin River, a closure structure at Fourteenmile Slough, and a closure structure at Smith Canal. The alternative also includes levee improvements to the Calaveras River and Stockton Diverting Canal. A summary of the design features associated with Alternative 8A are described below and shown on Plate 69.

7.1 Hydraulic Design Summary

a. General Design. All project features would be designed to meet current USACE design requirements. This alternative would combine the fix-in-place measures of cutoff wall, seismic deep soil mixing, seepage berm, and levee geometry improvements. Descriptions of these improvements are provided in the feasibility study report.

The performance analysis described below assumes the geotechnical performance of the project features would have negligible probability of failure below the design top of levee. It was assumed all levee features would fail completely if overtopped.

b. Levee Design Height. This project would include levee improvements as shown on Plate 69. The levee height would be based on the authorized design profile, the existing profile, or increased height to achieve the DWR ULDC requirements for 2070 sea level conditions, whichever is higher. The models used to define the height of the levee improvements assumed the levees in RD17 also met ULDC requirements. However improvements to the RD17 levees are not included in Alternative 8A and were not included in models used to assess the project performance. The height required to meet ULDC requirements was computed using the HEC-RAS models modified from the no action condition.

c. New Levees. Alternative 8A would extend the levee along the right bank of French Camp Slough upstream to the UPRR rail yard. The design height of new levees is described above.

d. Upstream Reservoir Operation. Alternative 8A does not include any modifications to upstream reservoirs. The hydraulic analysis assumes all upstream reservoirs are operated the same as no-action conditions.

e. Interior Drainage Facilities. Alternative 8A does not include any modifications to interior drainage facilities.

f. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions. Additional operation and maintenance would be required at the Smith Canal and Fourteenmile Slough Closure Structures. It is estimated that vegetation maintenance within 20 feet of the levee toe would have little to no impact on the hydraulic estimates.

g. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (*Design of I-Walls*, 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority. The RD17 and French Camp slough tie back levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile upstream of the project rather than being overtopped within the study area. Flow would outflank the levee only during the peak of the event and would reduce the flow and stage along the levee reaches. The outflanking would occur slowly and allow more evacuation time.

As described above, this alternative would extend the levee along the right bank of French Camp Slough further upstream. However, the natural ground upstream of the levee would remain lower than the proposed levee extension to maintain levee superiority.

h. Erosion Protection. Rock revetment erosion protection would be placed along the proposed delta front levees with long fetches. The results of wind wave analysis conducted for Alternative 8A are presented below.

i. Diversion structures. Alternative 8A does not include any additional diversion structures beyond the no action alternative.

j. Closure Structures.

(1) Smith Canal Closure Structure. A gate type closure structure would be constructed on Smith Canal to provide flood risk reduction from high stages in the Sacramento and San Joaquin Delta. The proposed closure structures would consist of a fixed sheet pile wall structure with an opening gate structure to allow for navigation. The opening portion of the closure structure would be a 50' wide miter gate structure. As needed, a sheet pile floodwall would be constructed adjacent to the control structures to tie the structures into the adjacent levee or high ground areas.

The structure would be closed during peak flood events when the stage reached approximately 8.0 feet NAVD88 or in the event of a levee breach along Smith Canal. The closure structures would prevent the extremely large volume of floodwaters in the Delta from flowing to the breach opening. As a result, the volume of floodwaters from a breach would be restricted to only the volume held in the canal.

(2) Fourteenmile Closure Structure. A gate type closure structure would also be constructed on Fourteenmile Canal to provide flood risk reduction from high stages in the Sacramento and San Joaquin Delta. The structure design is similar to the Smith Canal closure structure.

7.2 Hydrology.

The hydrology associated with Alternative 8A is identical to Alternative 1 (no-action conditions).

7.3 Hydraulic Models and Results

Hydraulic models associated with Alternative 8A were modified to reflect increased levee height required to meet the DWR ULDC requirements for 2070 sea level conditions. Height increases were limited to only the levees providing FRM to the study area and assume the upstream levees in RD17 were also improved to meet the ULDC requirements. Stage and Flow frequency curves are provided in Plates 31A through 31N and 32A through 32E.

7.4 Wind Wave Analysis

The wind wave analysis performed for Alternative 7A is applicable to Alternative 8A. No additional analysis was required to address the additional Calaveras River and Diverting Canal reaches in Alternative 8A because of the relatively short fetch lengths. The estimated wind wave runup results are presented in Table 44.

Table 44: Wind Wave Run-Up and Set Up Results, Alternative 8A

Representative Wind Wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) Grass Lined	1.3%	69	1900 ft	18.0 ft	2.36 ft	0.07 ft	1.0 ft
	5%	47			1.72 ft	0.03 ft	0.6 ft
	20%	33			1.28 ft	0.02 ft	0.3 ft
	50%	14			0.63 ft	0.0 ft	0.1 ft
	95%	5			0.26 ft	0.0 ft	0.0 ft
Delta Front-Fourteen Mile Slough (FM_30_L) Rock Lined	1.3%	54	9300 ft	17.0 ft	2.7 ft	0.2 ft	1.6 ft
	5%	36			1.9 ft	0.1ft	1.0 ft
	20%	25			1.4 ft	0.0 ft	0.6 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
Delta Front-Shima Tract (ST_20_R) Rock Lined	1.3%	54	10100 ft	14.0 ft	2.8 ft	0.3 ft	1.8 ft
	5%	36			2.0 ft	0.1 ft	1.0 ft
	20%	25			1.5 ft	0.1 ft	0.7 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
RD17 Tieback SJR_200_R (Grass Lined)	1.3%	69	24300 ft	14.0 ft	9.5 ft	1.1 ft	7.2 ft
	5%	47			6.4 ft	0.4 ft	4.1 ft
	20%	33			4.4 ft	0.2 ft	2.3 ft
	50%	14			1.7 ft	0.0 ft	0.5 ft
	95%	5			0.5 ft	0.0 ft	0.0 ft

Notes:

* Wave Runup calculated using EurOtop method

**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

7.5 Sedimentation and Channel Stability

Sedimentation and channel stability associated with Alternative 8A is identical to Alternative 1 (no action conditions).

7.6 Performance and Flood Risk

Flood risk to portions of North and Central Stockton would be reduced by Alternative 8A. The performance and residual flood risk associated with this alternative was modeled by adjusting the FDA inputs for breach simulations within the study area.

a. Performance. Performance is described by Annual Exceedance Probability (AEP) and assurance of passing a given hydrologic event. Performance estimates were recomputed assuming no failure until overtopping for reaches improved in the alternative. This was modeled by changing the with-project fragility curves so they had no probability of failure until overtopped. The levee height for the FR1 breach location was modified to account for the extension of the French Camp Slough levee further upstream. The levee height at the D3 breach location was modified to account for levee height increases to meet the ULDC requirement (assuming RD17 levees were also improved to ULDC requirements). These increases were determined to be economically feasible based on incremental net benefit analysis conducted for the initial and focused array of alternatives. All other inputs to calculate assurance were identical to Alternative 1, the no action condition. The FDA input assumptions are described in Table 45. The performance of the project at index points throughout the study area is provided in Table 46.

Table 45
FDA Input for San Joaquin River Performance Calculations
Alternative 8A

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR4	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR3	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR2	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR1	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1	***Raise to 18.5 (b)***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	FL1	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	No Action	***No Fragility***	No Action	MS at Bellota	EPR = 52 yrs
	SL2	No Action	***No Fragility***	No Action	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	No Action	***No Fragility***	No Action	MS at Bellota	EPR = 52 yrs
	Cl2	No Action	***No Fragility***	No Action	MS at Bellota	EPR = 52 yrs
Delta Front	D3	***Raise to 14.9***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D4	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D5	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D-BS	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs

Changes from no action plan shown in bold italics.
(a) Parameters at LR4 used to estimate performance of LRTB
(b) Hydraulic top of levee represented by natural bank upstream of levee.
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Floodplains. Maps showing composite floodplains were developed to demonstrate FRM reliability for Alternative 8A. The composite floodplains are provided in Plates 70 to 77. Table 32 provides the assurance values used to determine if a simulated breach was included in the composite floodplain map. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation project performance.

c. Flood Velocities. Flood velocities for a levee beach would be identical to Alternative 1.

Table 46
Performance at Simulated Levee Breach Locations, Alternative 8A
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0117	0.0110	0.2973	0.4446	0.9999	0.9984	0.9918	0.8749	0.5090	0.1908	0.0384
LR4	0.0073	0.0706	0.1971	0.3064	0.9999	0.9731	0.9525	0.9241	0.8826	0.8423	0.8095
LR3	0.0095	0.0913	0.2496	0.3803	0.9999	0.9761	0.9394	0.8998	0.7938	0.6627	0.5650
LR2	0.0211	0.1923	0.4731	0.6563	0.9999	0.9289	0.8683	0.7922	0.6831	0.5788	0.5161
LR1	0.0126	0.1188	0.3158	0.4688	0.9999	0.9610	0.9400	0.8830	0.7439	0.5772	0.4620
French Camp Slough											
FR1	0.0073	0.0705	0.1969	0.3062	0.9999	0.9999	0.9999	0.9766	0.7718	0.3554	0.0785
FL1	0.0132	0.1245	0.3290	0.4857	0.9999	0.9629	0.9460	0.9208	0.8269	0.6032	0.3857
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9828
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0003	0.0025	0.0076	0.0126	0.9999	0.9999	0.9998	0.9989	0.9896	0.9584	0.9226
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Cell shaded if assurance is less than criteria.											

d. Flood Warning Time. Alternative 8A will result in a significant increase in warning time to the population within North and Central Stockton because the probability of flooding from a geotechnical type failure (1-hour warning time) would be reduced and the warning time for overtopping type failures are significantly longer (24 to 36 hours). A description of flood warning time is provided in Alternative 1.

7.7 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system.

a. Flood Depth.

Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Alternative 8A includes fix in place levees, levee raises along the Delta Front, and an extension of French Camp slough levees upstream. Flood depths in the channel at all index points would be the same as the no action condition. Flood depths in Smith Canal and Fourteenmile slough are not described by index points and would be reduced to 8 feet NAVD88 by the proposed closure structures. It is unlikely that improvements along the delta front levees would increase water levels from delta sources. It is possible that the increased delta front levee height could result in increased flood depths in the floodplain if a levee failure occurred along the Calaveras River or Stockton Diverting Canal. However, the area would already be flooded by the upstream levee breach.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42 and are the same as the no-action condition. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration.

It is unlikely that improvements would change the duration of flooding throughout the system.

c. Frequency. The Delta Front raises and extension of French Camp slough levees upstream are unlikely to have hydraulic impacts that would impact flood frequency. The computed AEP and assurance values based on only the hydrology and hydraulic inputs are presented in Table 47. Changes to AEP and assurance values are presented in Table 48. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

Table 47
2010 Performance at Selected Locations, Alternative 8A
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0113	0.1075	0.2892	0.4338	0.9999	0.9999	0.9957	0.8808	0.5134	0.1915	0.0374
LR4	0.0001	0.0007	0.0022	0.0037	0.9999	0.9999	0.9999	0.9998	0.9975	0.9858	0.9693
LR3	0.0000	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9992	0.9982
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9998	0.9986	0.9972
LR1	0.0005	0.0050	0.0148	0.0245	0.9999	0.9999	0.9999	0.9990	0.9838	0.9251	0.8565
French Camp Slough											
FR1	0.0073	0.0705	0.1969	0.3062	0.9999	0.9999	0.9999	0.9766	0.7718	0.3554	0.0785
FL1	0.0031	0.0306	0.0889	0.1437	0.9999	0.9999	0.9999	0.9964	0.9407	0.7268	0.4865
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0028	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9829
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0003	0.0025	0.0076	0.0126	0.9999	0.9999	0.9998	0.9989	0.9896	0.9584	0.9226
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0010	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9995	0.9994	0.9994	0.9994
Paradise Cut at I-5 F-PCI5	0.0014	0.0140	0.0415	0.682	0.9999	0.9995	0.9995	0.9995	0.9995	0.9952	0.9779
Paradise Cut at Paradise Rd. F-PCPR	0.0017	0.0167	0.0492	0.0807	0.9999	0.9995	0.9995	0.9995	0.9995	0.9867	0.8641
SDWSC blw Burns Cutoff F-B95660	0.0002	0.0016	0.0049	0.0081	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995
Assurance estimates account for stage uncertainty, hydrologic uncertainty only. SDWSC- Stockton Deep Water Ship Channel											

Table 48
2010 Change in Performance at Selected Locations, Alternative 8A
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Change in Annual Exceedance Probability (Expected)	Change in Long Term Risk			Change in Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0	0	0	0	0	0	0	0	0	0	0
LR4	0	0	0	0	0	0	0	0	0	0	0
LR3	0	0	0	0	0	0	0	0	0	0	0
LR2	0	0	0	0	0	0	0	0	0	0	0
LR1	0	0	0	0	0	0	0	0	0	0	0
French Camp Slough											
FR1	-0.0036	-0.0331	-0.0827	-0.1149	0	0.0002	0.007	0.0739	0.2168	0.1678	0.0602
FL1	0	0	0	0	0	0	0	0	0	0	0
Stockton Diverting Canal											
SL1	0	0	0	0	0	0	0	0	0	0	0
SL2	0	0	0	0	0	0	0	0	0	0	0
Calaveras River											
CR2	0	0	0	0	0	0	0	0	0	0	0
CL2	0	0	0	0	0	0	0	0	0	0	0
Delta Front											
D3	-0.0026	-0.0263	-0.0763	-0.1232	0	0.0017	0.0067	0.0175	0.0724	0.196	0.3023
D4	0	0	0	0	0	0	0	0	0	0	0
D5	0	0	0	0	0	0	0	0	0	0	0
D-BS	0	0	0	0	0	0	0	0	0	0	0
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0	0	0	0	0	0	0	0	0	0	0
Old R. at Clifton Court Ferry F-B95340	0	0	0	0	0	0	0	0.0001	0	0	0
Paradise Cut at I-5 F-PCI5	0	0	0	0.6138	0	0	0	0	0	0	0
Paradise Cut at Paradise Rd. F-PCPR	0	0	0	0	0	0	0	0	0	0	0
SDWSC blw Burns Cutoff F-B95660	0	0	0	0	0	0	0	0	0	0	0

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.

SDWSC- Stockton Deep Water Ship Channel

7.8 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at

downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 49. Composite floodplain maps were not developed for 2070 conditions.

Table 49
Performance at Simulated Levee Breach Locations, Alternative8A
2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0118	0.1122	0.3002	0.4483	0.9999	0.9984	0.9912	0.8707	0.5026	0.4440	0.5153
LR4	0.0075	0.0726	0.2023	0.3139	0.9999	0.9725	0.9509	0.9228	0.8819	0.8417	0.8093
LR3	0.0101	0.0968	0.2632	0.3990	0.9999	0.9715	0.9362	0.8962	0.7875	0.6593	0.5652
LR2	0.0257	0.2295	0.5426	0.7285	0.9999	0.9153	0.8415	0.7718	0.6711	0.5736	0.5153
LR1	0.0141	0.1326	0.3475	0.5091	0.9999	0.9567	0.9334	0.8764	0.7412	0.5757	0.4616
French Camp Slough											
FR1	0.0078	0.0753	0.2093	0.3238	0.9999	0.9999	0.9994	0.9679	0.7401	0.3260	0.0673
FL1	0.0202	0.1849	0.4586	0.6403	0.9999	0.9443	0.9244	0.9005	0.8055	0.5999	0.3647
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9088	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9828
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0021	0.0207	0.0608	0.9992	0.9999	0.9968	0.9919	0.9830	0.9331	0.8107	0.6974
D4	0.0001	0.0013	0.0040	0.0067	0.9999	0.9999	0.9999	0.9992	0.9952	0.9826	0.9642
D5	0.0005	0.0047	0.0139	0.0231	0.9999	0.9998	0.9992	0.9965	0.9831	0.9402	0.8794
D-BS	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9993	0.9969	0.9938

7.9 California State Urban Levee Design Criteria

The hydraulic performance of alternative 8A relative to the ULDC requirements for 2070 conditions is provided in Table 50.

Table 50
Alternative 8A Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind Wave Run up (FT)	Minimum ULDC Required Freeboard	Mean 0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	33.9	10.6	10.6	30.0	3.9	99%
	LR4	RD17	33.9	2.4	3.0	30.0	3.9	99%
	LR3	RD17	31.0	2.4	3.0	25.6	5.4	99%
	LR2	RD17	27.8	2.4	3.0	23.0	4.8	99%
	LR1	RD17	25.0	2.4	3.0	22.6	2.4	93%
French Camp Slough	FR1	CS-02	21.8	<3.0	3.0	20.4	1.4	15%
	FL1	RD17	21.4	<3.0	3.0	20.4	1.0	70%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	30.3	8.1	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.8	4.8	99%
Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	26.5	3.2	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.5	4.9	99%
Delta Front	D3	NS-02	14.9	<3.0	3.0	13.6	1.3	81%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
LRTB assurance based on LR4 index point

8.0 ALTERNATIVE 8B

Alternative 8B is similar to 8A but includes additional levee fixes in RD17. A summary of the design features associated with Alternative 8B are described below and shown on Plate 78.

8.1 Hydraulic Design Summary

a. General Design. All project features would be designed to meet current USACE design requirements. This alternative would combine the fix-in-place measures of cutoff wall, seismic deep soil mixing, seepage berm, and levee geometry improvements. Descriptions of these improvements are provided in the feasibility study report.

The performance analysis described below assumes the geotechnical performance of the project features would have negligible probability of failure below the design top of levee. It was assumed all levee features would fail completely if overtopped.

b. Levee Design Height. This project would include levee improvements as shown on Plate 78. The levee height would be based on the authorized design profile, the existing profile, or increased height to achieve the DWR ULDC requirements for 2070 sea level conditions, whichever is higher. The height required to meet ULDC requirements was computed using the HEC-RAS models modified from the no action condition.

c. New Levees. Alternative 8B would extend and raise the RD17 tieback levee at Walthall Slough. The design height of new levees is described above. The levee would be extended to where the natural ground elevation was equivalent to the 0.5% (1/200) ACE median water surface. The extension of French Camp Slough levees described in Alternative 8A would not be included in this alternative.

d. Upstream Reservoir Operation. Alternative 8B does not include any modifications to upstream reservoirs. The hydraulic analysis assumes all upstream reservoirs are operated the same way as the no-action alternative.

e. Interior Drainage Facilities. Alternative 8B does not include any modifications to interior drainage facilities.

f. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions. Additional operation and maintenance would be required at the Smith Canal and Fourteenmile Slough Closure Structures. It is estimated that vegetation maintenance within 20 feet of the levee toe would have little to no impact on the hydraulic estimates.

g. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (*Design of I-Walls,* 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority.

The RD17 and French Camp slough tie back levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile upstream of the project rather than being overtopped within the study area. Flow would outflank the levee only during the peak of the event and would reduce the flow and stage along the levee reaches. The outflanking would occur slowly and allow more evacuation time.

h. Erosion Protection. Erosion protection would be similar to Alternative 8A. However, additional rock revetment erosion protection would be placed along the RD17 tieback levee to address wind wave erosion. The results of wind wave analysis conducted for Alternative 8B are presented below.

i. Diversion structures. Alternative 8B does not include any additional diversion structures beyond the no action alternative.

j. Closure Structures.

(1) Smith Canal Closure Structure. The Smith Canal Closure Structure is identical to Alternative 8A.

(2) Fourteenmile Closure Structure. The Fourteenmile Closure Structure is identical to Alternative 8A.

8.2 Hydrology.

The hydrology associated with Alternative 8B is identical to Alternative 1 (no-action conditions).

8.3 Hydraulic Models and Results

Hydraulic models associated with Alternative 8B were modified to reflect increased levee height required to meet the DWR ULDC requirements for 2070 sea level conditions. Height increases were limited to only the levees providing FRM to the study area. Levees in RD17 were also improved to meet the ULDC requirements. Stage and Flow frequency curves are provided in Plates 31A through 31N and 32A through 32E.

8.4 Wind Wave Analysis

The wind wave analysis performed for Alternative 7A and 7B is applicable to Alternative 8B. No additional analysis was required to address the additional Calaveras River and Diverting Canal reaches in Alternative 8B because of the relatively short fetch lengths. The wind wave estimates for Alternative 8B are provided in Table 51.

Table 51: Wind Wave Run-Up and Set Up Results, Alternative 8B

Representative Wind Wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) Grass Lined	1.3%	69	1900 ft	18.0 ft	2.36 ft	0.07 ft	1.0 ft
	5%	47			1.72 ft	0.03 ft	0.6 ft
	20%	33			1.28 ft	0.02 ft	0.3 ft
	50%	14			0.63 ft	0.0 ft	0.1 ft
	95%	5			0.26 ft	0.0 ft	0.0 ft
Delta Front-Fourteen Mile Slough (FM_30_L) Rock Lined	1.3%	54	9300 ft	17.0 ft	2.7 ft	0.2 ft	1.6 ft
	5%	36			1.9 ft	0.1ft	1.0 ft
	20%	25			1.4 ft	0.0 ft	0.6 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
Delta Front-Shima Tract (ST_20_R) Rock Lined	1.3%	54	10100 ft	14.0 ft	2.8 ft	0.3 ft	1.8 ft
	5%	36			2.0 ft	0.1 ft	1.0 ft
	20%	25			1.5 ft	0.1 ft	0.7 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
RD17 Tieback SJR_200_R (Rock Lined)	1.3%	69	24300 ft	14.0 ft	5.2 ft	1.1 ft	4.5 ft
	5%	47			3.5 ft	0.4 ft	2.4 ft
	20%	33			2.4 ft	0.2 ft	1.4 ft
	50%	14			0.9 ft	0.0 ft	0.3 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft

Notes:

* Wave Runup calculated using EurOtop method
**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

8.5 Sedimentation and Channel Stability

Sedimentation and channel stability associated with Alternative 8B is identical to Alternative 1 (no action conditions).

8.6 Performance and Flood Risk

Flood risk to portions of RD17, North Stockton, and Central Stockton would be reduced by Alternative 8B. The performance and residual flood risk associated with this alternative was modeled by adjusting the FDA inputs for breach simulations within the study area.

a. Performance. Performance is described by Annual Exceedance Probability (AEP) and assurance of passing a given hydrologic event. Performance estimates were recomputed assuming no failure until overtopping for reaches improved in the alternative. This was modeled by changing the with-project fragility curves so they had no probability of failure until overtopped. The levee height at the D3 breach location was modified to account for levee height increases to meet the ULDC requirement (assuming RD17 levees were also improved to ULDC requirements). The levee height of the LRTB index point was modified to account for the extension of the tieback levee. These increases were determined to be economically feasible based on incremental net benefit analysis conducted for the initial and focused array of alternatives. All other inputs to calculate assurance were identical to Alternative 1, the no action condition. The performance of the project at index points throughout the study area is provided in Table 52.

Table 52
FDA Input for San Joaquin River Performance Calculations
Alternative 8B

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	***Raise to 34.9***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	LR4	***Raise to 34.9***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	LR3	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	LR2	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	LR1	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1	***Raise to 18.5 (b)***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	FL1	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	No Action	***No Fragility***	No Action	MS at Bellota	EPR = 52 yrs
	SL2	No Action	***No Fragility***	No Action	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	No Action	***No Fragility***	No Action	MS at Bellota	EPR = 52 yrs
	Cl2	No Action	***No Fragility***	No Action	MS at Bellota	EPR = 52 yrs
Delta Front	D3	***Raise to 14.9***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D4	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D5	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D-BS	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs

Changes from no action plan shown in bold italics.
(a) Parameters at LR4 used to estimate performance of LRTB
(b) Hydraulic top of levee represented by natural bank upstream of levee.
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Floodplains. Maps showing composite floodplains were developed to demonstrate FRM reliability for Alternative 8B. The composite floodplains are provided in Plates 79 to 86. Table 50 provides the assurance values used to determine if a simulated breach was included in the composite floodplain map. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation project performance.

d. Flood Velocities. Flood velocities for a levee beach would be identical to Alternative 1.

Table 53
Performance at Simulated Levee Breach Locations, Alternative 8B
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR4	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR3	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9999	0.9982	0.9898	0.9781
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9997	0.9989	0.9978
LR1	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9999	0.9987	0.9951	0.9917
French Camp Slough											
FR1	0.0070	0.0679	0.1901	0.2963	0.9999	0.9997	0.9935	0.9328	0.7353	0.4974	0.3465
FL1	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9999	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9999	0.9998	0.9984	0.9912	0.9828
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0000	0.0003	0.0009	0.0014	0.9999	0.9999	0.9998	0.9996	0.9993	0.9990	0.9987
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0012	0.0019	0.9999	0.9999	0.9999	0.9999	0.9998	0.9997	0.9996

e. Flood Warning Time. Alternative 8B will result in a significant increase in warning time to the population within RD17, North Stockton, and Central Stockton because the probability of flooding from a geotechnical type failure (1-hour warning time) would be reduced and the warning time for overtopping type failures are significantly longer (24 to 36 hours). A description of flood warning time is provided in Alternative 1.

8.7 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system.

a. Flood Depth.

Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Alternative 8B includes fix in place levees, levee raises along the Delta Front, upstream extension of French Camp slough levees, and upstream extension of the RD17 tieback levee. Flood depths in Smith Canal and Fourteenmile slough are not described by index points and would be reduced to 8 feet NAVD88 by the proposed closure structures.

It is unlikely that improvements along French Camp Slough would increase water levels. For these increases to occur a breach of the San Joaquin levee would have had to already occur and the area would already be flooded. Improvement to the RD17 tieback levee was found to increase stages for events larger than 1% ACE for index points along the San Joaquin River, Old River, Middle River, and Paradise cut. It is unlikely that improvements along the delta front levees would increase water levels from delta sources. It is possible that the increased delta front levee height could result in increased flood depths in the floodplain if a levee failure occurred along the Calaveras River or Stockton Diverting Canal. However, the area would already be flooded by the upstream levee breach.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42 and are the same as the no-action condition. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration.

It is unlikely that improvements would change the duration of flooding throughout the system.

c. Frequency. The Delta Front raises and extension of French Camp slough levees upstream are unlikely impact flood frequency. However, improvements to the RD17 tieback levee would impact stages for events more rare than 1% ACE. The computed AEP and assurance values based on only the hydrology and hydraulic inputs are presented in Table 54. Changes to AEP and assurance values are presented in Table 55. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

Table 54
2010 Performance at Selected Locations, Alternative 8B
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR4	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR3	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9999	0.9982	0.9898	0.9781
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9997	0.9989	0.9978
LR1	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9990	0.9987	0.9951	0.9917
French Camp Slough											
FR1	0.0070	0.0679	0.1901	0.2963	0.9999	0.9997	0.9935	0.9328	0.7353	0.4974	0.3465
FL1	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0028	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9829
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0000	0.0003	0.0009	0.0014	0.9999	0.9999	0.9998	0.9996	0.9993	0.9990	0.9987
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0002	0.0023	0.0067	0.0112	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995
Paradise Cut at I-5 F-PCI5	0.0024	0.0240	0.0703	0.1143	0.9999	0.9995	0.9995	0.9995	0.9986	0.8753	0.5404
Paradise Cut at Paradise Rd. F-PCPR	0.0038	0.0376	0.1085	0.1743	0.9999	0.9995	0.9995	0.9995	0.9993	0.6660	0.1373
SDWSC blw Burns Cutoff F-B95660	0.0001	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995
Assurance estimates account for stage uncertainty, hydrologic uncertainty only. SDWSC- Stockton Deep Water Ship Channel											

Table 55
2010 Change in Performance at Selected Locations, Alternative 8B
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Change in Annual Exceedance Probability (Expected)	Change in Long Term Risk			Change in Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	-0.011	-0.1041	-0.2791	-0.417	0	0	0.0042	0.1187	0.4754	0.7416	0.817
LR4	0.0002	0.0027	0.0079	0.0131	0	0	0	-0.0003	-0.0087	-0.0527	-0.1149
LR3	0	0.0004	0.0013	0.0022	0	0	0	0	-0.0016	-0.0094	-0.0201
LR2	0	0	0	0	0	0	0	0	-1E-04	0.0003	0.0006
LR1	-0.0005	-0.0047	-0.0138	-0.0228	0	0	0	0	0.0149	0.07	0.1352
French Camp Slough											
FR1	-0.0039	-0.0357	-0.0895	-0.1248	0	0	0.0006	0.0301	0.1803	0.3098	0.3282
FL1	-0.0031	-0.0302	-0.0876	-0.1415	0	0	0	0.0035	0.0591	0.2728	0.5128
Stockton Diverting Canal											
SL1	0	0	0	0	0	0	0	0	0	0	0
SL2	0	0	0	0	0	0	0	0	0	0	0
Calaveras River											
CR2	0	0	0	0	0	0	0	0	0	0	0
CL2	0	0	0	0	0	0	0	0	0	0	0
Delta Front											
D3	-0.0029	-0.0285	-0.083	-0.1344	0	0.0017	0.0067	0.0182	0.0821	0.2366	0.3784
D4	0	0	0	0	0	0	0	0	0	0	0
D5	0	0	0	0	0	0	0	0	0	0	0
D-BS	0	0	0	0	0	0	0	0	0	0	0
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0	0	0	0	0	0	0	0	0	0	0
Old R. at Clifton Court Ferry F-B95340	-0.0008	0.0013	0.0037	0.0062	0	0	0	0.0001	0.0001	0.0001	0.0001
Paradise Cut at I-5 F-PCI5	0.001	0.01	0.0288	0.0461	0	0	0	0	-0.0009	-0.1199	-0.4375
Paradise Cut at Paradise Rd. F-PCPR	0.0021	0.0209	0.0593	0.0936	0	0	0	0	-0.0002	-0.3207	-0.7268
SDWSC blw Burns Cutoff F-B95660	-0.0001	-0.0006	-0.0019	-0.0031	0	0	0	0	0	0	0

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.

SDWSC- Stockton Deep Water Ship Channel

8.8 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 56. Composite floodplain maps were not developed for 2070 conditions.

Table 56
Performance at Simulated Levee Breach Locations, Alternative 8B
2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9999	0.9998	0.9976	0.9934	0.9909
LR4	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9999	0.9998	0.9976	0.9934	0.9909
LR3	0.0000	0.0002	0.0005	0.0008	0.9999	0.9999	0.9999	0.9999	0.9995	0.9983	0.9976
LR2	0.0000	0.0005	0.0014	0.0024	0.9999	0.9999	0.9999	0.9999	0.9996	0.9993	0.9991
LR1	0.0013	0.0128	0.0380	0.0626	0.9999	0.9999	0.9999	0.9958	0.9554	0.8735	0.8231
French Camp Slough											
FR1	0.0120	0.1137	0.3037	0.4530	0.9999	0.9938	0.9549	0.8333	0.5886	0.3619	0.2332
FL1	0.0000	0.0001	0.0003	0.0010	0.9999	0.9999	0.9999	0.9999	0.9997	0.9992	0.9987
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0000	0.0002	0.0006	0.0010	0.9999	0.9999	0.9999	0.9999	0.9998	0.9991	0.9976
Calaveras River											
CR2	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9999	0.9998	0.9984	0.9924	0.9828
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0021	0.0207	0.0608	0.9992	0.9999	0.9968	0.9919	0.9830	0.9331	0.9777	0.6974
D4	0.0001	0.0013	0.0040	0.0067	0.9999	0.9999	0.9999	0.9992	0.9952	0.9826	0.9642
D5	0.0005	0.0047	0.0139	0.0231	0.9999	0.9998	0.9992	0.9965	0.9831	0.9402	0.8794
D-BS	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9993	0.9996	0.9938

8.9 California State Urban Levee Design Criteria

The hydraulic performance of alternative 8B relative to the ULDC requirements for 2070 conditions is provided in Table 57.

Table 57
Alternative 8B Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind Wave Run up (FT)	Minimum ULDC Required Freeboard	Mean 0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	38.2	6.3	6.3	31.9	6.3	99%
	LR4	RD17	34.9	2.4	3.0	31.9	3.0	99%
	LR3	RD17	31.0	2.4	3.0	27.0	4.0	99%
	LR2	RD17	27.8	2.4	3.0	22.7	5.1	99%
	LR1	RD17	25.0	2.4	3.0	20.8	4.2	87%
French Camp Slough	FR1	CS-02	21.8	<3.0	3.0	16.8	5.0	36%
	FL1	RD17	21.4	<3.0	3.0	16.8	4.6	99%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	30.3	8.1	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.8	4.8	99%
Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	26.5	3.2	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.5	4.9	99%
Delta Front	D3	NS-02	14.9	<3.0	3.0	11.9	3.0	98%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
LRTB assurance based on LR4 index point

9.0 ALTERNATIVE 9A

Alternative 9A provides flood risk reduction benefits to portions of North and Central Stockton economic impact areas. The alternative includes new delta front levee segments, Fix-in-Place levee segments along the Delta front and San Joaquin River, a closure structure at Fourteenmile Slough, and a closure structure at Smith Canal. The alternative also includes a diversion structure to divert floodwaters from the Stockton diverting canal into the Mormon channel (Mormon Slough Bypass) and channel improvements to safely convey those flows to the Stockton Deep Water Ship Channel. A summary of the design features associated with Alternative 9A are described below and shown on Plate 87.

9.1 Hydraulic Design Summary

a. General Design. All project features would be designed to meet current USACE design requirements. This alternative would combine the fix-in-place measures of cutoff wall, seismic deep soil mixing, seepage berm, and levee geometry improvements. Descriptions of these improvements are provided in the feasibility study report.

The performance analysis described below assumes the geotechnical performance of the project features would have negligible probability of failure below the design top of levee. It was assumed all levee features would fail completely if overtopped.

b. Levee Design Height. This project would include levee improvements as shown on Plate 87. The levee height would be based on the authorized design profile, the existing profile, or increased height to achieve the DWR ULDC requirements for 2070 sea level conditions, whichever is higher. The models used to define the improvements assumed the levees in RD17 also met ULDC requirements. However improvements to the RD17 levees are not included in Alternative 9A and were not included in models used to assess the project performance. The height required to meet ULDC requirements was computed using the HEC-RAS models modified from the no action condition.

c. New Levees. Alternative 9A would extend the levee along the right bank of French Camp Slough upstream to the UPRR rail yard. The design height of new levees is described above.

d. Upstream Reservoir Operation. Alternative 9A does not include any modifications to upstream reservoirs. The hydraulic analysis assumes all upstream reservoirs are operated the same as no-action conditions.

e. Interior Drainage Facilities. Alternative 9A does not include any modifications to interior drainage facilities.

f. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions. Additional operation and maintenance would be required at the Smith Canal and Fourteenmile Slough Closure Structures. It is estimated that vegetation maintenance within 20 feet of the levee toe would have little to no impact on the hydraulic estimates.

g. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (*Design of I-Walls*, 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority. The RD17 and French Camp slough tie back levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile upstream of the project rather than being overtopped within the study area. Flow would outflank the levee only during the peak of the event and would reduce the flow and stage along the levee reaches. The outflanking would occur slowly and allow more evacuation time.

As described above, this alternative would extend the levee along the right bank of French Camp Slough further upstream. However, the natural ground upstream of the levee would remain lower than the proposed levee extension to maintain levee superiority.

h. Erosion Protection. Rock revetment erosion protection would be placed along the proposed delta front levees with long fetches. The results of wind wave analysis conducted for Alternative 9A are presented below.

i. Diversion structures. The design includes of a diversion structure to divert floodwaters from the Stockton Diverting canal into the Mormon Channel (Mormon Slough Bypass) and channel improvements to safely convey those flows to the Stockton Deep Water Ship Channel. The diversion structure would consist of an inlet apron, series of 8 radial gates, a box culvert, and outlet apron. A maximum flood flow diversion rate of 1,200cfs was selected based on the ability of downstream channel improvements to pass this flow including additional localized runoff with 90% assurance of not overtopping. The design flow, allowing for localized inflow, is 1,200cfs from the diversion structure to Highway 99, 1,550cfs from Highway 99 to Stanislaus Street, and 1,700 cfs from Stanislaus Street to the Deep Water Ship Channel. The design includes no levees along the bypass. The selected design of the downstream improvements was estimated to maximize economic benefits because a larger size would require a substantial increase in the scale of improvements.

j. Closure Structures.

(1) Smith Canal Closure Structure. A gate type closure structure would be constructed on Smith Canal to provide flood risk reduction from high stages in the Sacramento and San Joaquin Delta. The proposed closure structures would consist of a fixed sheet pile wall structure with an opening gate structure to allow for navigation. The opening portion of the closure structure would be a 50' wide miter gate structure. As needed, a sheet pile floodwall would be constructed adjacent to the control structures to tie the structures into the adjacent levee or high ground areas

The structure would be closed during peak flood events when the stage reached approximately 8.0 feet NAVD88 or in the event of a levee breach along Smith Canal. The closure structures would prevent the extremely large volume of floodwaters in the Delta from flowing to the breach opening. As a result, the volume of floodwaters from a breach would be restricted to only the volume held in the canal.

(2) Fourteenmile Closure Structure. A gate type closure structure would also be constructed on Fourteenmile Canal to provide flood risk reduction from high stages in the Sacramento and San Joaquin Delta. The structure design is similar to the Smith Canal closure structure.

9.2 Hydrology.

The diversion into the Mormon Slough Bypass Channel would change the flood flow frequency for the Stockton Diverting Canal, Lower Calaveras River, and Mormon Slough Bypass Channel. The estimated flow diversion is described in Table 58. Inflow to the diversion was based on flow at the SL2 index point for the no action alternative.

Table 58
Estimated Flood Flow Frequency of Mormon Slough Bypass

Parameter	Annual Chance Exceedance						
	50% ACE	**10% ACE**	**4% ACE**	**2% ACE**	**1% ACE**	**0.5% ACE**	**0.2% ACE**
Inflow to Proposed Diversion (CFS)	3740	9650	11920	12720	14810	15200	18240
Flow to Stockton Diverting Canal (CFS)	3740	8450	10720	11510	13610	14000	17240
Flow to Mormon Bypass (CFS)	0	1200	1200	1200	1200	1200	1200
Average Duration of Diversion (Days)	0	5	8	9	11	12	14
Diversion flows obtained from PBI, 2013C							

9.3 Hydraulic Models and Results

Hydraulic models associated with Alternative 9A were modified to reflect increased levee height required to meet the DWR ULDC requirements for 2070 sea level conditions. Height increases were limited to only the levees providing FRM to the study area. It was assumed the upstream levees in RD17 were also improved to meet the ULDC requirements. Stage and Flow frequency curves are provided in Plates 31A through 31N and 32A through 32E.

9.4 Wind Wave Analysis

The wind wave analysis performed for Alternative 7A is applicable to Alternative 9A. No additional analysis was required to address the additional Calaveras River and Diverting Canal reaches or Mormon Slough Bypass in Alternative 9A because of the relatively short fetch lengths. The estimated wind wave runup results are presented in Table 59.

Table 59: Wind Wave Run-Up and Set Up Results, Alternative 9A

Representative Wind Wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) Grass Lined	1.3%	69	1900 ft	18.0 ft	2.36 ft	0.07 ft	1.0 ft
	5%	47			1.72 ft	0.03 ft	0.6 ft
	20%	33			1.28 ft	0.02 ft	0.3 ft
	50%	14			0.63 ft	0.0 ft	0.1 ft
	95%	5			0.26 ft	0.0 ft	0.0 ft
Delta Front-Fourteen Mile Slough (FM_30_L) Rock Lined	1.3%	54	9300 ft	17.0 ft	2.7 ft	0.2 ft	1.6 ft
	5%	36			1.9 ft	0.1ft	1.0 ft
	20%	25			1.4 ft	0.0 ft	0.6 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
Delta Front-Shima Tract (ST_20_R) Rock Lined	1.3%	54	10100 ft	14.0 ft	2.8 ft	0.3 ft	1.8 ft
	5%	36			2.0 ft	0.1 ft	1.0 ft
	20%	25			1.5 ft	0.1 ft	0.7 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
RD17 Tieback SJR_200_R (Grass Lined)	1.3%	69	24300 ft	14.0 ft	9.5 ft	1.1 ft	7.2 ft
	5%	47			6.4 ft	0.4 ft	4.1 ft
	20%	33			4.4 ft	0.2 ft	2.3 ft
	50%	14			1.7 ft	0.0 ft	0.5 ft
	95%	5			0.5 ft	0.0 ft	0.0 ft

Notes:

* Wave Runup calculated using EurOtop method
**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

9.5 Sedimentation and Channel Stability

Sedimentation and channel stability associated with Alternative 9A is identical to Alternative 1 (no action conditions) for all locations except the Stockton Deep Water Ship Channel. The proposed project could increase sediment deposition in the Turning Basin of the Stockton Ship Channel. Although the proposed diversion will likely divert negligible bed load, it will divert suspended load. This material size will likely be transported in the higher transport capacity reaches of the proposed bypass without deposition. However, it will likely fall out of suspension in the low transport capacity ship channel turning basin. Without any analysis it should be assumed that about half of the suspended sediment in the diverted flood flows would be deposited in the ship channel turning basin. This estimate could be used to estimate the potential for additional O&M dredging in the turning basin associated with the proposed diversion

9.6 Performance and Flood Risk

Flood risk to portions of North and Central Stockton would be reduced by Alternative 9A. The performance and residual flood risk associated with this alternative was modeled by adjusting the FDA inputs for breach simulations within the study area.

a. Performance. Performance is described by Annual Exceedance Probability (AEP) and assurance of passing a given hydrologic event. Performance estimates were recomputed assuming no failure until overtopping for reaches improved in the alternative. This was modeled by changing the with-project fragility curves so they had no probability of failure until overtopped. The levee height for the FR1 breach location was modified to account for the extension of the French Camp Slough levee further upstream. The levee height at the D3 breach location was modified to account for levee height increases to meet the ULDC requirement (assuming RD17 levees were also improved to ULDC requirements). These increases were determined to be economically feasible based on incremental net benefit analysis conducted for the initial and focused array of alternatives. All other inputs to calculate assurance were identical to Alternative 1, the no action condition. The FDA input assumptions are described in Table 60. The performance of the project at index points throughout the study area is provided in Table 61.

Table 60
FDA Input for San Joaquin River Performance Calculations
Alternative 9A

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR4	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR3	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR2	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
	LR1	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1	***Raise to 18.5 (b)***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	FL1	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
	SL2	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
	Cl2	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
Delta Front	D3	***Raise to 14.9***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D4	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D5	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D-BS	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs

Changes from no action plan shown in bold italics.
(a) Parameters at LR4 used to estimate performance of LRTB
(b) Hydraulic top of levee represented by natural bank upstream of levee.
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Floodplains. Maps showing composite floodplains were developed to demonstrate FRM reliability for Alternative 9A. The composite floodplains are provided in Plates 88 to 96. Table 57provides the assurance values used to determine if a simulated breach was included in the composite floodplain map. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation project performance.

c. Flood Velocities. Flood velocities for a levee beach would be identical to Alternative 1.

Table 61
Performance at Simulated Levee Breach Locations, Alternative 9A
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0117	0.0110	0.2973	0.4446	0.9999	0.9984	0.9918	0.8749	0.5090	0.1908	0.0384
LR4	0.0073	0.0706	0.1971	0.3064	0.9999	0.9731	0.9525	0.9241	0.8826	0.8423	0.8095
LR3	0.0095	0.0913	0.2496	0.3803	0.9999	0.9761	0.9394	0.8998	0.7938	0.6627	0.5650
LR2	0.0211	0.1923	0.4731	0.6563	0.9999	0.9289	0.8683	0.7922	0.6831	0.5788	0.5161
LR1	0.0126	0.1188	0.3158	0.4688	0.9999	0.9610	0.9400	0.8830	0.7439	0.5772	0.4620
French Camp Slough											
FR1	0.0073	0.0705	0.1969	0.3062	0.9999	0.9999	0.9999	0.9766	0.7718	0.3554	0.0785
FL1	0.0132	0.1245	0.3290	0.4857	0.9999	0.9629	0.9460	0.9208	0.8269	0.6032	0.3857
Stockton Diverting Canal											
SL1	0.0089	0.0859	0.2363	0.3619	0.9999	0.9670	0.9661	0.9606	0.9469	0.9262	0.9057
SL2	0.0109	0.1036	0.2797	0.4211	0.9999	0.9700	0.9432	0.9194	0.8897	0.8480	0.8029
Calaveras River											
CR2	0.0051	0.0497	0.1419	0.2251	0.9999	0.9916	0.9619	0.9320	0.8920	0.8444	0.7965
CL2	0.0145	0.1361	0.3552	0.5187	0.9999	0.9577	0.9533	0.9374	0.9110	0.8813	0.8536
Delta Front											
D3	0.0003	0.0025	0.0076	0.0126	0.9999	0.9999	0.9998	0.9989	0.9896	0.9584	0.9226
D4	0.0000	0.0003	0.0008	0.0014	0.9999	0.9999	0.9999	0.9999	0.9995	0.9909	0.9950
D5	0.0000	0.0005	0.0016	0.0026	0.9999	0.9999	0.9999	0.9998	0.9986	0.9799	0.9864
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Cell shaded if assurance is less than criteria.											

d. Flood Warning Time. Alternative 9A will result in a significant increase in warning time to the population within North and Central Stockton because the probability of flooding from a geotechnical type failure (1-hour warning time) would be reduced and the warning time for overtopping type failures are significantly longer (24 to 36 hours). A description of flood warning time is provided in Alternative 1.

9.7 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system.

a. Flood Depth.

Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Alternative 9A includes fix in place levees, levee raises along the Delta Front, and diversion of flood flows into old mormon channel. Flood depths in the channel at all index points would be the same as the no action condition except the Stockton Diverting Canal and Lower Calaveras River. Stages in the Stockton Diverting Canal and Lower Calaveras River would be lowered because of the upstream diversion to Old Mormon Channel. Flood depths in Smith Canal and Fourteenmile slough are not described by index points and would be reduced to 8 feet NAVD88 by the proposed closure structures. For magnitudes greater than 33% (1/3) ACE, stages in Old Mormon Channel would be increased due to the upstream diversion. It is unlikely that improvements along the delta front levees would increase water levels from delta sources. It is possible that the increased delta front levee height could result in increased flood depths in the floodplain if a levee failure occurred along the Calaveras River or Stockton Diverting Canal. However, the area would already be flooded by the upstream levee breach.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42 and are the same as the no-action condition. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration.

It is unlikely that improvements would change the duration of flooding throughout the system.

c. Frequency. The Delta Front raises and extension of French Camp slough levees upstream are unlikely to have hydraulic impacts that would impact flood frequency. The frequency of flood flows in the Old Mormon Channel would be increased due to the upstream diversion. The computed AEP and assurance values based on only the hydrology and hydraulic inputs are presented in Table 62. Changes to AEP and assurance values are presented in Table 63. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

Table 62
2010 Performance at Selected Locations, Alternative 9A
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0113	0.1075	0.2892	0.4338	0.9999	0.9999	0.9957	0.8808	0.5134	0.1915	0.0374
LR4	0.0001	0.0007	0.0022	0.0037	0.9999	0.9999	0.9999	0.9998	0.9975	0.9858	0.9693
LR3	0.0000	0.0001	0.0003	0.0005	0.9999	0.9999	0.9999	0.9999	0.9998	0.9992	0.9982
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9998	0.9986	0.9972
LR1	0.0005	0.0050	0.0148	0.0245	0.9999	0.9999	0.9999	0.9990	0.9838	0.9251	0.8565
French Camp Slough											
FR1	0.0073	0.0705	0.1969	0.3062	0.9999	0.9999	0.9999	0.9766	0.7718	0.3554	0.0785
FL1	0.0031	0.0306	0.0889	0.1437	0.9999	0.9999	0.9999	0.9964	0.9407	0.7268	0.4865
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0001	0.0007	0.0021	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Calaveras River											
CR2	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9997	0.9985	0.9963
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0003	0.0025	0.0076	0.0126	0.9999	0.9999	0.9998	0.9989	0.9896	0.9584	0.9226
D4	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9998	0.9980	0.9909	0.9799
D5	0.0000	0.0005	0.0016	0.0026	0.9999	0.9999	0.9999	0.9998	0.9986	0.9939	0.9864
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0010	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9995	0.9994	0.9994	0.9994
Paradise Cut at I-5 F-PCI5	0.0014	0.0140	0.0415	0.682	0.9999	0.9995	0.9995	0.9995	0.9995	0.9952	0.9779
Paradise Cut at Paradise Rd. F-PCPR	0.0017	0.0167	0.0492	0.0807	0.9999	0.9995	0.9995	0.9995	0.9995	0.9867	0.8641
SDWSC blw Burns Cutoff F-B95660	0.0002	0.0016	0.0049	0.0081	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.

SDWSC- Stockton Deep Water Ship Channel

Table 63
2010 Change in Performance at Selected Locations, Alternative 9A
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Change in Annual Exceedance Probability (Expected)	Change in Long Term Risk			Change in Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0	0	0	0	0	0	0	0	0	0	0
LR4	0	0	0	0	0	0	0	0	0	0	0
LR3	0	0	0	0	0	0	0	0	0	0	0
LR2	0	0	0	0	0	0	0	0	0	0	0
LR1	0	0	0	0	0	0	0	0	0	0	0
French Camp Slough											
FR1	-0.0036	-0.0331	-0.0827	-0.1149	0	0.0002	0.007	0.0739	0.2168	0.1678	0.0602
FL1	0	0	0	0	0	0	0	0	0	0	0
Stockton Diverting Canal											
SL1	0	0	0	0	0	0	0	0	0	0	0
SL2	0.0001	0.0005	0.0015	0.0024	0	0	0	0	1E-04	0.0007	0.0022
Calaveras River											
CR2	-0.0001	-0.0004	-0.001	-0.0016	0	0	0	1E-04	0.0013	0.0061	0.0134
CL2	0	0	0	0	0	0	0	0	0	0	0
Delta Front											
D3	-0.0026	-0.0263	-0.0763	-0.1232	0	0.0017	0.0067	0.0175	0.0724	0.196	0.3023
D4	0	0	0	0	0	0	0	0	0	0	0
D5	-0.0001	-0.0009	-0.0025	-0.0042	0	0	0	0.0004	0.0035	0.014	0.03
D-BS	0	0	0	0	0	0	0	0	0	0	0
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0	0	0	0	0	0	0	0	0	0	0
Old R. at Clifton Court Ferry F-B95340	0	0	0	0	0	0	0	0.0001	0	0	0
Paradise Cut at I-5 F-PCI5	0	0	0	0.6138	0	0	0	0	0	0	0
Paradise Cut at Paradise Rd. F-PCPR	0	0	0	0	0	0	0	0	0	0	0
SDWSC blw Burns Cutoff F-B95660	0	0	0	0	0	0	0	0	0	0	0
Assurance estimates account for stage uncertainty, hydrologic uncertainty only. SDWSC- Stockton Deep Water Ship Channel											

9.8 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at

downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 64. Composite floodplain maps were not developed for 2070 conditions.

Table 64
Performance at Simulated Levee Breach Locations, Alternative9A
2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0118	0.1122	0.3002	0.4483	0.9999	0.9984	0.9912	0.8707	0.5026	0.4440	0.5153
LR4	0.0075	0.0726	0.2023	0.3139	0.9999	0.9725	0.9509	0.9228	0.8819	0.8417	0.8093
LR3	0.0101	0.0968	0.2632	0.3990	0.9999	0.9715	0.9362	0.8962	0.7875	0.6593	0.5652
LR2	0.0257	0.2295	0.5426	0.7285	0.9999	0.9153	0.8415	0.7718	0.6711	0.5736	0.5153
LR1	0.0141	0.1326	0.3475	0.5091	0.9999	0.9567	0.9334	0.8764	0.7412	0.5757	0.4616
French Camp Slough											
FR1	0.0078	0.0753	0.2093	0.3238	0.9999	0.9999	0.9994	0.9679	0.7401	0.3260	0.0673
FL1	0.0202	0.1849	0.4586	0.6403	0.9999	0.9443	0.9244	0.9005	0.8055	0.5790	0.3647
Stockton Diverting Canal											
SL1	0.0089	0.0859	0.2363	0.3619	0.9999	0.9670	0.9661	0.9606	0.9469	0.9262	0.9057
SL2	0.0109	0.1036	0.2797	0.4211	0.9999	0.9700	0.9432	0.9194	0.8897	0.8480	0.8029
Calaveras River											
CR2	0.0051		0.1419	0.2251	0.9999	0.9916	0.9619	0.9320	0.8921	0.8444	0.7965
CL2	0.0145	0.1361	0.3552	0.5187	0.9999	0.9577	0.9533	0.9374	0.9110	0.8813	0.8536
Delta Front											
D3	0.0021	0.0207	0.0608	0.9992	0.9999	0.9968	0.9919	0.9830	0.9331	0.8107	0.6974
D4	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9999	0.9997	0.9983	0.9826	0.9861
D5	0.0002	0.0019	0.0058	0.0096	0.9999	0.9999	0.9997	0.9987	0.9932	0.9753	0.9482
D-BS	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9993	0.9969	0.9938

9.9 California State Urban Levee Design Criteria

The hydraulic performance of alternative 9A relative to the ULDC requirements for 2070 conditions is provided in Table 65.

Table 65
Alternative 9A Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind Wave Run up (FT)	Minimum ULDC Required Freeboard	Mean 0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	33.9	10.6	10.6	30.0	3.9	99%
	LR4	RD17	33.9	2.4	3.0	30.0	3.9	99%
	LR3	RD17	31.0	2.4	3.0	25.6	5.4	99%
	LR2	RD17	27.8	2.4	3.0	23.0	4.8	99%
	LR1	RD17	25.0	2.4	3.0	22.6	2.4	93%
French Camp Slough	FR1	CS-02	21.8	<3.0	3.0	20.4	1.4	15%
	FL1	RD17	21.4	<3.0	3.0	20.4	1.0	70%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	29.8	9.4	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.3	5.3	99%
Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	25.1	4.6	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.0	5.4	99%
Delta Front	D3	NS-02	14.9	<3.0	3.0	13.6	1.3	81%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
LRTB assurance based on LR4 index point

10.0 ALTERNATIVE 9B

Alternative 9B is similar to 9A but includes additional levee fixes in RD17. A summary of the design features associated with Alternative 9B are described below and shown on Plate 96.

10.1 Hydraulic Design Summary

a. General Design. All project features would be designed to meet current USACE design requirements. This alternative would combine the fix-in-place measures of cutoff wall, seismic deep soil mixing, seepage berm, and levee geometry improvements. Descriptions of these improvements are provided in the feasibility study report.

The performance analysis described below assumes the geotechnical performance of the project features would have negligible probability of failure below the design top of levee. It was assumed all levee features would fail completely if overtopped.

b. Levee Design Height. This project would include levee improvements as shown on Plate 96. The levee height would be based on the authorized design profile, the existing profile, or increased height to achieve the DWR ULDC requirements for 2070 sea level conditions, whichever is higher. The height required to meet ULDC requirements was computed using the HEC-RAS models modified from the no action condition.

c. New Levees. Alternative 9B would extend and raise the RD17 tieback levee at Walthall Slough. The levee would be extended to where the natural ground elevation was equivalent to the 0.5% (1/200) ACE median water surface. The design height of new levees is described above. The extension of French Camp Slough levees described in Alternative 9A would not be included in this alternative.

d. Upstream Reservoir Operation. Alternative 9B does not include any modifications to upstream reservoirs. The hydraulic analysis assumes all upstream reservoirs are operated the same way as the no-action alternative.

e. Interior Drainage Facilities. Alternative 9B does not include any modifications to interior drainage facilities.

f. Operation and Maintenance. The hydraulic analysis assumes vegetation conditions within the channel will be maintained with similar hydraulic conditions as the existing conditions. Additional operation and maintenance would be required at the Smith Canal and Fourteenmile Slough Closure Structures. It is estimated that vegetation maintenance within 20 feet of the levee toe would have little to no impact on the hydraulic estimates.

g. Levee Superiority. The definition of levee superiority per EC 1110-2-6066 (*Design of I-Walls,* 31 October 2010) is the increment of additional height added to a flood risk management system to increase the likelihood that when the design event is exceeded, controlled flooding will occur at the design overtopping section. Water surface profiles from the HEC-RAS hydraulic model indicate the existing levee system includes design features that address levee superiority.

The RD17 and French Camp slough tie back levees have a higher assurance than the natural ground profile upstream from the levee. As a result, it is more likely the levee would be outflanked along the natural ground profile upstream of the project rather than being overtopped within the study area. Flow would outflank the levee only during the peak of the event and would reduce the flow and stage along the levee reaches. The outflanking would occur slowly and allow more evacuation time.

h. Erosion Protection. Erosion protection would be similar to Alternative 9A. However, additional rock revetment erosion protection would be placed along the RD17 tieback levee to address wind wave erosion. The results of wind wave analysis conducted for Alternative 9B are presented below.

i. Diversion structures. Alternative 9B does not include any additional diversion structures beyond the no action alternative.

j. Smith Canal Closure Structure. The Smith Canal Closure Structure is identical to Alternative 9A.

j. Fourteenmile Closure Structure. The Fourteenmile Closure Structure is identical to Alternative 9A.

10.2 Hydrology.

The diversion into the Mormon Slough Bypass Channel would change the flood flow frequency for the Stockton Diverting Canal, Lower Calaveras River, and Mormon Slough Bypass Channel. The estimated flow diversion is described in Table 66. Inflow to the diversion was based on flow at the SL2 index point for the no action alternative.

Table 66
Estimated Flood Flow Frequency of Mormon Slough Bypass

Parameter	Annual Chance Exceedance						
	50% ACE	**10% ACE**	**4% ACE**	**2% ACE**	**1% ACE**	**0.5% ACE**	**0.2% ACE**
Inflow to Proposed Diversion (CFS)	3740	9650	11920	12720	14810	15200	18240
Flow to Stockton Diverting Canal (CFS)	3740	8450	10720	11510	13610	14000	17240
Flow to Mormon Bypass (CFS)	0	1200	1200	1200	1200	1200	1200
Average Duration of Diversion (Days)	0	5	8	9	11	12	14
Diversion flows obtained from PBI, 2013C							

10.3 Hydraulic Models and Results

Hydraulic models associated with Alternative 9B were modified to reflect increased levee height required to meet the DWR ULDC requirements for 2070 sea level conditions. Height increases were limited to only the levees providing FRM to the study area. Levees in RD17 were also improved to meet the ULDC requirements. Stage and Flow frequency curves are provided in Plates 31A through 31N and 32A through 32E.

10.4 Wind Wave Analysis

The wind wave analysis performed for Alternative 7A and 7B is applicable to Alternative 9B. No additional analysis was required to address the additional Calaveras River, Diverting Canal, and Mormon Slough Bypass Reaches in Alternative 9B because of the relatively short fetch lengths. The wind wave estimates for Alternative 7B are provided in Table 67.

Table 67: Wind Wave Run-Up and Set Up Results, Alternative 9B

Representative Wind Wave Reaches	Wind Frequency (ACE)	1-hr Wind Stress (mph)	Average Fetch Length (Feet)	Average Fetch Depth (Feet)	Wave Runup* Ru2% (Feet)	Wind Setup (Feet)	Likely Wind Induced Overtopping Failure Point** (Feet below Levee Crest)
San Joaquin River Main Stem (SJR_160_R) Grass Lined	1.3%	69	1900 ft	18.0 ft	2.36 ft	0.07 ft	1.0 ft
	5%	47			1.72 ft	0.03 ft	0.6 ft
	20%	33			1.28 ft	0.02 ft	0.3 ft
	50%	14			0.63 ft	0.0 ft	0.1 ft
	95%	5			0.26 ft	0.0 ft	0.0 ft
Delta Front-Fourteen Mile Slough (FM_30_L) Rock Lined	1.3%	54	9300 ft	17.0 ft	2.7 ft	0.2 ft	1.6 ft
	5%	36			1.9 ft	0.1ft	1.0 ft
	20%	25			1.4 ft	0.0 ft	0.6 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
Delta Front-Shima Tract (ST_20_R) Rock Lined	1.3%	54	10100 ft	14.0 ft	2.8 ft	0.3 ft	1.8 ft
	5%	36			2.0 ft	0.1 ft	1.0 ft
	20%	25			1.5 ft	0.1 ft	0.7 ft
	50%	10			0.6 ft	0.0 ft	0.1 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft
RD17 Tieback SJR_200_R (Rock Lined)	1.3%	69	24300 ft	14.0 ft	5.2 ft	1.1 ft	4.5 ft
	5%	47			3.5 ft	0.4 ft	2.4 ft
	20%	33			2.4 ft	0.2 ft	1.4 ft
	50%	14			0.9 ft	0.0 ft	0.3 ft
	95%	5			0.3 ft	0.0 ft	0.0 ft

Notes:

* Wave Runup calculated using EurOtop method
**Likely Wind Induced Overtopping Failure Point is the height the levee crest must be above the still water level (SWL) to have less than 0.05 cfs/ft of overtopping discharge from the design wind.

10.5 Sedimentation and Channel Stability

Sedimentation and channel stability associated with Alternative 9B is identical to Alternative 1 (no action conditions) for all locations except the Stockton Deep Water Ship Channel. The proposed project could increase sediment deposition in the Turning Basin of the Stockton Ship Channel. Although the proposed diversion will likely divert negligible bed load, it will divert suspended load. This material size will likely be transported in the higher transport capacity reaches of the proposed bypass without deposition. However, it will likely fall out of suspension in the low transport capacity ship channel turning basin. Without any analysis it should be assumed that about half of the suspended sediment in the diverted flood flows would be deposited in the ship channel turning basin. This estimate could be used to estimate the potential for additional O&M dredging in the turning basin associated with the proposed diversion.

10.6 Performance and Flood Risk

Flood risk to portions of RD17, North Stockton, and Central Stockton would be reduced by Alternative 9B. The performance and residual flood risk associated with this alternative was modeled by adjusting the FDA inputs for breach simulations within the study area.

a. Performance. Performance is described by Annual Exceedance Probability (AEP) and assurance of passing a given hydrologic event. Performance estimates were recomputed assuming no failure until overtopping for reaches improved in the alternative. This was modeled by changing the with-project fragility curves so they had no probability of failure until overtopped. The levee height at the D3 breach location was modified to account for levee height increases to meet the ULDC requirement (assuming RD17 levees were also improved to ULDC requirements). The levee height of the LRTB index point was modified to account for the extension of the tieback levee. These increases were determined to be economically feasible based on incremental net benefit analysis conducted for the initial and focused array of alternatives. All other inputs to calculate assurance were identical to Alternative 1, the no action condition. The FDA input assumptions are described in Table 68. The performance of the project at index points throughout the study area is provided in Table 69.

Table 68
FDA Input for San Joaquin River Performance Calculations
Alternative 9B

Flood Source	Breach Location	Hydraulic Top of Levee (FT-NAVD88)	Geotechnical Fragility Curve	Stage-Discharge Curve	Unregulated Flow Frequency Curve	Notes
San Joaquin River	LRTB	***Raise to 34.9***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	LR4	***Raise to 34.9***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	LR3	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	LR2	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	LR1	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
French Camp Slough	FR1	***Raise to 18.5 (b)***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	FL1	No Action	No Action	No Action	SJR nr Vernalis	EPR = 82yrs
Stockton Diverting Canal	SL1	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
	SL2	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
Calaveras River	CR2	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
	Cl2	No Action	No Action	No Action	MS at Bellota	EPR = 52 yrs
Delta Front	D3	***Raise to 14.9***	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D4	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D5	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs
	D-BS	No Action	***No Fragility***	No Action	SJR nr Vernalis	EPR = 82yrs

Changes from no action plan shown in bold italics.
(a) Parameters at LR4 used to estimate performance of LRTB
(b) Hydraulic top of levee represented by natural bank upstream of levee.
EPR - Equivalent Period of Record
SJR - San Joaquin River
MS - Mormon Slough

b. Composite Floodplains. Maps showing composite floodplains were developed to demonstrate FRM reliability for Alternative 9B. The composite floodplains are provided in Plates 98 to 104. Table 64 provides the assurance values used to determine if a simulated breach was included in the composite floodplain map. The composite flood maps demonstrate the variation of flood risk management assurance throughout the study area. The maps are not directly comparable with FEMA or DWR ULOP criteria because those criteria do not include fragility in the estimation project performance.

d. Flood Velocities. Flood velocities for a levee beach would be identical to Alternative 1.

Table 69
Performance at Simulated Levee Breach Locations, Alternative 9B
2010 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR4	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR3	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9999	0.9982	0.9898	0.9781
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9997	0.9989	0.9978
LR1	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9999	0.9987	0.9951	0.9917
French Camp Slough											
FR1	0.0070	0.0679	0.1901	0.2963	0.9999	0.9997	0.9935	0.9328	0.7353	0.4974	0.3465
FL1	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
Stockton Diverting Canal											
SL1	0.0089	0.0859	0.2363	0.3619	0.9999	0.9670	0.9661	0.9606	0.9469	0.9262	0.9057
SL2	0.0166	0.1540	0.3945	0.5666	0.9999	0.9496	0.9177	0.8895	0.8542	0.8480	0.7616
Calaveras River											
CR2	0.0051	0.0497	0.1419	0.2251	0.9999	0.9916	0.9619	0.9320	0.8921	0.8349	0.7965
CL2	0.0145	0.1361	0.3552	0.5187	0.9999	0.9577	0.9533	0.9374	0.9110	0.8813	0.8536
Delta Front											
D3	0.0000	0.0003	0.0009	0.0014	0.9999	0.9999	0.9998	0.9996	0.9993	0.9990	0.9987
D4	0.0000	0.0003	0.0008	0.0014	0.9999	0.9999	0.9999	0.9999	0.9995	0.9978	0.9950
D5	0.0001	0.0014	0.0041	0.0068	0.9999	0.9999	0.9999	0.9994	0.9951	0.9799	0.9564
D-BS	0.0000	0.0005	0.0016	0.0026	0.9999	0.9999	0.9999	0.9998	0.9986	0.9939	0.9864

e. Flood Warning Time. Alternative 9B will result in a significant increase in warning time to the population within RD17, North Stockton, and Central Stockton because the probability of flooding from a geotechnical type failure (1-hour warning time) would be reduced and the warning time for overtopping type failures are significantly longer (24 to 36 hours). A description of flood warning time is provided in Alternative 1.

10.7 Potential Adverse Effects.

A potential adverse hydraulic effect would be induced flooding within the system. Induced flooding could result from a project increasing the depth, duration, or frequency of flooding. The potential for induced flooding was evaluated by comparing with-project and no action plans throughout the system.

a. Flood Depth.

Flood depths in the channel at index points throughout the study area are shown in plates 31 and 32. The index points are shown on Plates 21 and 22. Alternative 9A includes fix in place levees, levee raises along the Delta Front, upstream extension of the RD17 tieback levee and diversion of flood flows into old mormon channel. Flood depths in the channel at all index points would be the same as the no action condition except the Stockton Diverting Canal and Lower Calaveras River. Stages in the Stockton Diverting Canal and Lower Calaveras River would be lowered because of the upstream diversion to Old Mormon Channel. Flood depths in Smith Canal and Fourteenmile slough are not described by index points and would be reduced to 8 feet NAVD88 by the proposed closure structures. For magnitudes greater than 33% (1/3) ACE, stages in Old Mormon Channel would be increased due to the upstream diversion. Stages in Old Mormon Channel would be increased due to the upstream diversion.

It is unlikely that improvements along the delta front levees would increase water levels from delta sources. Improvement to the RD17 tieback levee was found to increase stages for events larger than 1% ACE for index points along the San Joaquin River, Old River, Middle River, and Paradise cut. It is possible that the increased delta front levee height could result in increased flood depths in the floodplain if a levee failure occurred along the Calaveras River or Stockton Diverting Canal. However, the area would already be flooded by the upstream levee breach.

Potential flood depths within the floodplain of the study area, assuming a levee failed, are shown on Plates 35 through 42 and are the same as the no-action condition. These maps represent a composite (overlay) of individual levee failure simulations for same ACE event magnitude. The extent of flooding would depend on the number and location of levee breaks to occur during an event.

b. Duration.

It is unlikely that improvements would change the duration of flooding throughout the system.

c. Frequency. The Delta Front raises and extension of French Camp slough levees upstream are unlikely to have hydraulic impacts that would impact flood frequency. However, improvements to the RD17 tieback levee would impact stages for events more rare than 1% ACE. The frequency of flood flows in the Old Mormon Channel would be increased due to the upstream diversion. The computed AEP and assurance values based on only the hydrology and hydraulic inputs are presented in Table 70. Changes to AEP and assurance values are presented in Table 71. A positive change in Annual Exceedance Probability (AEP) represents an increase in the long term average probability of a levee failing at the index point. A positive increase in AEP is an increase in the probability of being flooded. A positive change in assurance represents an increase in probability of passing a given hydrologic event frequency without failure. A positive change reflects a better chance of passing the event magnitude.

Table 70
2010 Performance at Selected Locations, Alternative 9B
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR4	0.0003	0.0034	0.0101	0.0168	0.9999	0.9999	0.9999	0.9995	0.9888	0.9331	0.8544
LR3	0.0000	0.0005	0.0016	0.0027	0.9999	0.9999	0.9999	0.9999	0.9982	0.9898	0.9781
LR2	0.0000	0.0001	0.0004	0.0006	0.9999	0.9999	0.9999	0.9999	0.9997	0.9989	0.9978
LR1	0.0000	0.0003	0.0010	0.0017	0.9999	0.9999	0.9999	0.9990	0.9987	0.9251	0.9917
French Camp Slough											
FR1	0.0070	0.0679	0.1901	0.2963	0.9999	0.9997	0.9935	0.9328	0.7353	0.4974	0.3465
FL1	0.0000	0.0004	0.0013	0.0022	0.9999	0.9999	0.9999	0.9999	0.9998	0.9996	0.9993
Stockton Diverting Canal											
SL1	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
SL2	0.0001	0.0007	0.0021	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Calaveras River											
CR2	0.0000	0.0002	0.0007	0.0012	0.9999	0.9999	0.9999	0.9999	0.9997	0.9985	0.9963
CL2	0.0001	0.0007	0.0020	0.0034	0.9999	0.9999	0.9999	0.9999	0.9999	0.9998	0.9998
Delta Front											
D3	0.0000	0.0003	0.0009	0.0014	0.9999	0.9999	0.9998	0.9996	0.9993	0.9990	0.9987
D4	0.0000	0.0003	0.0008	0.0014	0.9999	0.9999	0.9999	0.9999	0.9995	0.9978	0.9950
D5	0.0000	0.0005	0.0016	0.0026	0.9999	0.9999	0.9999	0.9998	0.9986	0.9939	0.9864
D-BS	0.0000	0.0004	0.0013	0.0021	0.9999	0.9999	0.9999	0.9999	0.9999	0.9997	0.9996
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0.0001	0.0010	0.0030	0.0050	0.9999	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995
Old R. at Clifton Court Ferry F-B95340	0.0002	0.0023	0.0067	0.0112	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995
Paradise Cut at I-5 F-PCI5	0.0024	0.0240	0.0703	0.1143	0.9999	0.9995	0.9995	0.9995	0.9986	0.8753	0.5404
Paradise Cut at Paradise Rd. F-PCPR	0.0038	0.0376	0.1085	0.1743	0.9999	0.9995	0.9995	0.9995	0.9993	0.6660	0.1373
SDWSC blw Burns Cutoff F-B95660	0.0001	0.0010	0.0030	0.0050	0.9999	0.9995	0.9995	0.9995	0.9995	0.9995	0.9995
Assurance estimates account for stage uncertainty, hydrologic uncertainty only. SDWSC- Stockton Deep Water Ship Channel											

Table 71
2010 Change in Performance at Selected Locations, Alternative 9B
Hydrologic and Hydraulic Parameters Only

Breach Location or Index Point	Change in Annual Exceedance Probability (Expected)	Change in Long Term Risk			Change in Flood Risk Management Assurance by Event Flood Frequency						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	-0.011	-0.1041	-0.2791	-0.417	0	0	0.0042	0.1187	0.4754	0.7416	0.817
LR4	0.0002	0.0027	0.0079	0.0131	0	0	0	-0.0003	-0.0087	-0.0527	-0.1149
LR3	0	0.0004	0.0013	0.0022	0	0	0	0	-0.0016	-0.0094	-0.0201
LR2	0	0	0	0	0	0	0	0	-1E-04	0.0003	0.0006
LR1	-0.0005	-0.0047	-0.0138	-0.0228	0	0	0	0	0.0149	0	0.1352
French Camp Slough											
FR1	-0.0039	-0.0357	-0.0895	-0.1248	0	0	0.0006	0.0301	0.1803	0.3098	0.3282
FL1	-0.0031	-0.0302	-0.0876	-0.1415	0	0	0	0.0035	0.0591	0.2728	0.5128
Stockton Diverting Canal											
SL1	0	0	0	0	0	0	0	0	0	0	0
SL2	0.0001	0.0005	0.0015	0.0024	0	0	0	0	1E-04	0.0007	0.0022
Calaveras River											
CR2	-0.0001	-0.0004	-0.001	-0.0016	0	0	0	1E-04	0.0013	0.0061	0.0134
CL2	0	0	0	0	0	0	0	0	0	0	0
Delta Front											
D3	-0.0029	-0.0285	-0.083	-0.1344	0	0.0017	0.0067	0.0182	0.0821	0.2366	0.3784
D4	-0.0001	-0.0004	-0.0012	-0.002	0	0	0	1E-04	0.0015	0.0069	0.0151
D5	-0.0001	-0.0009	-0.0025	-0.0042	0	0	0	0.0004	0.0035	0.014	0.03
D-BS	0	0	0	0	0	0	0	0	0	0	0
Outside Study Area											
Middle R. at Borden Hwy F-B95500	0	0	0	0	0	0	0	0	0	0	0
Old R. at Clifton Court Ferry F-B95340	-0.0008	0.0013	0.0037	0.0062	0	0	0	0.0001	0.0001	0.0001	0.0001
Paradise Cut at I-5 F-PCI5	0.001	0.01	0.0288	0.0461	0	0	0	0	-0.0009	-0.1199	-0.4375
Paradise Cut at Paradise Rd. F-PCPR	0.0021	0.0209	0.0593	0.0936	0	0	0	0	-0.0002	-0.3207	-0.7268
SDWSC blw Burns Cutoff F-B95660	-0.0001	-0.0006	-0.0019	-0.0031	0	0	0	0	0	0	0

Assurance estimates account for stage uncertainty, hydrologic uncertainty only.
SDWSC- Stockton Deep Water Ship Channel

10.8 Climate Change

The delta reaches of the study area are affected by changes in sea level. Performance was estimated for 2070 conditions using the hydraulic model results for 2070 sea level conditions at

downstream boundary conditions. The estimated performance for the 2070 condition is presented in Table 72. Composite floodplain maps were not developed for 2070 conditions.

Table 72
Performance at Simulated Levee Breach Locations, Alternative9B
2070 Conditions

Breach Location	Annual Exceedance Probability (Expected)	Long Term Risk			Flood Risk Management Assurance by Event Flood Frequency (Breach included in floodplain map if shaded)						
		10 Years	30 Years	50 Years	50% ACE	10% ACE	4% ACE	2% ACE	1% ACE	0.5% ACE	0.2% ACE
San Joaquin River											
LRTB	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9999	0.9998	0.9976	0.9934	0.9909
LR4	0.0000	0.0005	0.0015	0.0025	0.9999	0.9999	0.9999	0.9998	0.9976	0.9934	0.9909
LR3	0.0000	0.0002	0.0005	0.0008	0.9999	0.9999	0.9999	0.9999	0.9995	0.9983	0.9976
LR2	0.0000	0.0005	0.0014	0.0024	0.9999	0.9999	0.9999	0.9999	0.9996	0.9993	0.9991
LR1	0.0013	0.0128	0.0380	0.0626	0.9999	0.9999	0.9999	0.9958	0.9554	0.8735	0.8231
French Camp Slough											
FR1	0.0120	0.1137	0.3037	0.4530	0.9098	0.9938	0.9549	0.8333	0.5886	0.3619	0.2332
FL1	0.0000	0.0001	0.0003	0.0010	0.9999	0.9999	0.9999	0.9999	0.9997	0.9992	0.9987
Stockton Diverting Canal											
SL1	0.0089	0.0859	0.2363	0.3619	0.9999	0.9670	0.9661	0.9606	0.9469	0.9262	0.9057
SL2	0.0109	0.1036	0.2797	0.4211	0.9999	0.9700	0.9432	0.9194	0.8897	0.8480	0.8029
Calaveras River											
CR2	0.0051	0.0497	0.1419	0.2251	0.9999	0.9916	0.9619	0.9320	0.8920	0.8444	0.7965
CL2	0.0145	0.1361	0.3552	0.5187	0.9999	0.9577	0.9533	0.9374	0.9110	0.8813	0.8536
Delta Front											
D3	0.0010	0.0099	0.0294	0.0485	0.9999	0.9967	0.9917	0.9873	0.9824	0.9777	0.9742
D4	0.0001	0.0006	0.0017	0.0029	0.9999	0.9999	0.9999	0.9997	0.9983	0.9934	0.9861
D5	0.0002	0.0019	0.0058	0.0096	0.9999	0.9999	0.9997	0.9987	0.9932	0.9655	0.9482
D-BS	0.0000	0.0004	0.0012	0.0020	0.9999	0.9999	0.9999	0.9998	0.9997	0.9996	0.9996

10.9 California State Urban Levee Design Criteria

The hydraulic performance of alternative 9B relative to the ULDC requirements for 2070 conditions is provided in Table 73.

Table 73
Alternative 9B Performance Relative to DWR Urban Levee Design Criteria, 2070 Conditions

Flood Source	Location	Economic Impact Area	Top of Levee Elevation FT-NAVD88	1.3% ACE Wind Wave Run up (FT)	Minimum ULDC Required Freeboard	Mean 0.5% Water Surface (FT-NAVD88	Freeboard (feet)	H&H Assurance
San Joaquin River	LRTB	RD17	38.2	6.3	6.3	31.9	6.3	99%
	LR4	RD17	34.9	2.4	3.0	31.9	3.0	99%
	LR3	RD17	31.0	2.4	3.0	27.0	4.0	99%
	LR2	RD17	27.8	2.4	3.0	22.7	5.1	99%
	LR1	RD17	25.0	2.4	3.0	20.8	4.2	87%
French Camp Slough	FR1	CS-02	21.8	<3.0	3.0	16.8	5.0	36%
	FL1	RD17	21.4	<3.0	3.0	16.8	4.6	99%
Stockton Diverting Canal	SL1	CS-01,CS03	39.2	<3.0	3.0	29.8	9.4	99%
	SL2	CS-01,CS-02,CS-03	44.6	<3.0	3.0	39.3	5.3	99%
Calaveras River	CR2	NS-04, NS-03	29.7	<3.0	3.0	25.1	4.6	99%
	Cl2	CS-01,CS-02,CS-03	31.4	<3.0	3.0	26.0	5.4	99%
Delta Front	D3	NS-02	14.9	<3.0	3.0	11.9	3.0	98%
	D4	CS-01	18.8	<3.0	3.0	15.0	3.8	98%
	D5	NS-03	17.5	<3.0	3.0	14.4	3.1	94%
	D-BS	NS-03	18.0	<3.0	3.0	13.6	4.4	99%

H&H assurance only includes hydrology and hydraulics. Wind runup and setup, and geotechnical factors are not included.
LRTB assurance based on LR4 index point

11.0 SUMMARY

This report describes hydraulic, sedimentation, and operations and maintenance analyses performed for the final alternatives of the Lower San Joaquin Interim Feasibility Study. Analyses were performed for without-project and six project alternative conditions.

The study is focused on Lower San Joaquin Interim Feasibility Study area. Composite floodplain delineations are provided for 50% (1/2) ACE, 10% (1/10) ACE, 4% (1/25) ACE, 2% (1/50) ACE, 1% (1/100) ACE, 0.5% (1/200) ACE, and 0.2% (1/500) ACE events for the existing and alternative conditions.

12.0 REFERENCES

1. Chow, 1959, Open Channel Hydraulics, McGraw Hill, 1959.
2. CNRFC, 2013, Weather Forecast Office (WFO) Hydrologic Products, California Nevada River Forecast Center, http://www.cnrfc.noaa.gov/wfo_hydro.php, 27 Feb, 2013
3. DAS, 2011. Potential Increase in Flood in California's Sierra Nevada under Future Climate Projections, June 2011.
4. DWR, 2010. State Plan of Flood Control Descriptive Document, California Department of Water Resources, November 2010.
5. DWR,2012. Urban Levee Design Criteria, California Department of Water Resources, May 2012.
6. FEMA, 2011, Floodplain Management Requirements, A Study Guide and Desk Reference for Local Officials http://www.fema.gov/plan/prevent/floodplain/fm_sg.shtm.
7. FEMA, 2012, Levee Certification vs. Accreditation, Federal Emergancy Management Agency, October 2012.
8. FEMA, 2009, Flood Insurance Study, San Joaquin County California and incorporated Areas, Study Number 06077CV001A, Federal Emergency Management Agency, 16 October 2009.
9. FLO-2D Software Inc. Flo-2D Flood Routing Model, Version 2006.01, 2004
10. HEC, 2008. Hydrologic Engineering Center, HEC-RAS River Analysis Program Version 4.0.0, March, 2008.
11. HEC, 2010. Hydrologic Engineering Center, HEC-FDA Flood Damage Assessment Program Version 1.2.5a, November, 2010.
12. PBI, 2013A. Technical Memorandum, San Joaquin River Main Stem HEC-RAS Model Setup, September 2013.
13. PBI, 2013B. Technical Memorandum, Review and Update of the CVFED Calaveras River HEC-RAS Model, September 2013.
14. PBI, 2013C. Technical Memorandum, Mormon Channel Bypass Cost Estimate, September 2013.
15. SJAFCA, 2012. Lower San Joaquin River Feasibility Study Hydrology Appendix, July 2012.
16. SJAFCA, 2013. Lower San Joaquin and Delta South Regional Flood Management Plan, November 2013.
17. USACE, 1952. Operation and Maintenance Manual for Duck Creek Diversion, A Unit of Farmington Reservoir Project, December 1952.
18. USACE, 1955. San Joaquin River Levees General Design, December 1955.
19. USACE, 1974. Civil Works Project Maps, River and Harbor, Flood Control, and California Debris Commission, U.S. Army Engineer District, Sacramento, Corps of Engineers.

20. USACE, 1975. Southwest Stream Group, December 1975.

21. USACE, 1989. Expected Annual Flood Damage Computation, Users Manual, CPD-30 US Army Corps of Engineers Hydrologic Engineering Center, March 1989.

22. USACE 1993. San Joaquin River Mainstem, California, January 1993

23. USACE, 1996. Engineering and Design, Risk-Based Analysis for Flood Damage Reduction Studies, EM110-2-1619, 1 August 1996

24. USACE, 1999. Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies, May 1999.

25. USACE, 2001. Mormon Channel 1135 Restoration, Stockton, California, 90% Final Alternatives Report, Prepared by HDR Engineering Inc for San Joaquin Area Flood Control Agency and USACE, August 2001.

26. USACE, 2002. Sacramento San Joaquin Comprehensive Study. United States Army Corps of Engineers, December 2002.

27. USACE, 2008. Memorandum, Clarification on the Policy and Procedural Guidance for the Approval of Modifications and Alterations of Corps of Engineers Projects, Director of Civil Works, 17 November 2008.

28. USACE, 2009, Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums, ER 1110-2-8160, 1 March 2009

29. USACE, 2014. Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures, April 2014.

30. USACE, 2011, Memorandum, Corps of Engineer Civil Works Cost Definitions and Applicability, Director of Civil Works, 25 August 2011

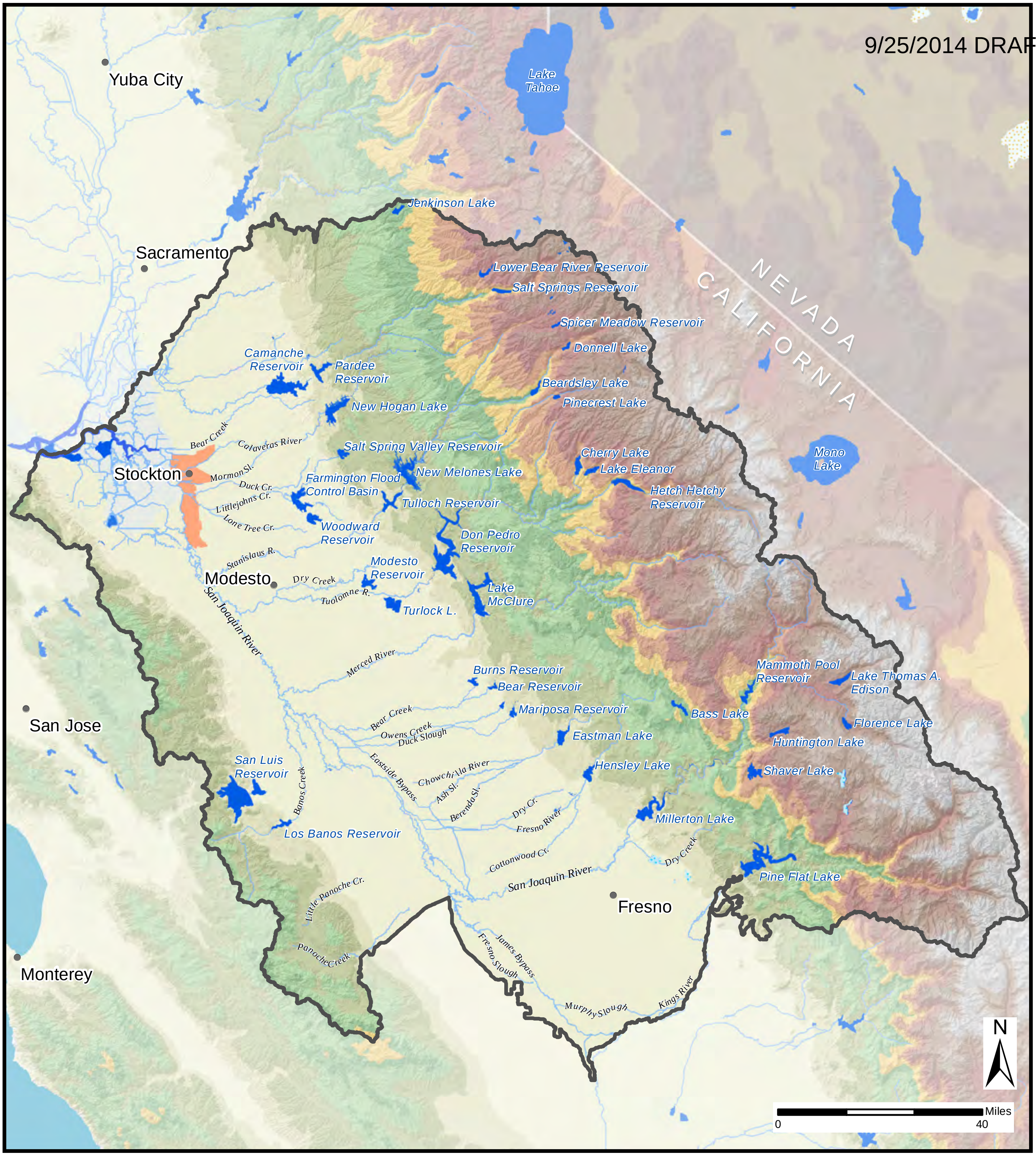
31. USACE, 2012 Engineering and Design: USACE Process for the National Flood Insurance Program (NFIP) Levee System Evaluation, EC 1110-2-6067, 31 August 2010

32. USACE, 2014, Engineering and Design: Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures, ETL 1110-2-583, 30 April 2014

33. U.S. Census Bureau, 2010. 2010 Census Block-Derived Housing and Population Density, Tiger/Line Shapefile: census2010den_11_1.shp, U.S. Department of Commerce, U.S. Census Bureau, Geography Division, 11 April 2011, Retrieved from http://www.census.gov/cgi-bin/geo/shapefiles2010/main

34. USDOT, 2012. United States Department of Transportation Federal Highway Administration, Hydraulic Design of Highway Culverts, Third Edition, Hydraulic Design Series Number 5, Publication No. FHWA-HIF-12-026, April 2012.

35. Jonkman, 2008. Methods for the estimation of loss of life due to floods: a literature review and a proposal for a new method.



9/25/2014 DRAFT

Legend

- ● Cities
- LSJ Watershed Boundary
- Rivers or Streams
- Lake or Reservoir
- 🟧 Study Extent

NOTE: Analysis Limited to Study Extent.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

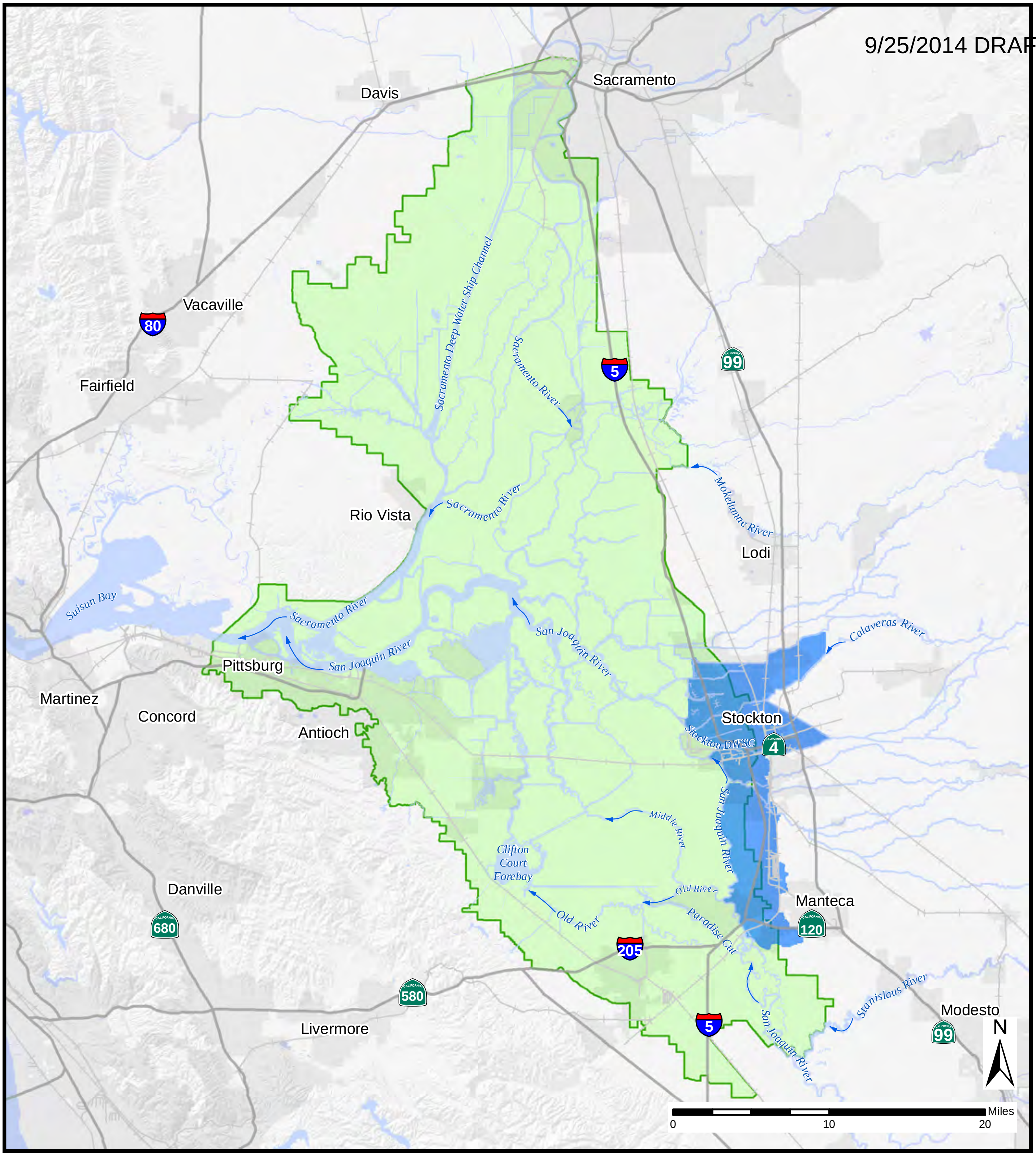
SAN JOAQUIN WATERSHED BOUNDARY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 1



Legend

- Highway
- Railroads
- 🟩 Delta Legal Boundary
- 🟦 Study Extent

Note:
Downstream of Rio Vista, the Sacramento River is maintained as part of the Sacramento Deep Water Ship Channel
Downstream of Stockton, the San Joaquin River is maintained as part of the Stockton Deep Water Ship Channel.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

SACRAMENTO-SAN JOAQUIN DELTA

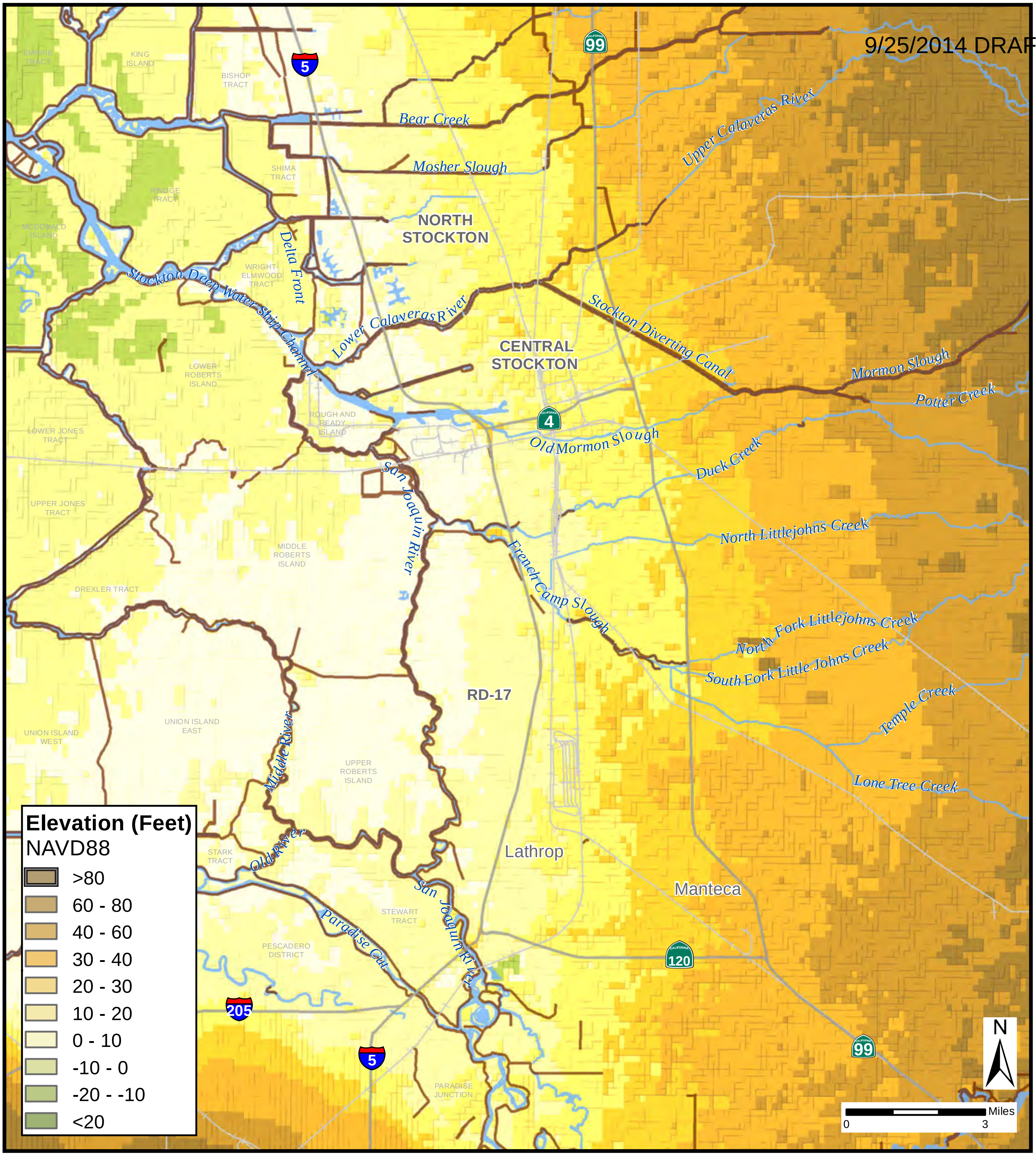
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 2

9/25/2014 DRAFT



Legend

Highway

Railroads

Levees (Fed/Non-Fed)

NOTE: Elevation Based on IFSAR 2.0 (feet)

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

REGIONAL TOPOGRAPHY

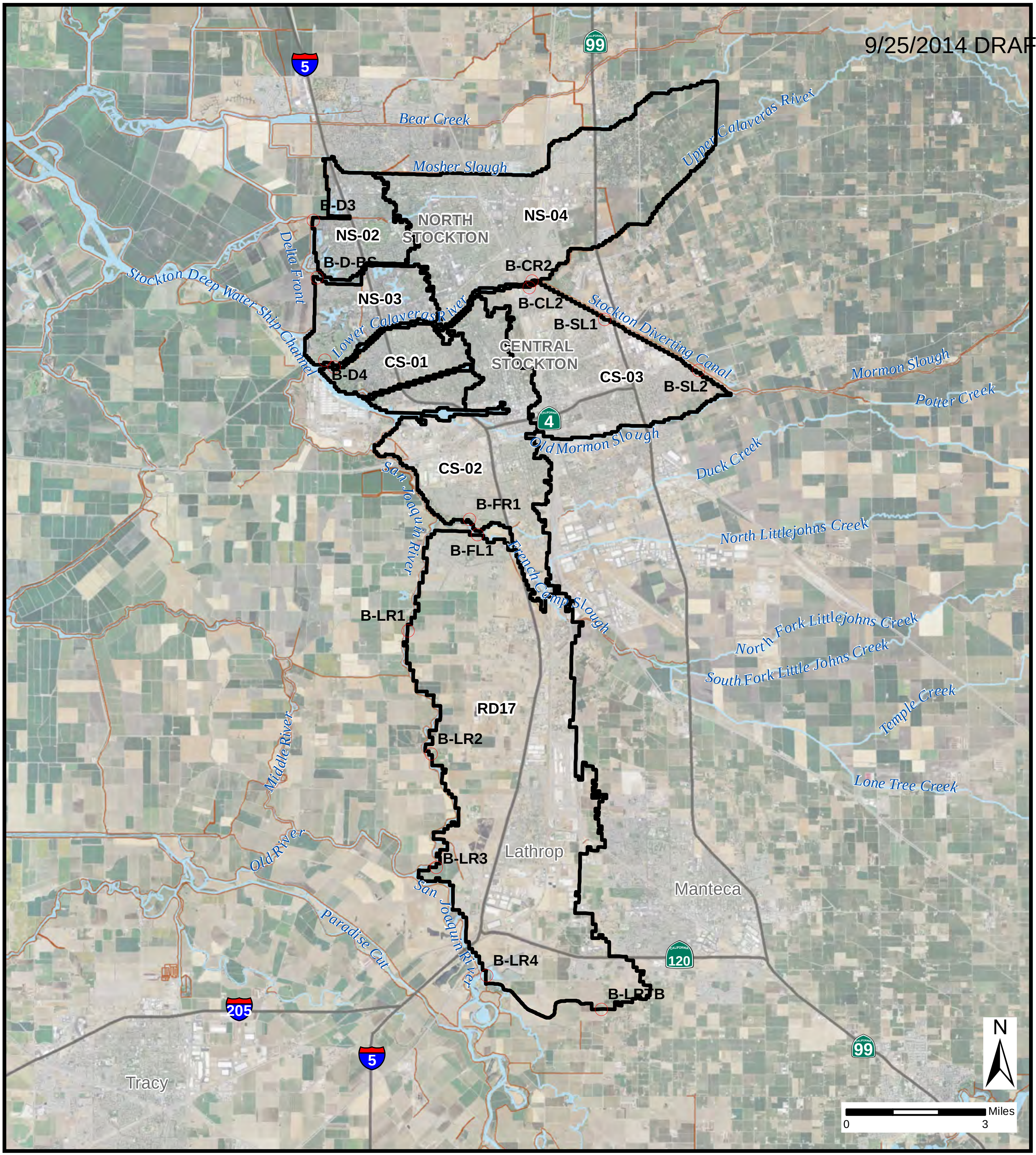
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 3

9/25/2014 DRAFT



Legend

○ Economic Index Point

Highway

Railroads

Levees (Fed/Non-Fed)

NOTE: Economic Impact Areas Limited to Study Extent.
Aerial Imagery: NAIP 2012, 1 m.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

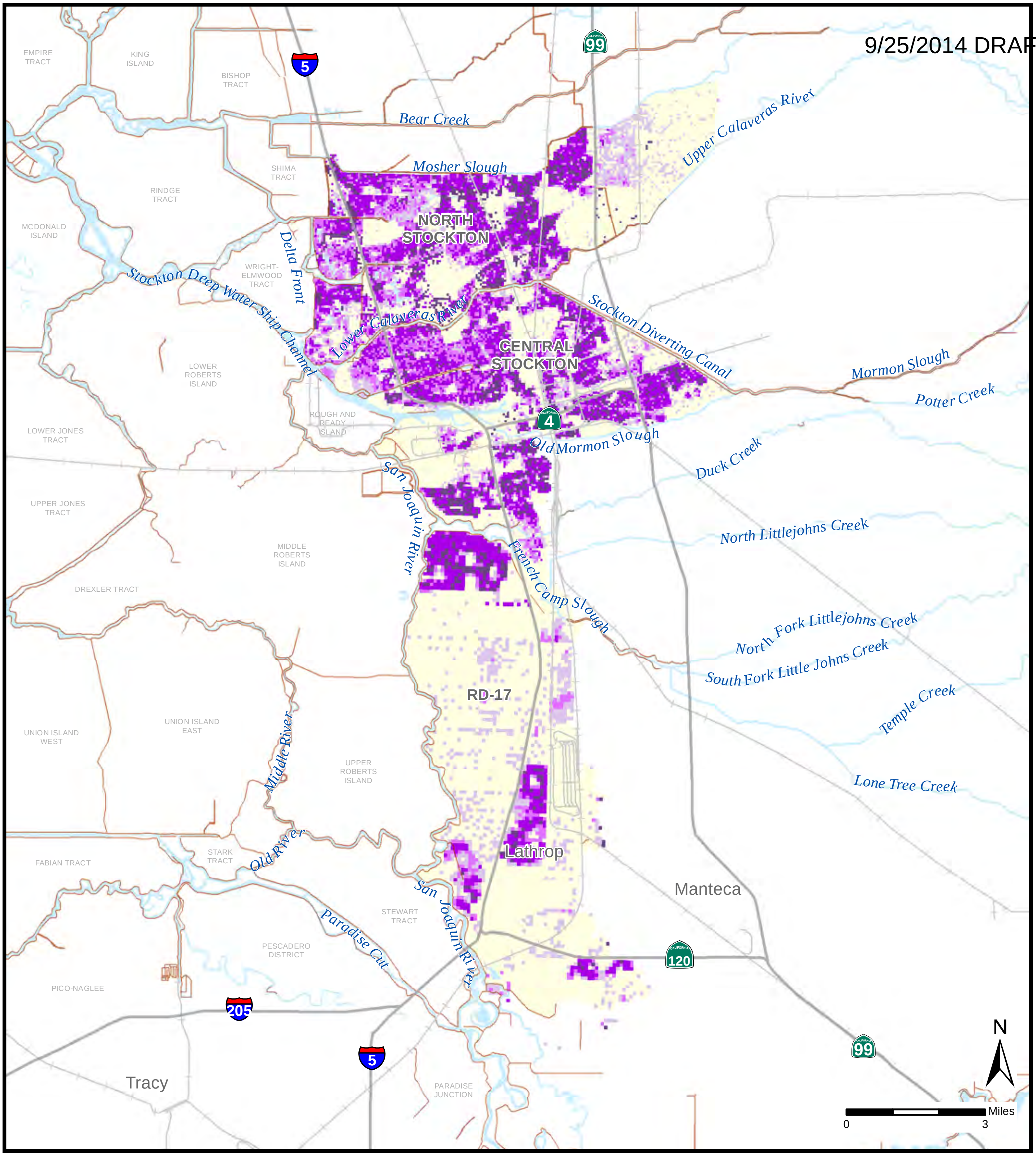
ECONOMIC IMPACT AREAS AND AERIAL IMAGERY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 4



9/25/2014 DRAFT

Legend

- Highway
- Railroads
- Levees
- Study Extent

Population Per Acre

- ▨ Sparse (0 - 5)
- ▨ Low Density (5 - 10)
- ▨ Medium Density (10 - 20)
- ▨ High Density (Greater than 20)

NOTE: Population Only Shown within Study Extent

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

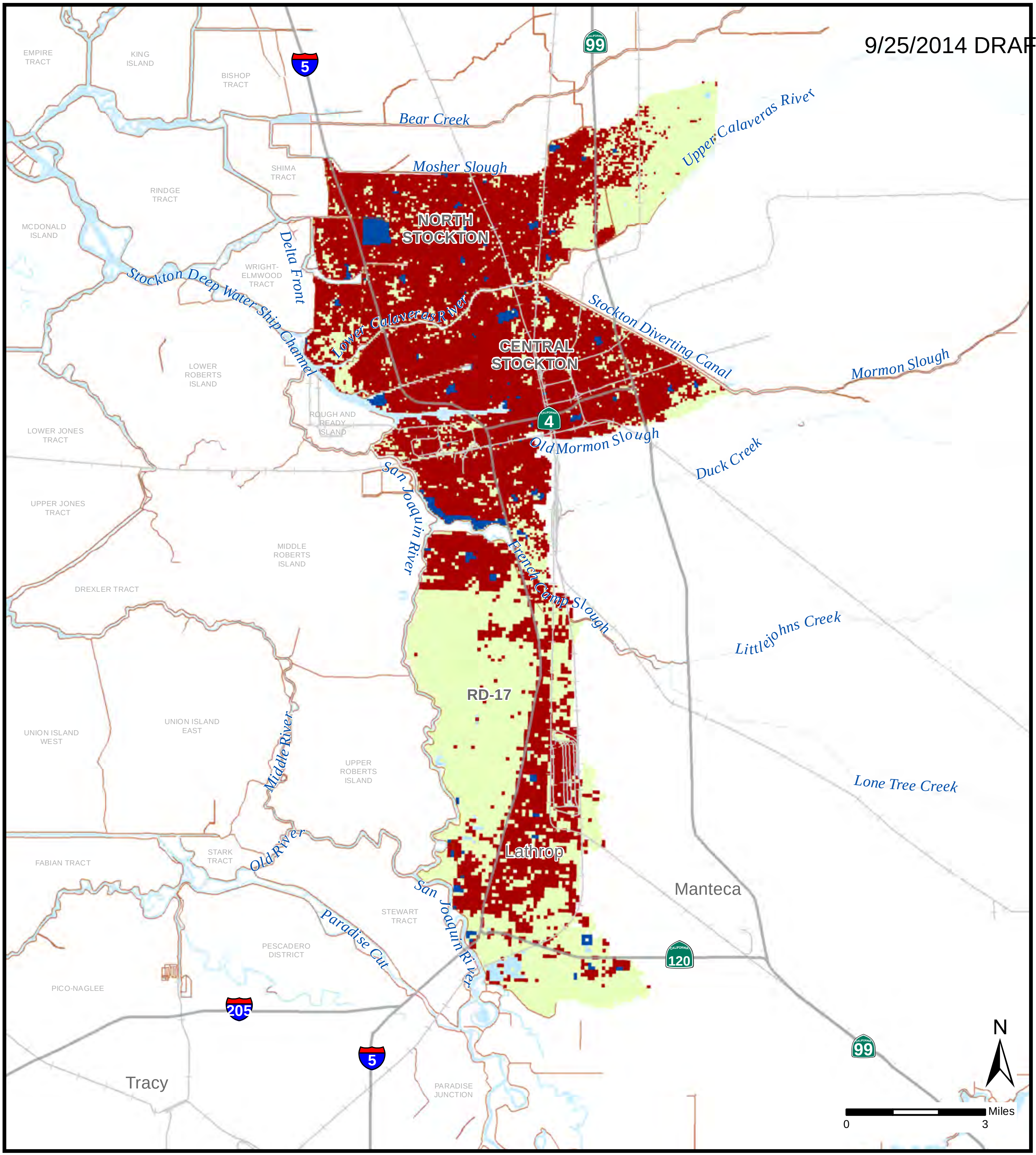
POPULATION
STUDY AREA DENSITY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 5



9/25/2014 DRAFT

Legend

Highway

Railroads

Levees (Fed/Non-Fed)

LSJ Study Extent

Landuse

🟦 Protected Area

🟩 Agriculture

🟥 Developed Land

🟦 Open Water

NOTE: Land Use Limited to Study Extent.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

EXISTING LANDUSE

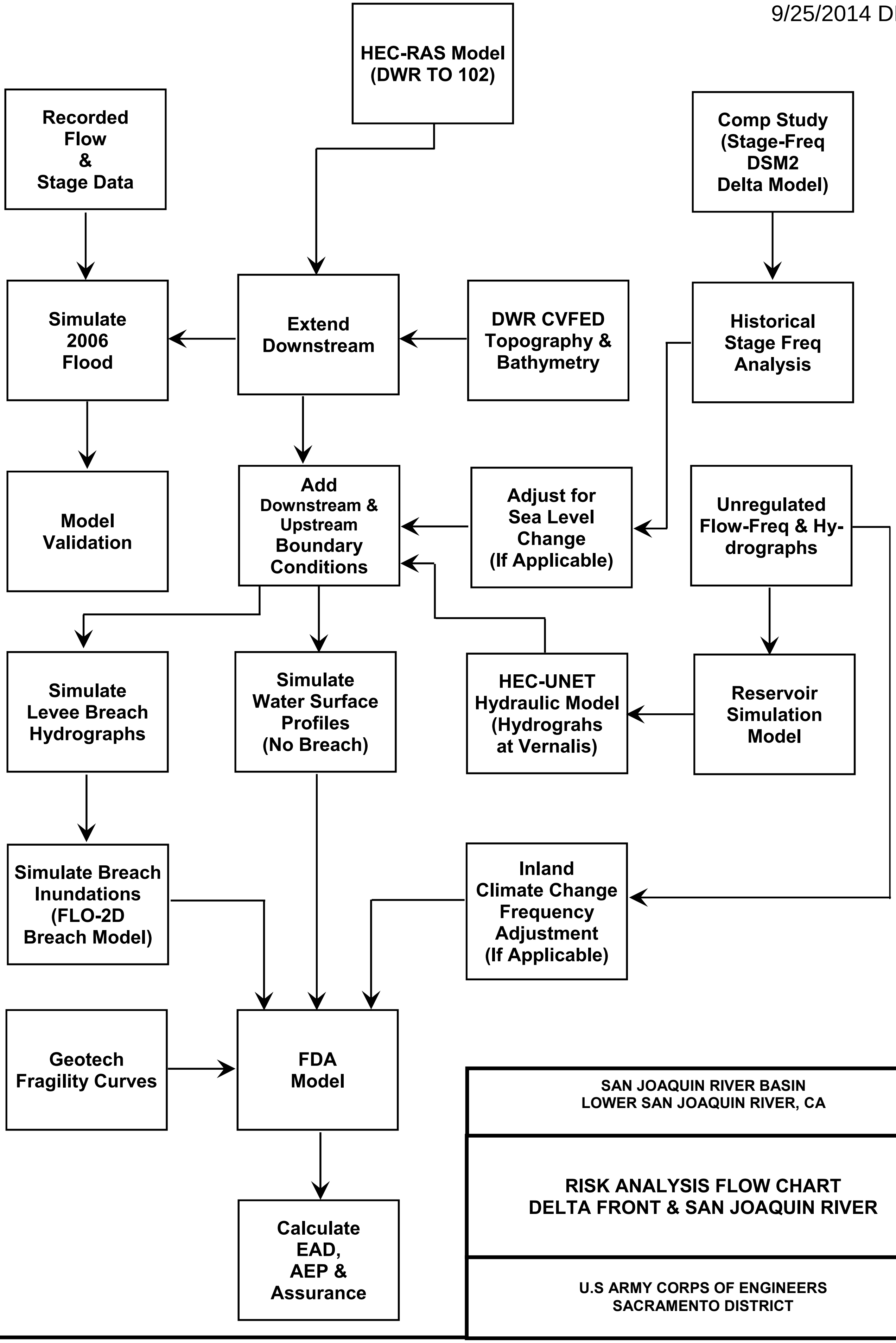
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 6

9/25/2014 DRAFT



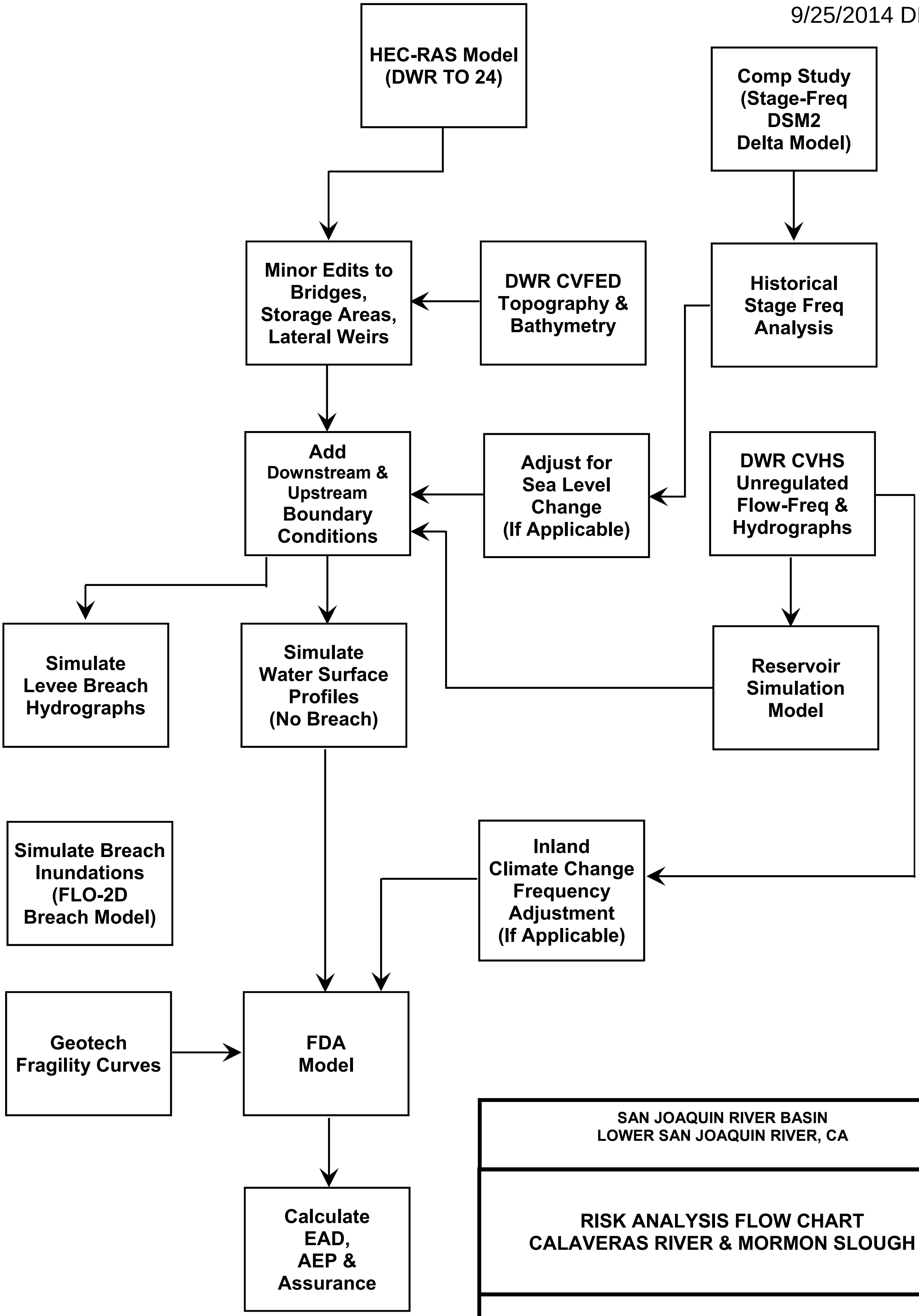
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA

RISK ANALYSIS FLOW CHART
DELTA FRONT & SAN JOAQUIN RIVER

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014

9/25/2014 DRAFT

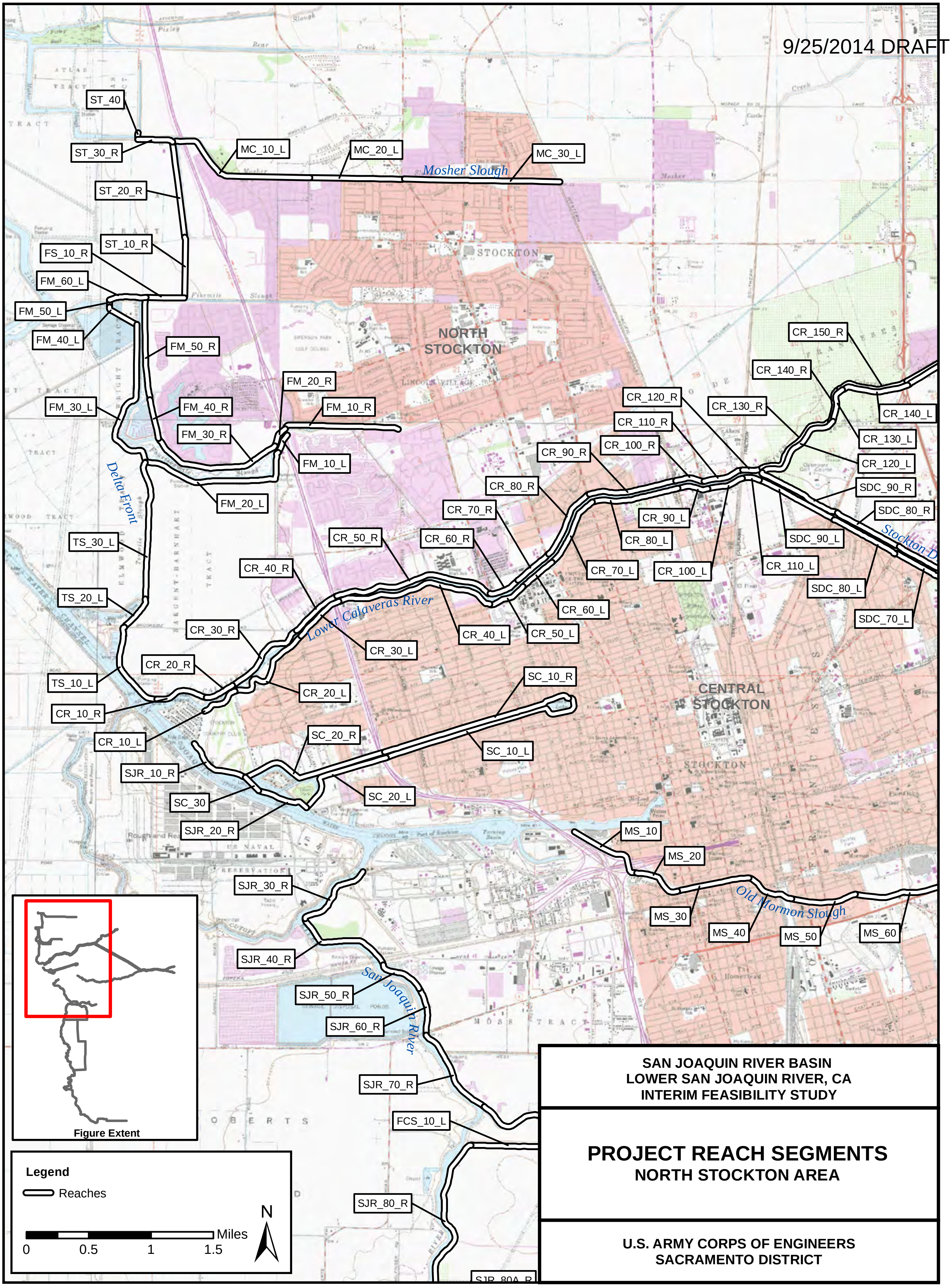


SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA

RISK ANALYSIS FLOW CHART
CALAVERAS RIVER & MORMON SLOUGH

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

9/25/2014 DRAFT



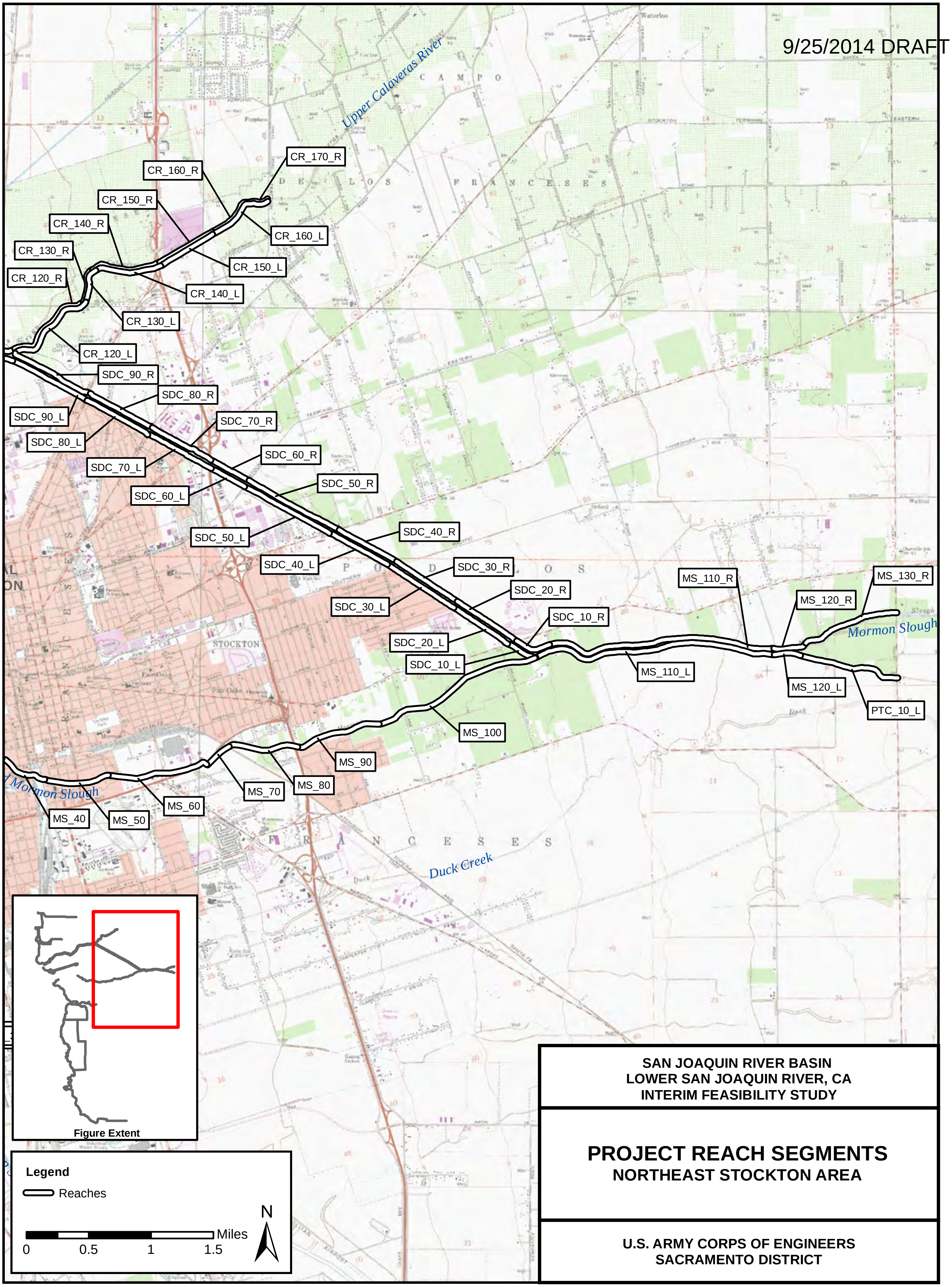
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

PROJECT REACH SEGMENTS
NORTH STOCKTON AREA

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014

9/25/2014 DRAFT



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

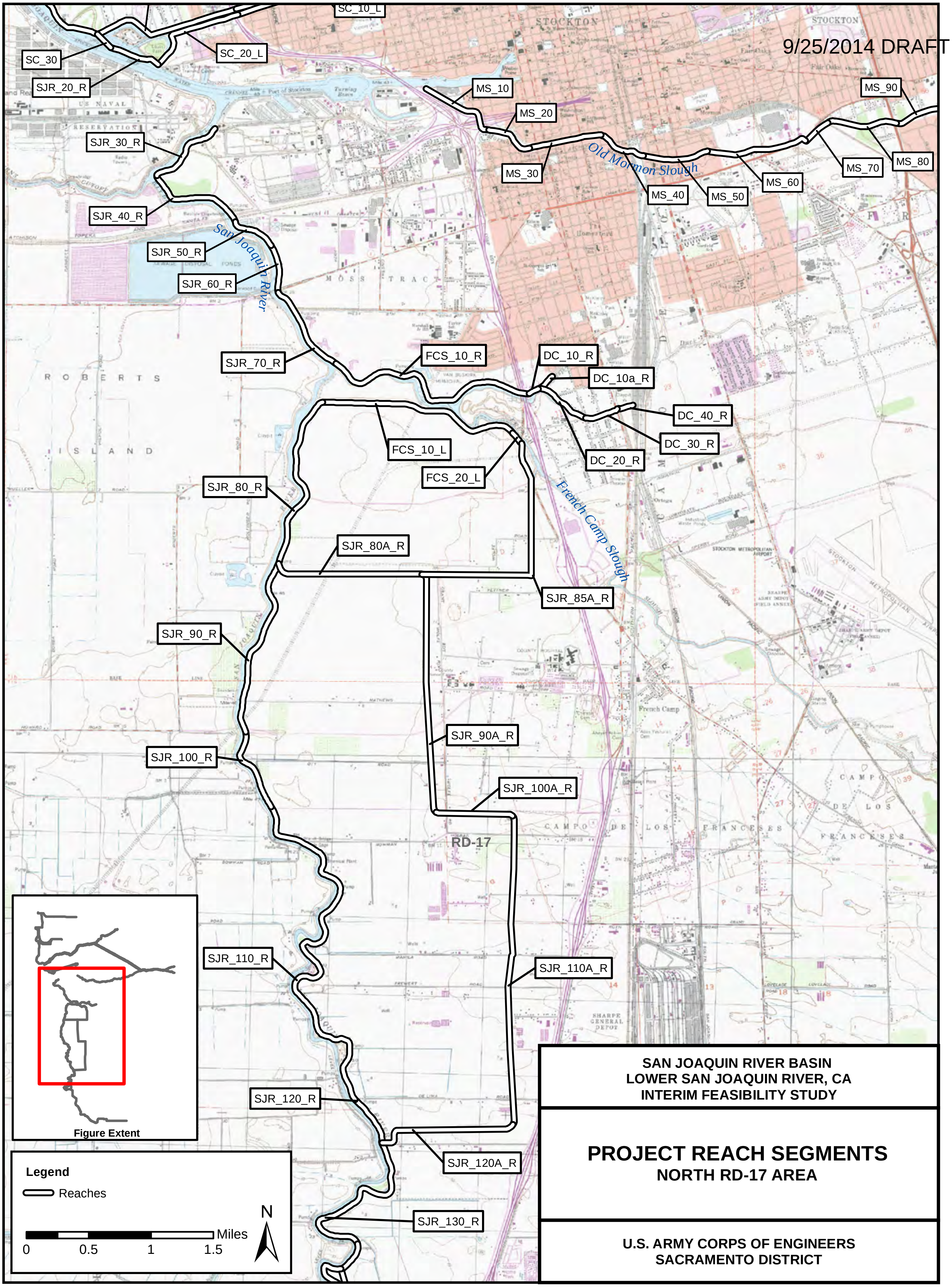
PROJECT REACH SEGMENTS
NORTHEAST STOCKTON AREA

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014

PLATE 9B

9/25/2014 DRAFT



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

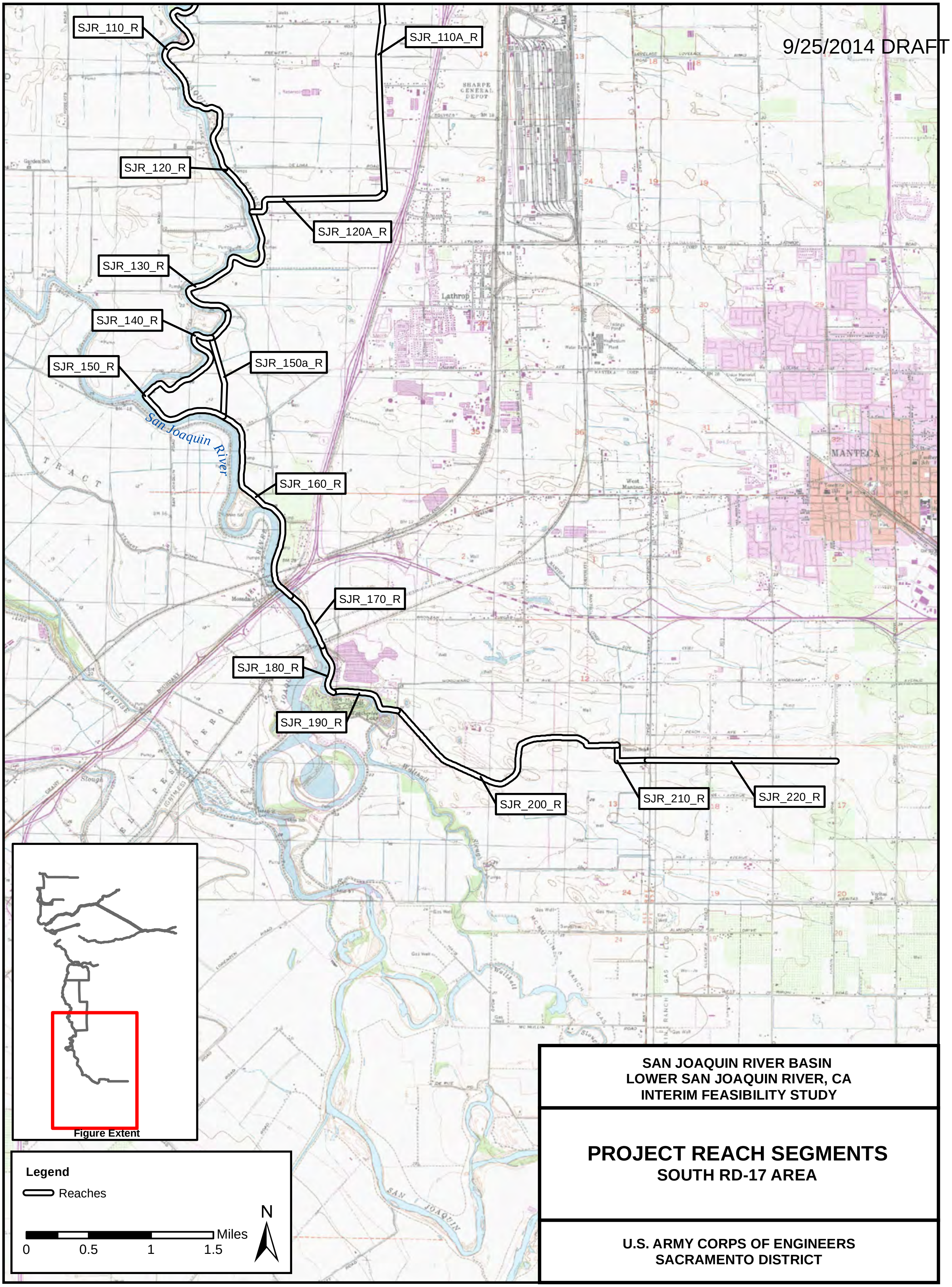
PROJECT REACH SEGMENTS
NORTH RD-17 AREA

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014

PLATE 9C

9/25/2014 DRAFT



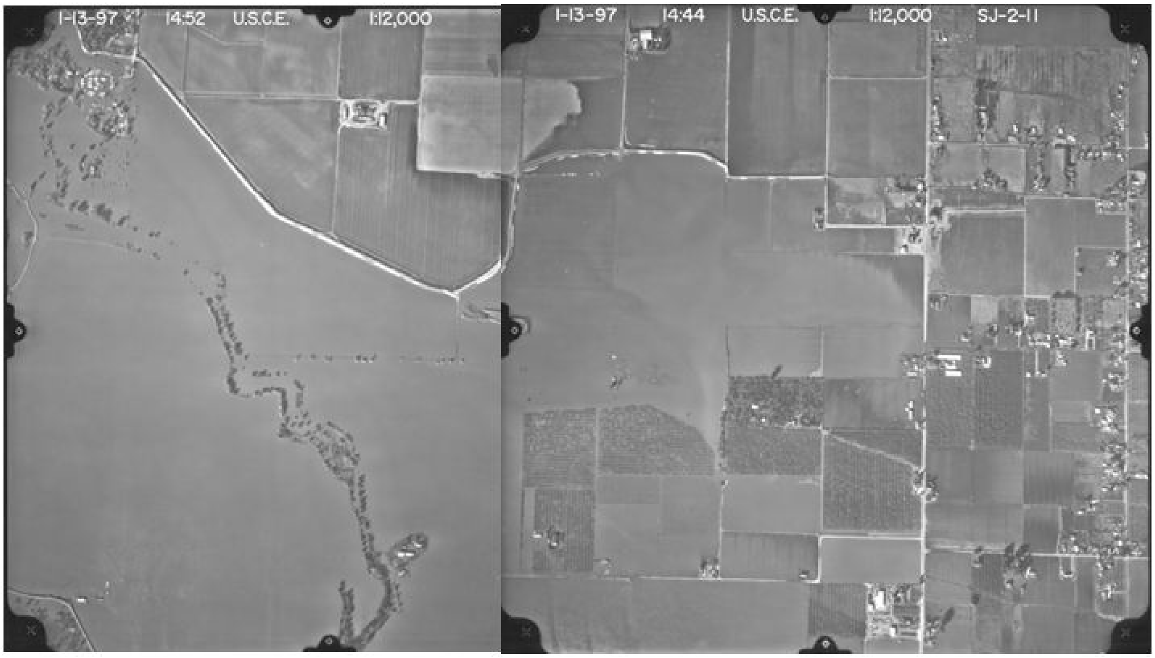
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

PROJECT REACH SEGMENTS
SOUTH RD-17 AREA

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014

PLATE 9D



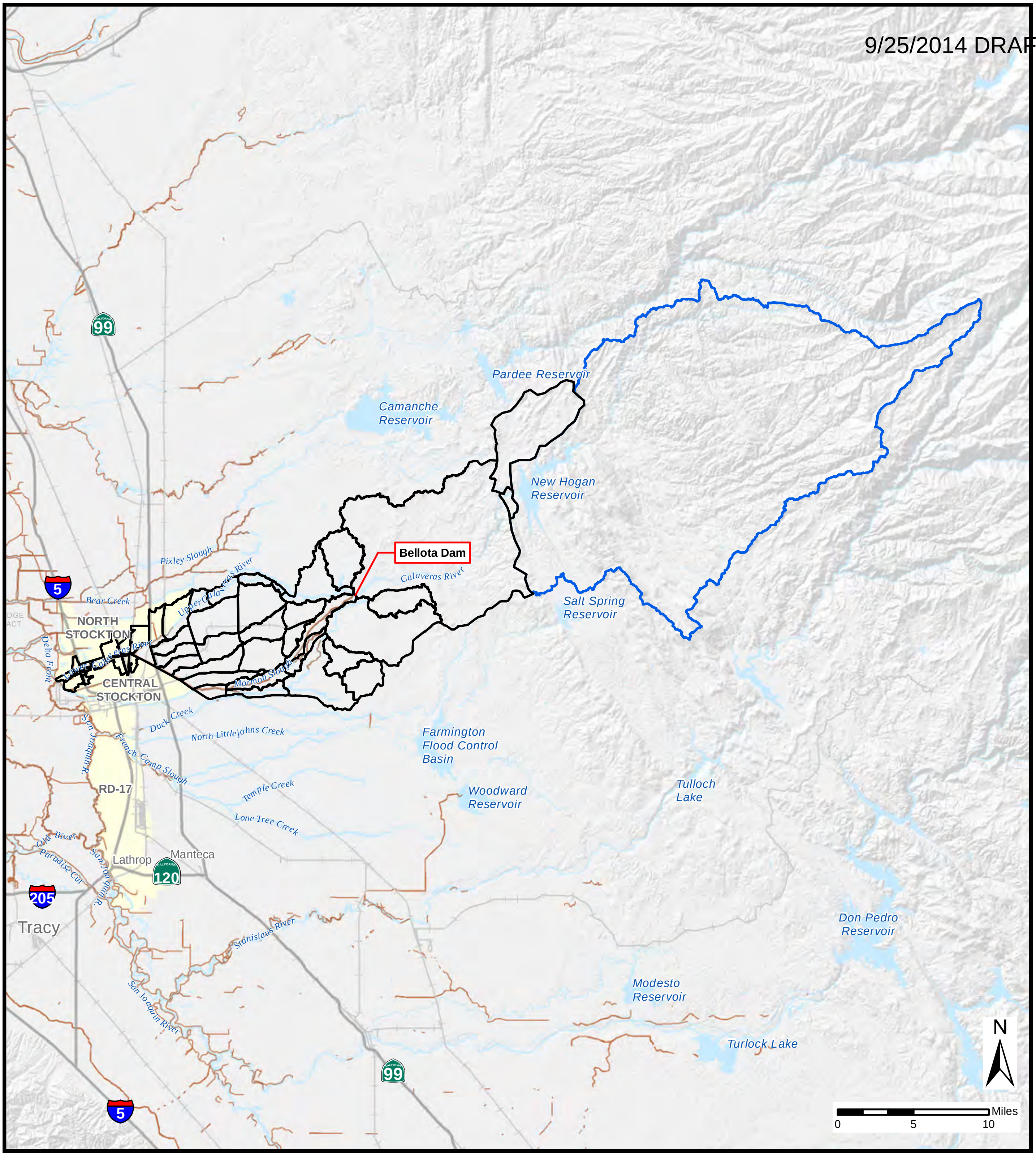
Tieback Levee at Upstream end of RD17, 13 January 1997

Floodwaters within Wetherbee Lake (Walthal Slough)
upstream of RD17 Tieback Levee Looking East

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

1997 FLOOD
WETHERBEE LAKE AND RD17 TIEBACK LEVEE

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT



Legend

- Calaveras Subbasins
- New Hogan Reservoir Wathershed Boundary
- Highway
- Railroads
- Levees (Fed/Non-Fed)
- Study Extent

NOTE: Analysis Limited to Study Extent.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

CALAVERAS RIVER WATERSHED BOUNDARY

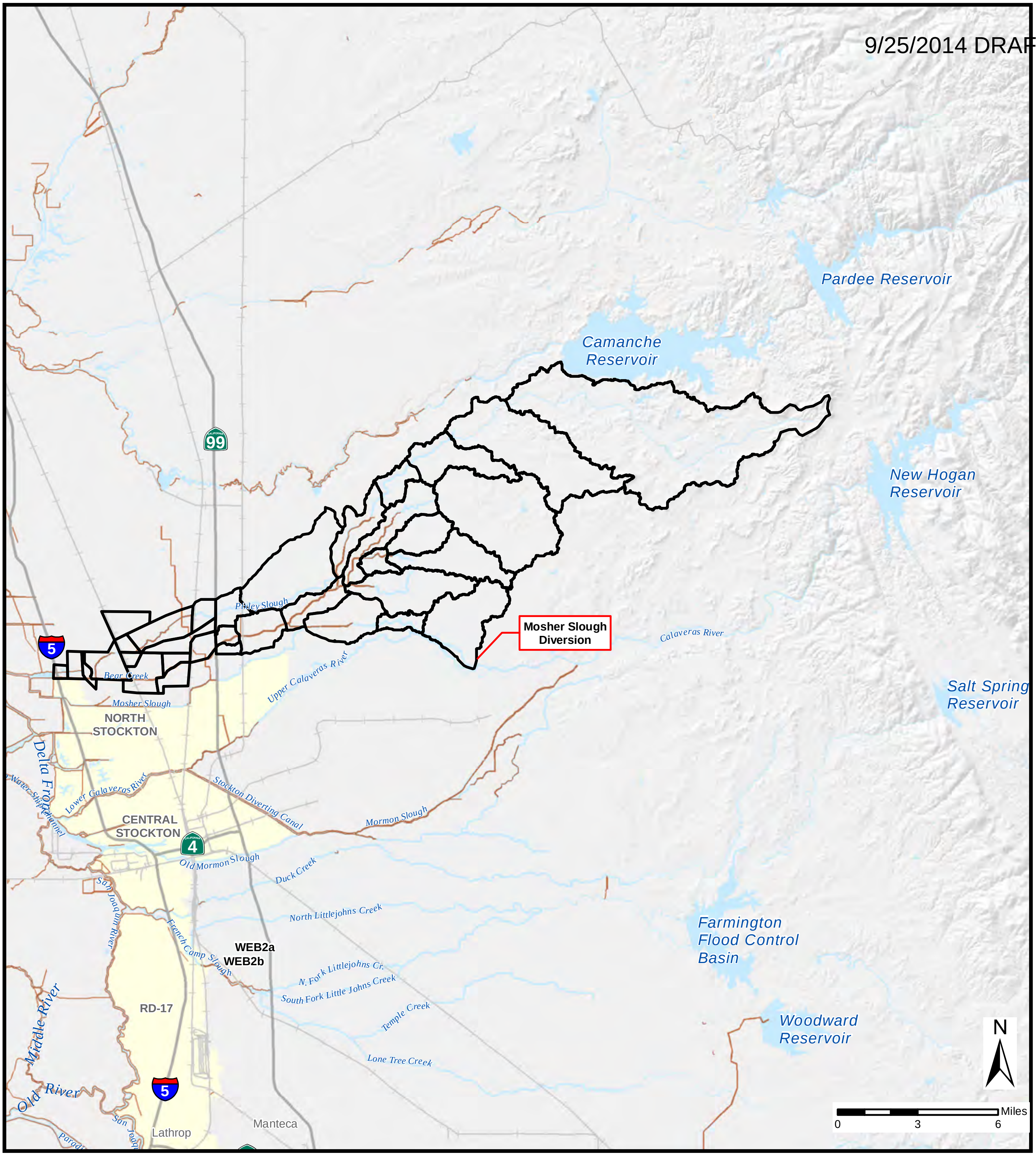
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 11

9/25/2014 DRAFT



Legend

- Highway
- Railroads
- Levees (Fed/Non-Fed)
- Bear Creek Subbasins
- Study Extent

NOTE: Analysis Limited to Study Extent.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BEAR CREEK
WATERSHED BOUNDARY

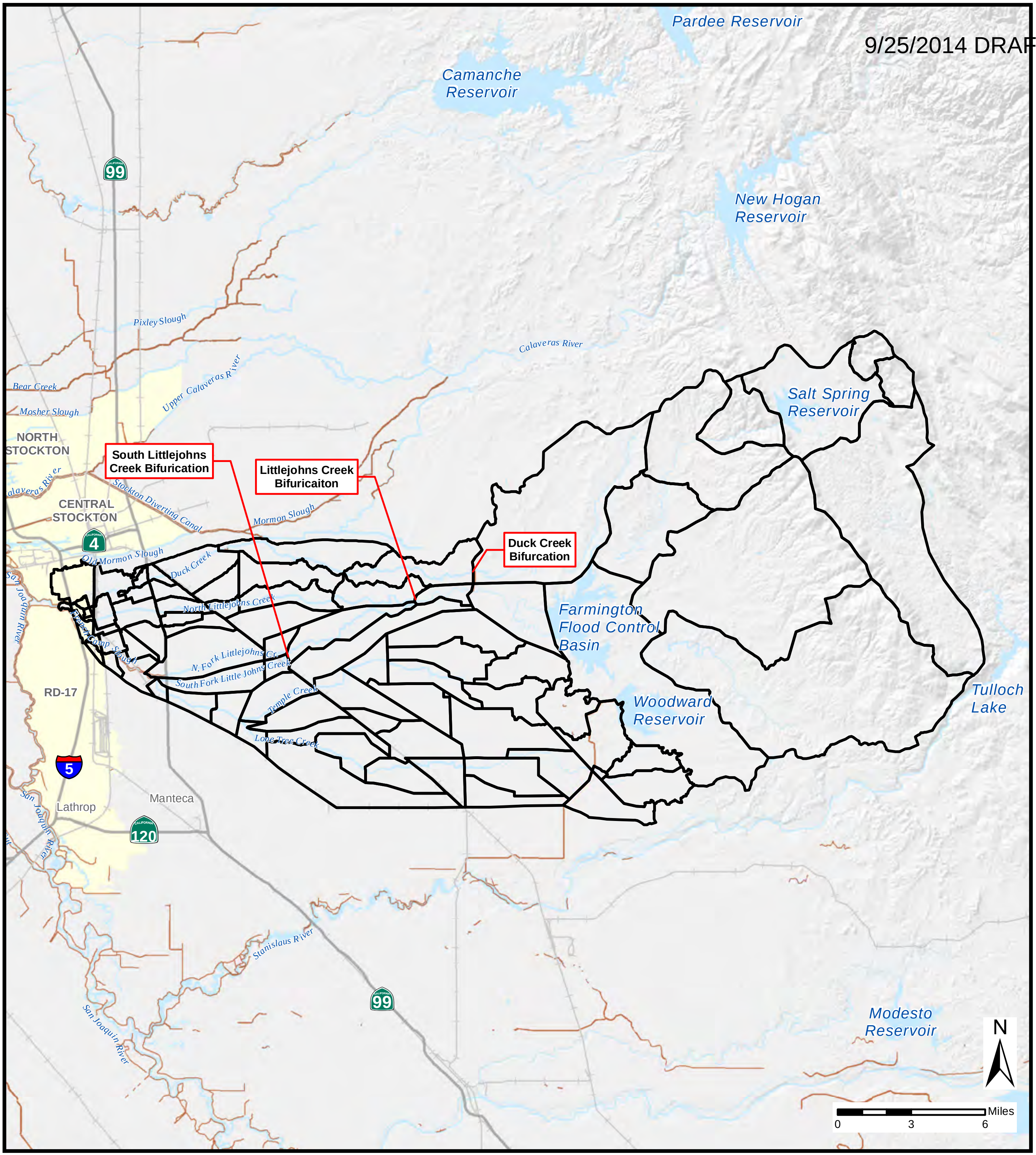
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 12

9/25/2014 DRAFT



Legend

- Highway
- Railroads
- Levees (Fed/Non-Fed)
- FCS Subbasins
- Study Extent

NOTE: Analysis Limited to Study Extent.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

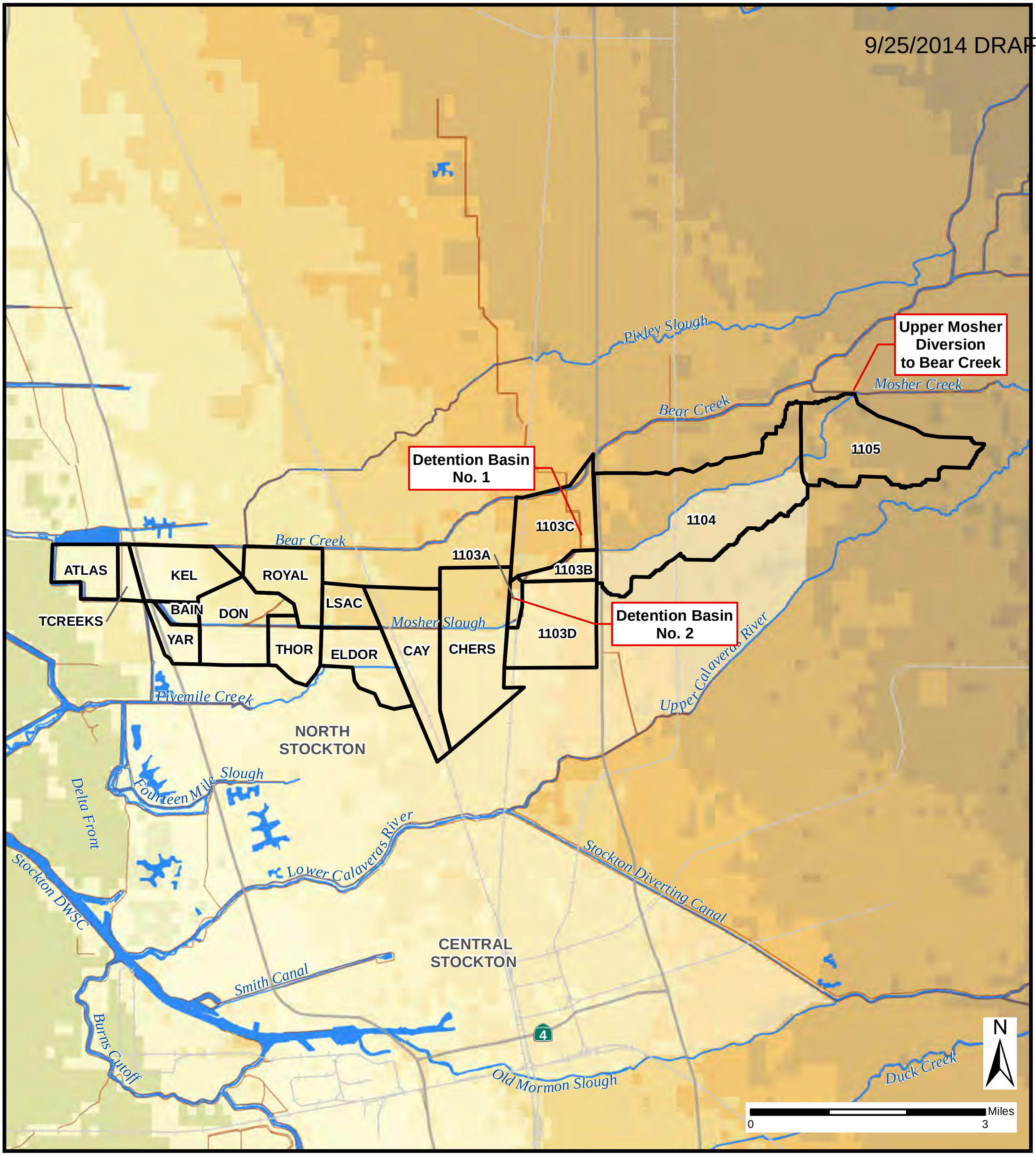
FRENCH CAMP SLOUGH WATERSHED BOUNDARY

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 13



Legend

- Highway
- Railroads
- Levees (Fed/Non-Fed)
- Mosher Subbasins
- Study Extent

NOTE: Elevation Based on IFSAR 2.0 (NAVD 88, feet)

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

MOSHER SLOUGH WATERSHED BOUNDARY

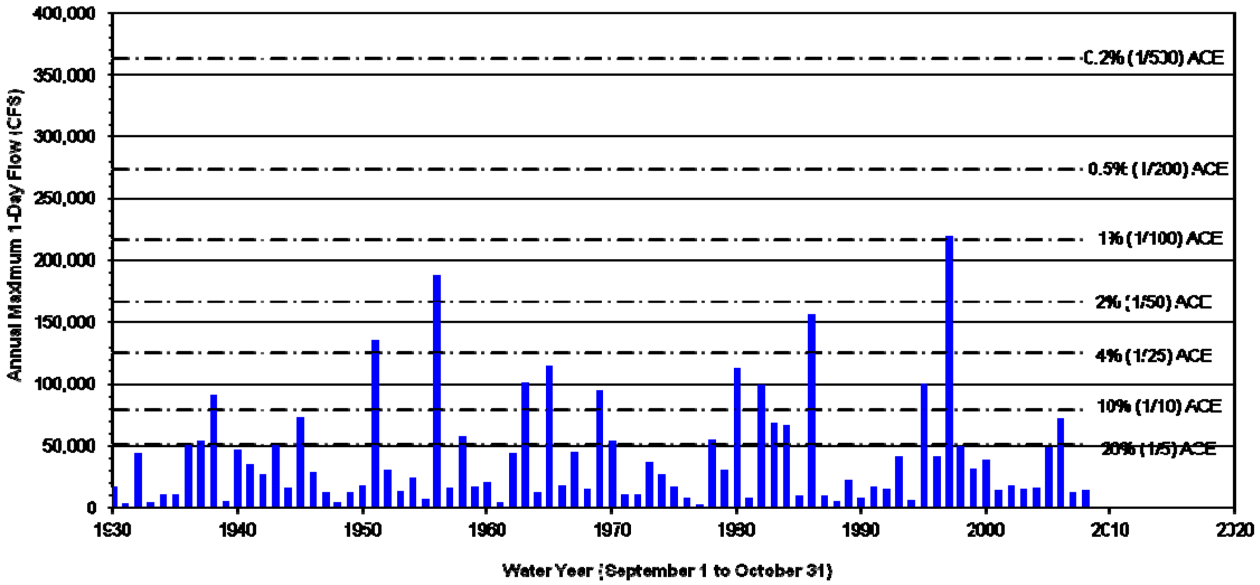
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

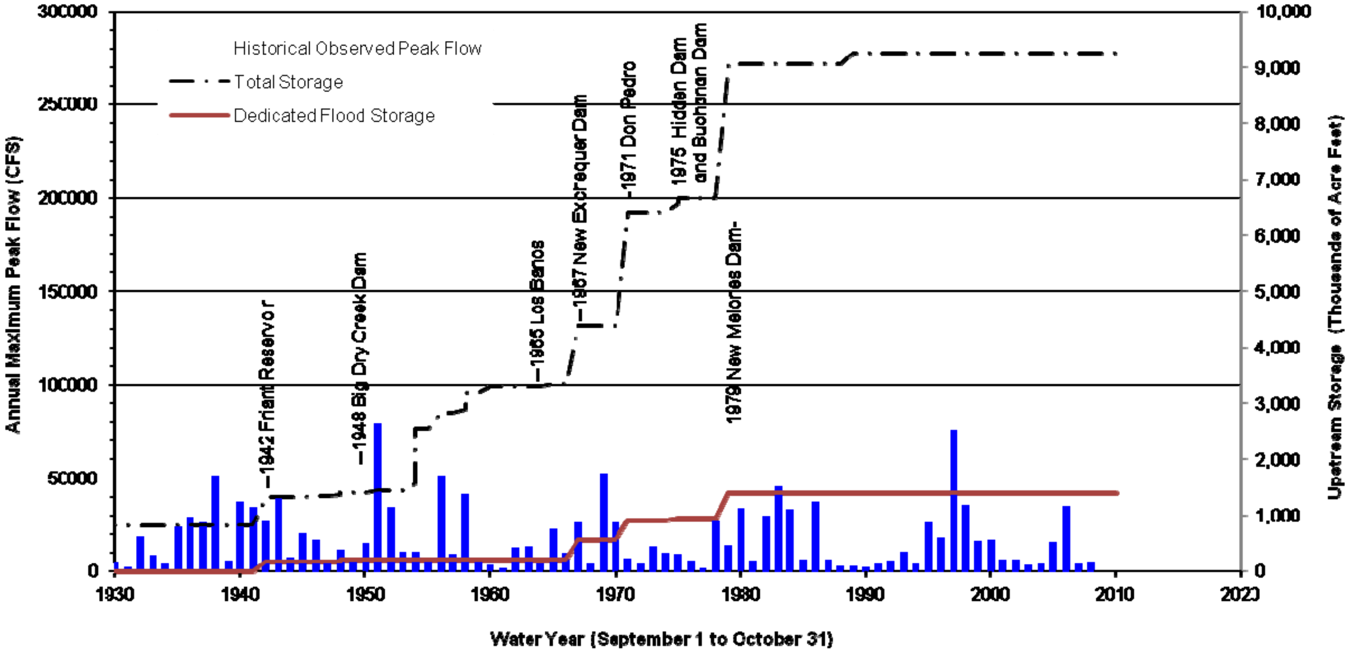
JUN 2014

PLATE 14

9/25/2014 DRAFT



Unregulated Conditions



Regulated Conditions

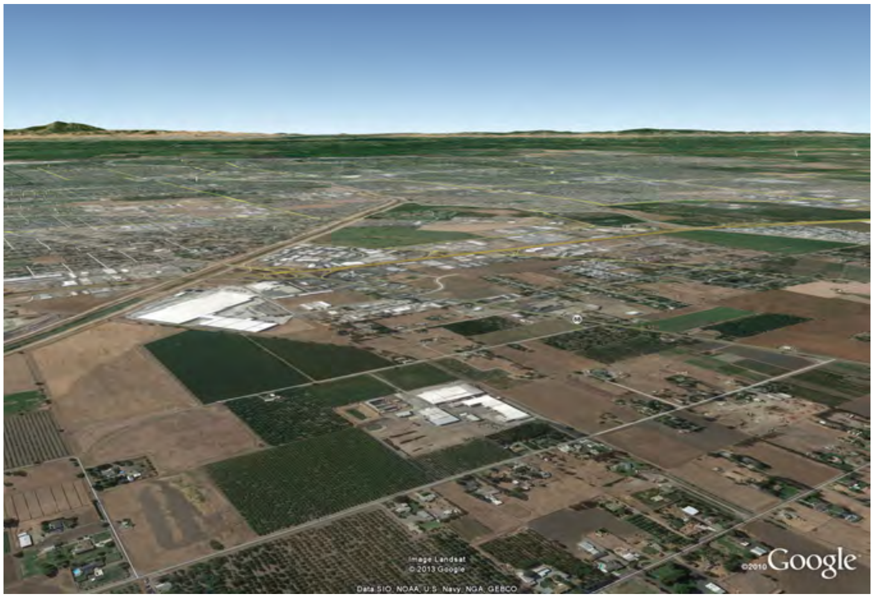
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASBILITY STUDY

ANNUAL MAXIMUM 1-DAY FLOW
SAN JOAQUIN RIVER AT VERNALIS
UNREGULATED AND REGULATED CONDITIONS

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by PJB

1955 Flood



2013 Conditions, Source: Google

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

MORMAN DIVERTING CANAL
1955 FLOOD COMPARED TO 2013 CONDITIONS

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014

Looking downstream (west) towards San Joaquin River, 1955 Flood

2013 Conditions, Source: Google Earth

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

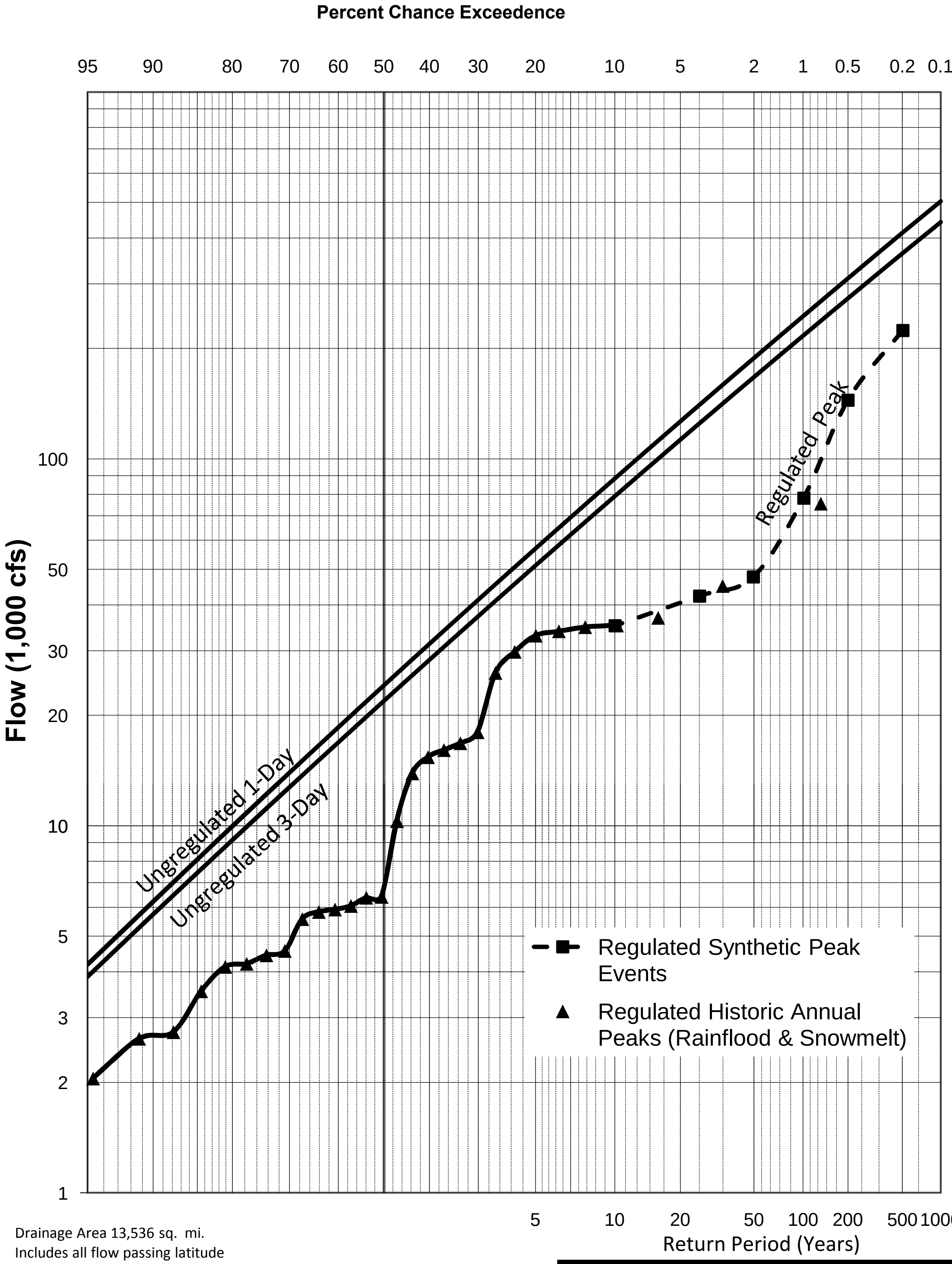
MORMAN SLOUGH
1955 FLOOD COMPARED TO 2013 CONDITIONS

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

NOV 2013

PLATE 10

9/25/2014 DRAFT



Drainage Area 13,536 sq. mi.
Includes all flow passing latitude
Median Plotting Positions

UNREGULATED FLOW
Log Statistics

	Mean	Std Dev	Skew
1 - Day	4.375	0.450	-0.1
3 - Day	4.333	0.445	-0.1

Period of Record 1917-1998
Source: Sacramento-San Joaquin Basin Comprehensive Study, March 2002

REGULATED PEAK FLOW
Beard Plotting Positions
Graphical Plot
Period of Record 1979 -2006

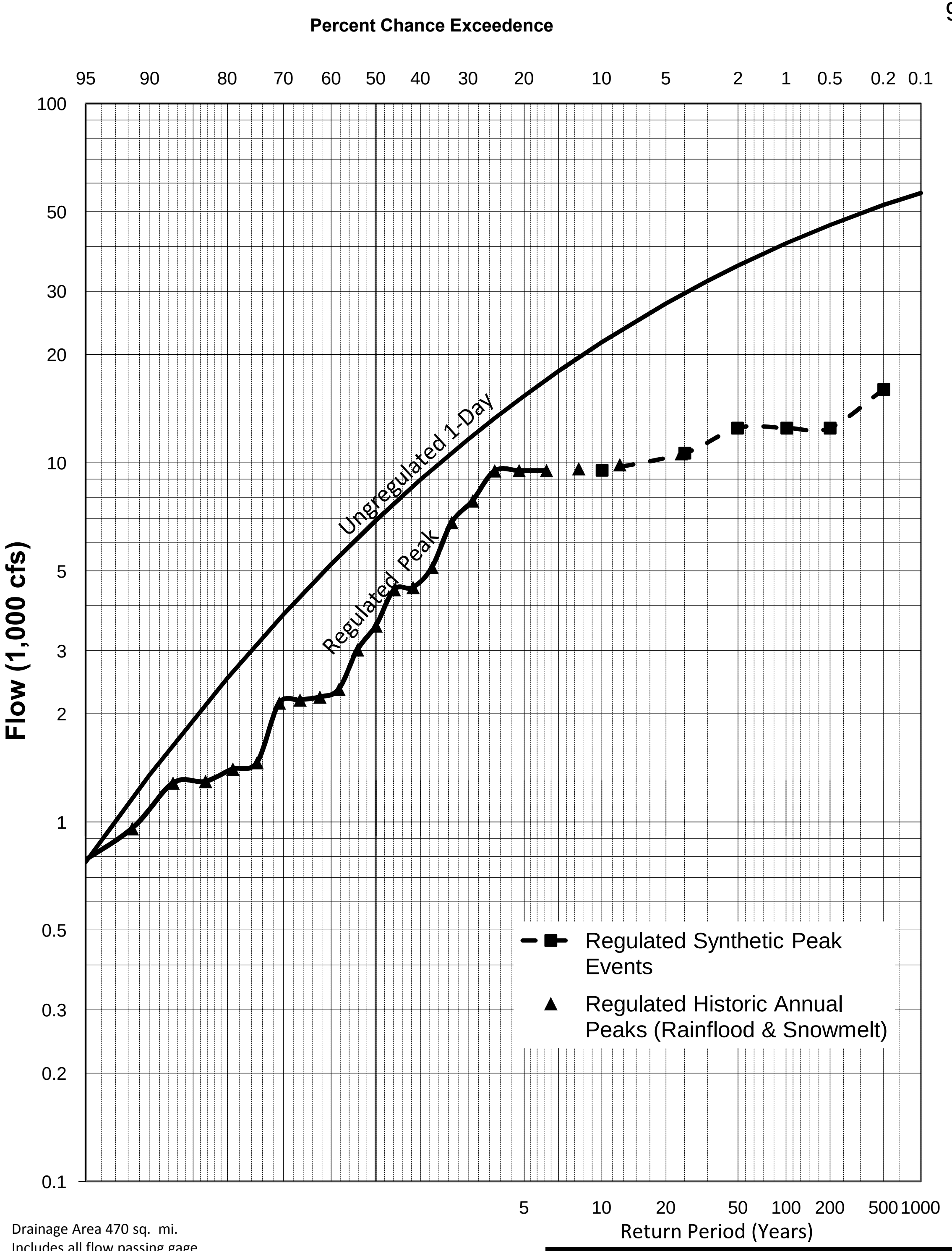
Regulated Hypothetical Events based on UNET modeling conducted for Sacramento-San Joaquin Basin Comprehensive Study, March 2002
1997 Peak Flow estimate did not account for overbank flow, USGS, 2013

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

SAN JOAQUIN RIVER NEAR VERNALIS
FLOOD FLOW FREQUENCY

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014



Drainage Area 470 sq. mi.
Includes all flow passing gage
Median Plotting Positions

UNREGULATED FLOW
Log Statistics

	Mean	Std Dev	Skew
1 - Day	3.775	0.482	-0.81

Period of Record 1907-2010
Source: Lower San Joaquin Feasibility Study
Hydraulic Appendix, May 2014

REGULATED PEAK FLOW
Weibull Plotting Positions
Graphical Plot
Period of Record 1988 -2011

Regulated Hypothetical Events based on Reservoir Simulation Model

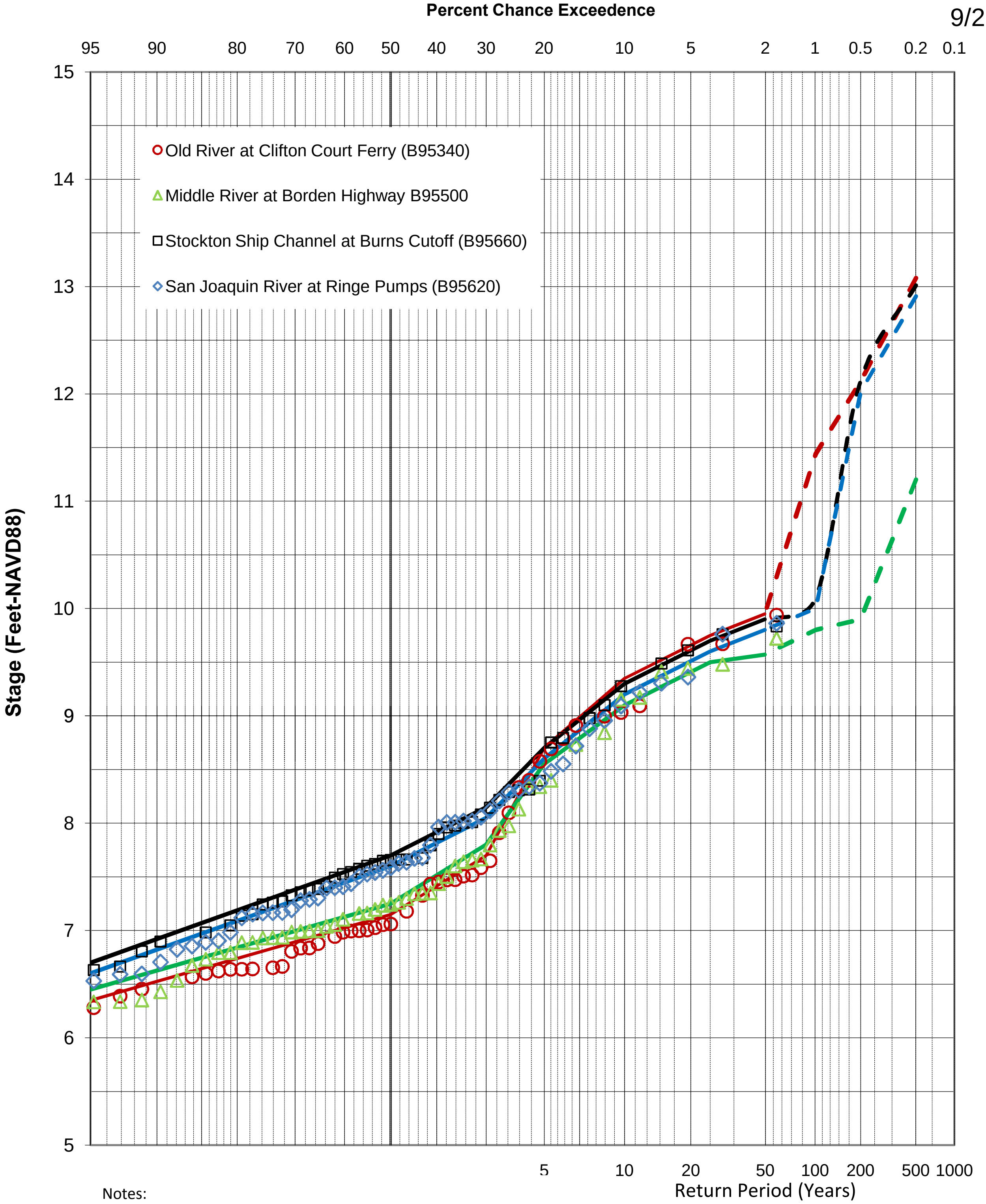
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

MORMON SLOUGH AT BELLOTA
FLOOD FLOW FREQUENCY

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014

9/25/2014 DRAFT



Notes:
1. Period of Record 1953 to 2009
2. Missing Records estimated by correlation:
   B95340: 1953-1957,1971,1987,1997
   B95500: 1958,1973,1989
   B95620: No missing data
   B95660: 1953-1958
3. Historic stages adjusted to 2010 Sea Level using historical 1.7mm/yr eustatic sea level rise
4. Extrapolation to from 1% to 0.2% (dashed) based on HEC-RAS Model results. While suitable for economic analysis, estimates should be refined for design purposes.

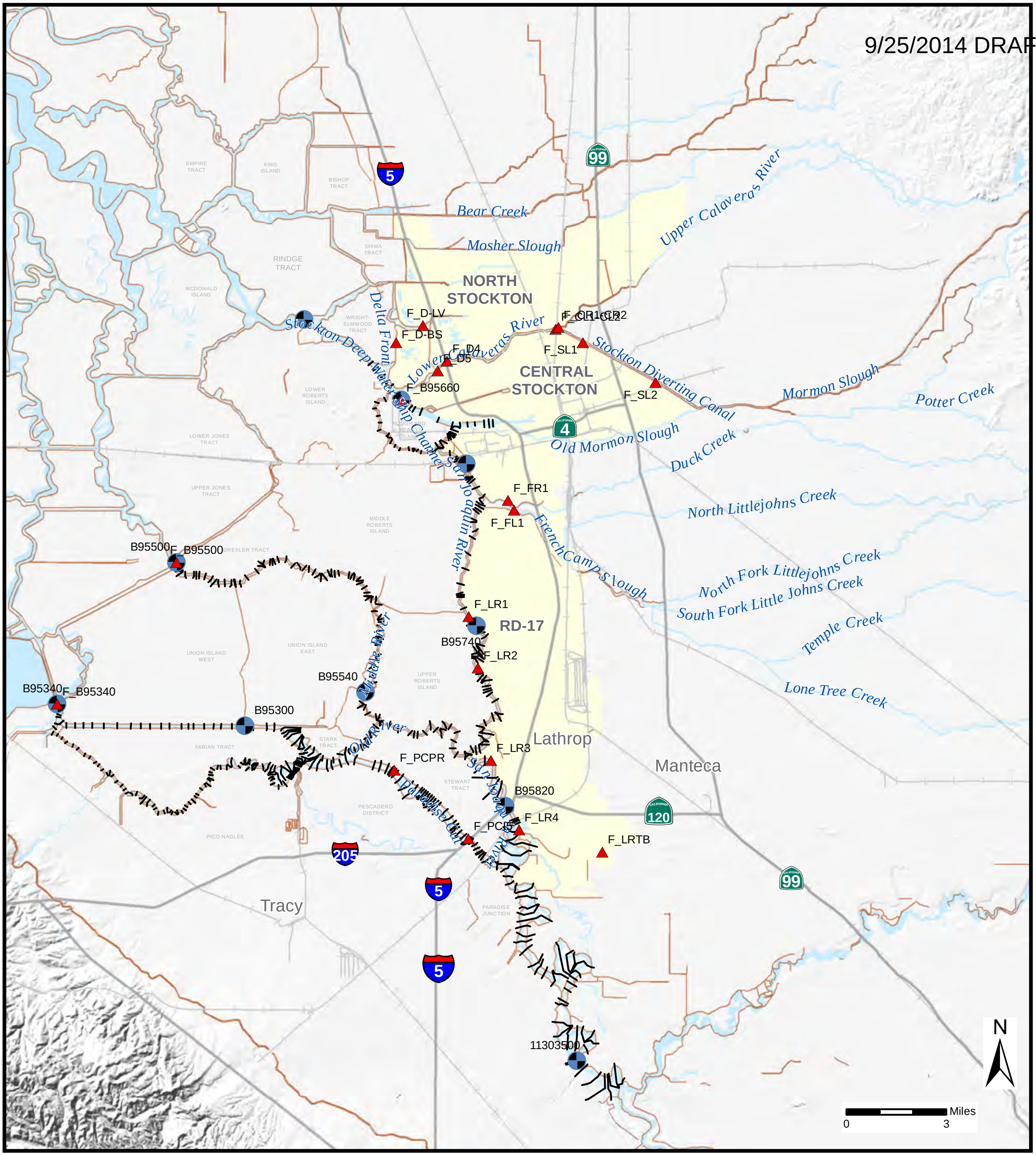
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

STAGE-FREQUENCY CURVES
HEC-RAS DOWNSTREAM BOUNDARIES
2010 CONDITIONS

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

MAY 2014

9/25/2014 DRAFT



Legend

- ● Stream Gages
- — Model Cross-Sections
- ╱ Highway
- ┼ Railroads
- ∼ Levees (Fed/Non-Fed)
- ▭ Study Extent
- ▲ Index Point

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

SAN JOAQUIN RIVER
HEC-RAS MODEL EXTENT

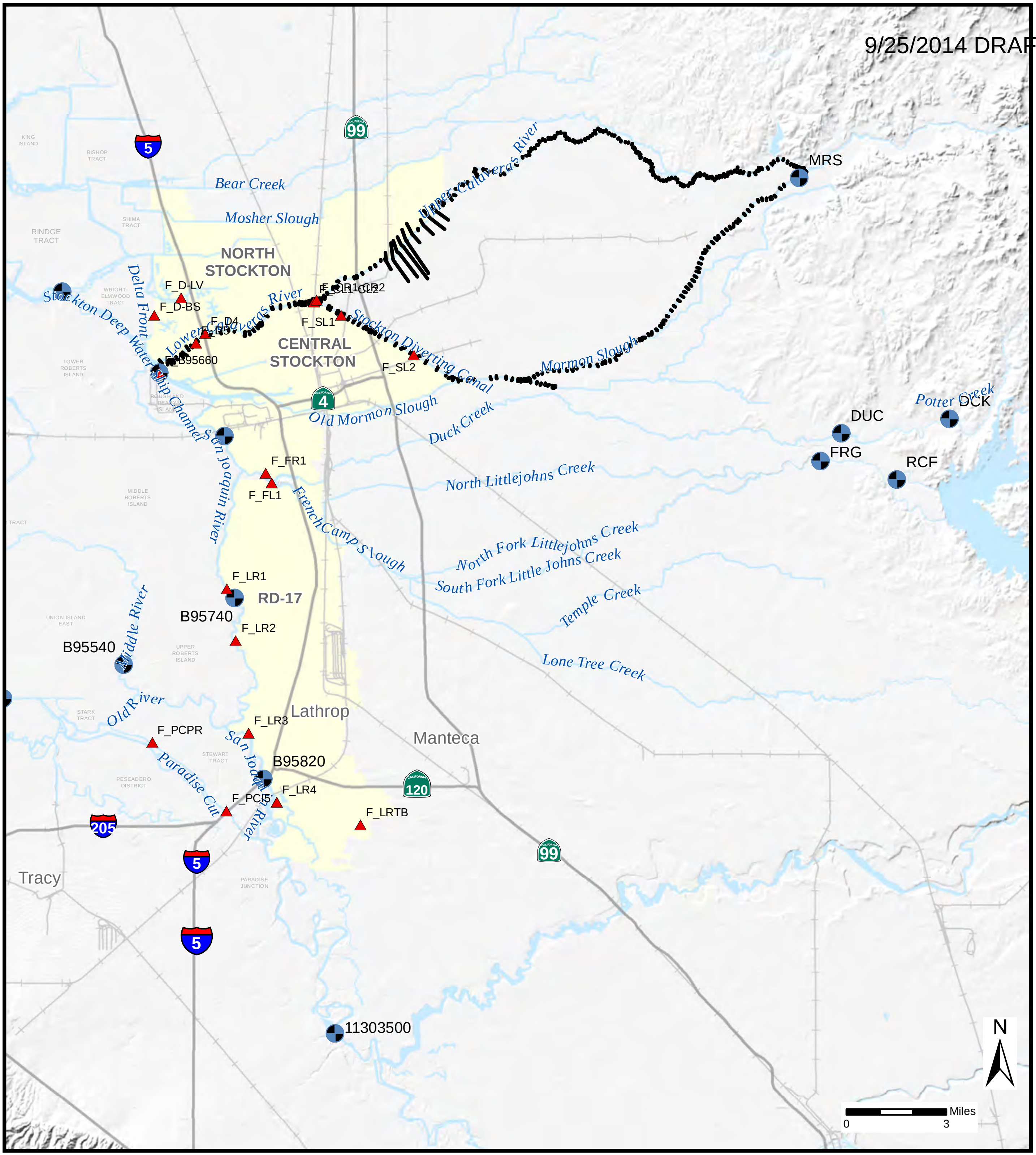
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUG 2014

PLATE 21

9/25/2014 DRAFT



Legend

- ◕ Stream Gages
- ━ Model Cross Sections
- ╱ Highway
- ┼ Railroads
- ∿ Levees (Fed/Non-Fed)
- ▭ Study Extent
- ▲ Index Point

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

CALAVERAS RIVER
HEC-RAS MODEL

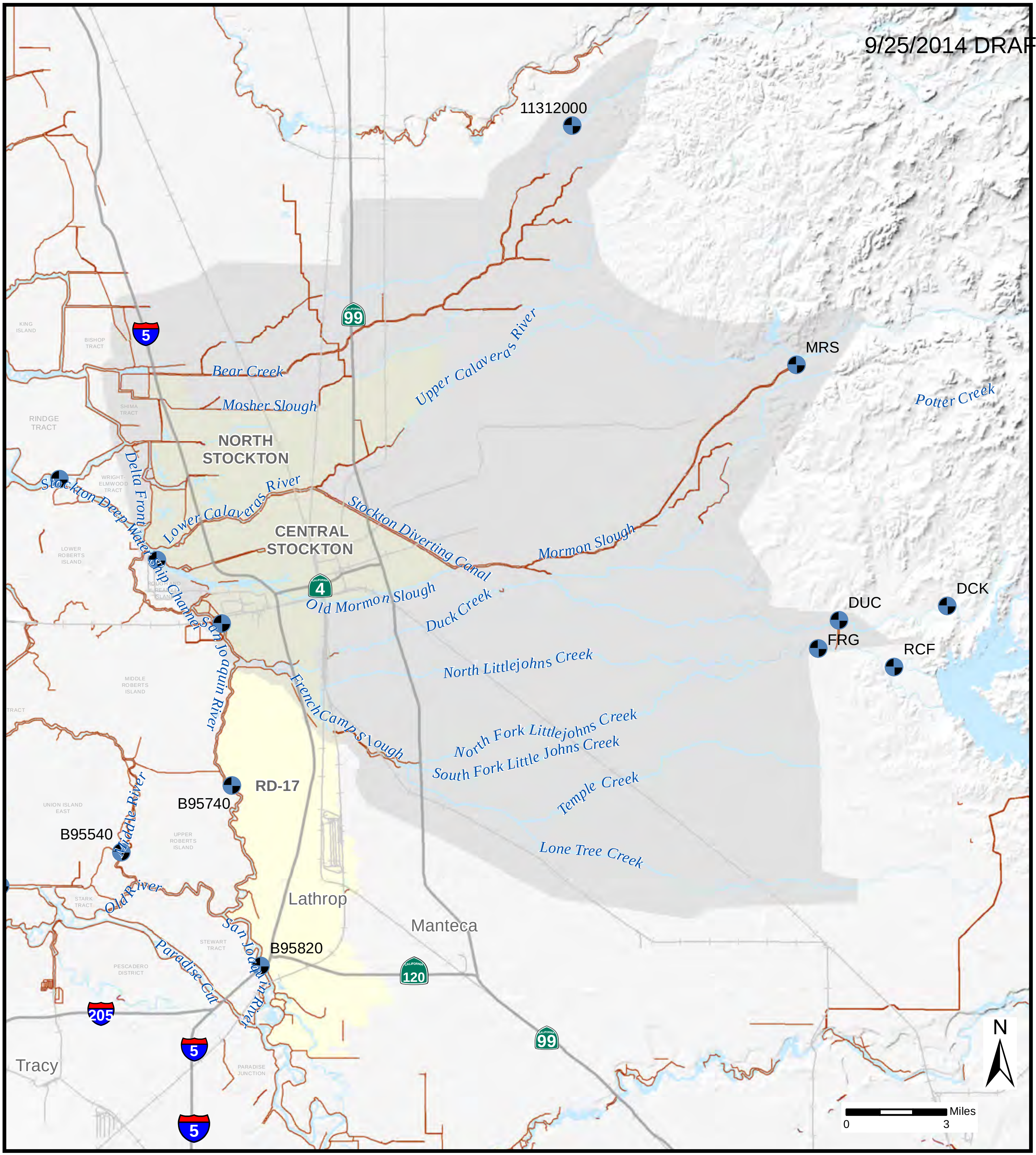
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

AUG 2014

PLATE 22

9/25/2014 DRAFT



Legend

● Stream Gages

Highway

Railroads

Study Extent

North FLO2D Model Extent

NOTE: Analysis Limited to Study Extent.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

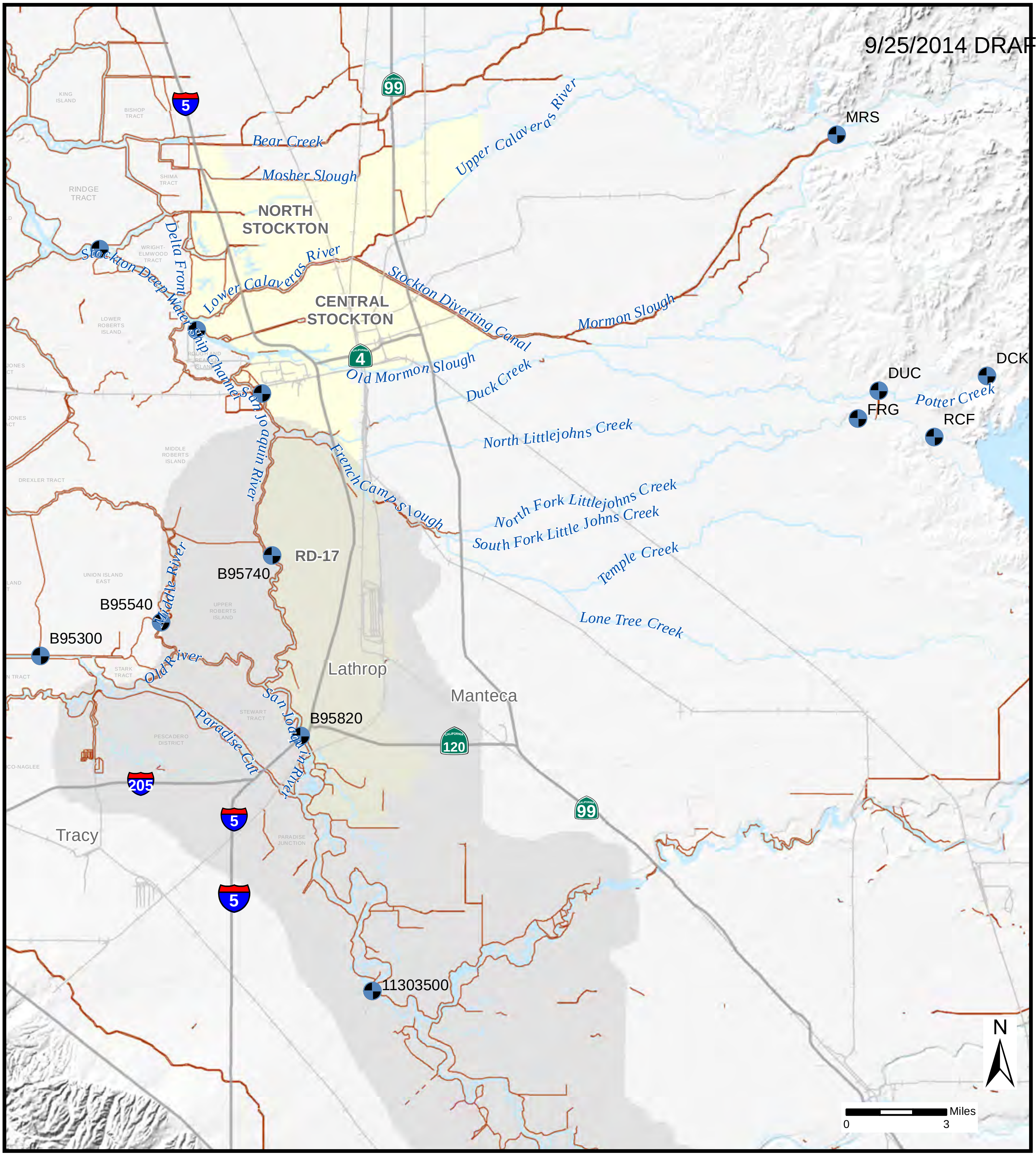
NORTH FLO-2D
MODEL DOMAIN

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 23



Legend

- ◐ Stream Gages
- ／ Highway
- ┼ Railroads
- 🟨 Study Extent
- ⬜ South FLO2D Model Extents

NOTE: Analysis Limited to Study Extent.

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

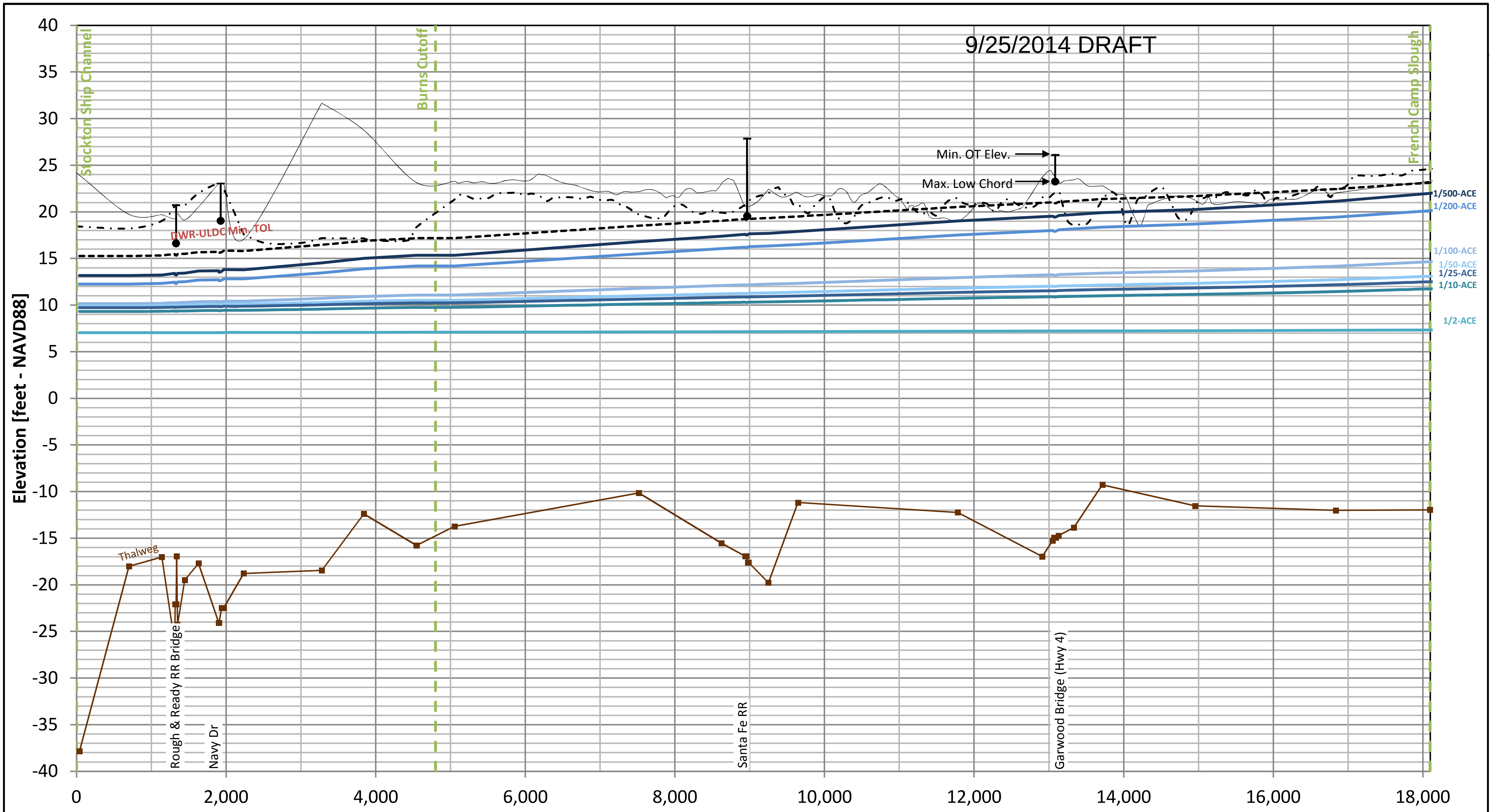
SOUTH FLO-2D MODEL DOMAIN

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 24



River Station [feet]

(Distance from Stockton Ship Channel)

Peak Flows d/s FCS	
1/2-ACE	- 2,450 cfs
1/10-ACE	- 9,870 cfs
1/25-ACE	- 11,150 cfs
1/50-ACE	- 12,300 cfs
1/100-ACE	- 15,900 cfs
1/200-ACE	- 28,700 cfs
1/500-ACE	- 35,200 cfs

Peak Flows d/s Burns Cutoff	
1/2-ACE	- 2,260 cfs
1/10-ACE	- 8,930 cfs
1/25-ACE	- 10,040 cfs
1/50-ACE	- 11,040 cfs
1/100-ACE	- 14,200 cfs
1/200-ACE	- 25,000 cfs
1/500-ACE	- 28,500 cfs

—— Top of Levee - RB
— · — · Top of Levee - LB
– – Stream Junction
●——┤ Bridge (Max. Low Chord; Min. OT Elev.)

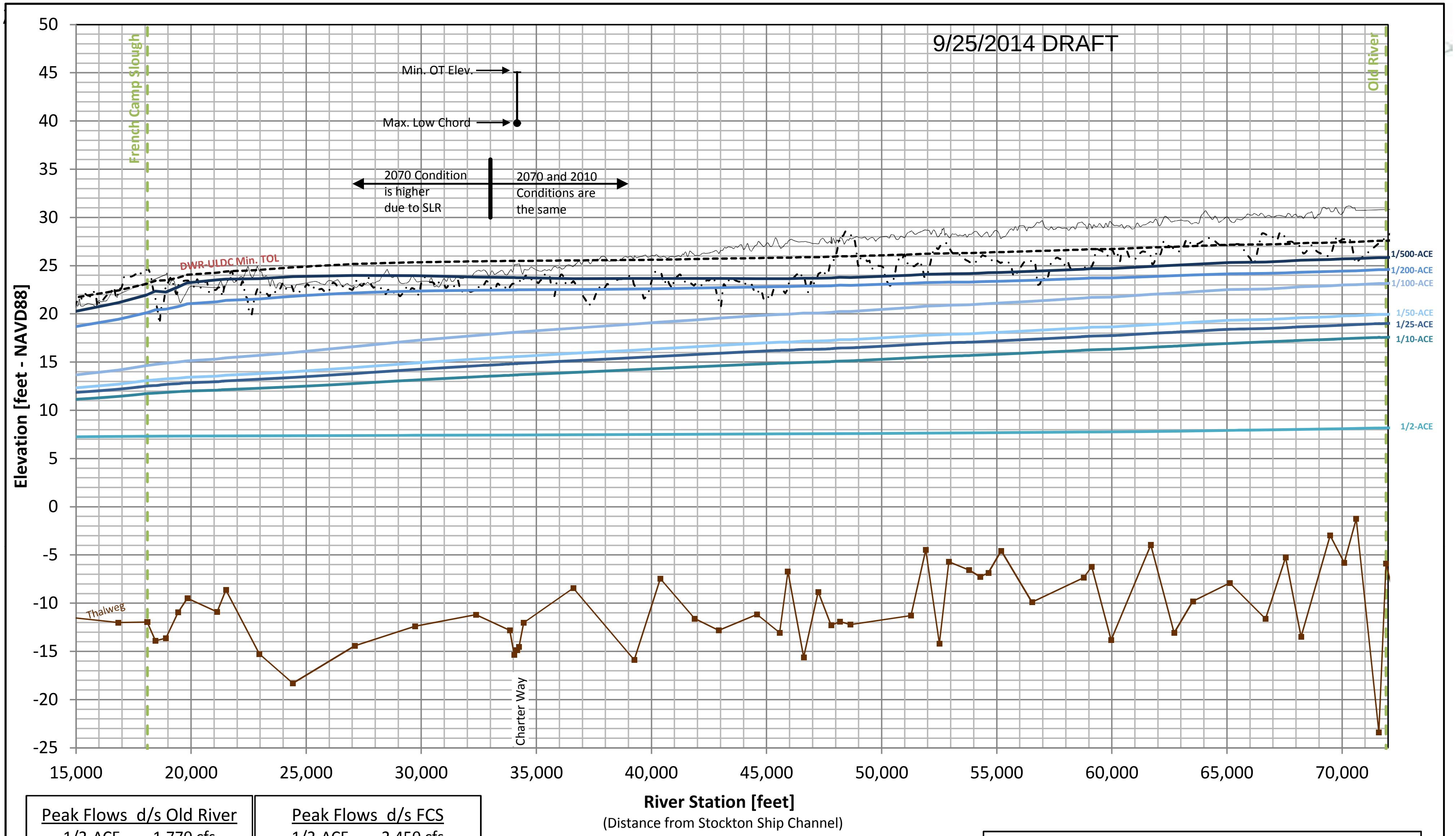
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
Stockton Ship Channel to French Camp Slough
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 25A



Peak Flows d/s Old River
1/2-ACE - 1,770 cfs
1/10-ACE - 7,800 cfs
1/25-ACE - 9,100 cfs
1/50-ACE - 10,100 cfs
1/100-ACE - 13,900 cfs
1/200-ACE - 15,800 cfs
1/500-ACE - 17,000 cfs

Peak Flows d/s FCS
1/2-ACE - 2,450 cfs
1/10-ACE - 9,870 cfs
1/25-ACE - 11,150 cfs
1/50-ACE - 12,300 cfs
1/100-ACE - 15,900 cfs
1/200-ACE - 28,700 cfs
1/500-ACE - 35,200 cfs

—— Top of Levee - RB
-·-·- Top of Levee - LB
- - Stream Junction
●—— Bridge (Max. Low Chord; Min. OT Elev.)

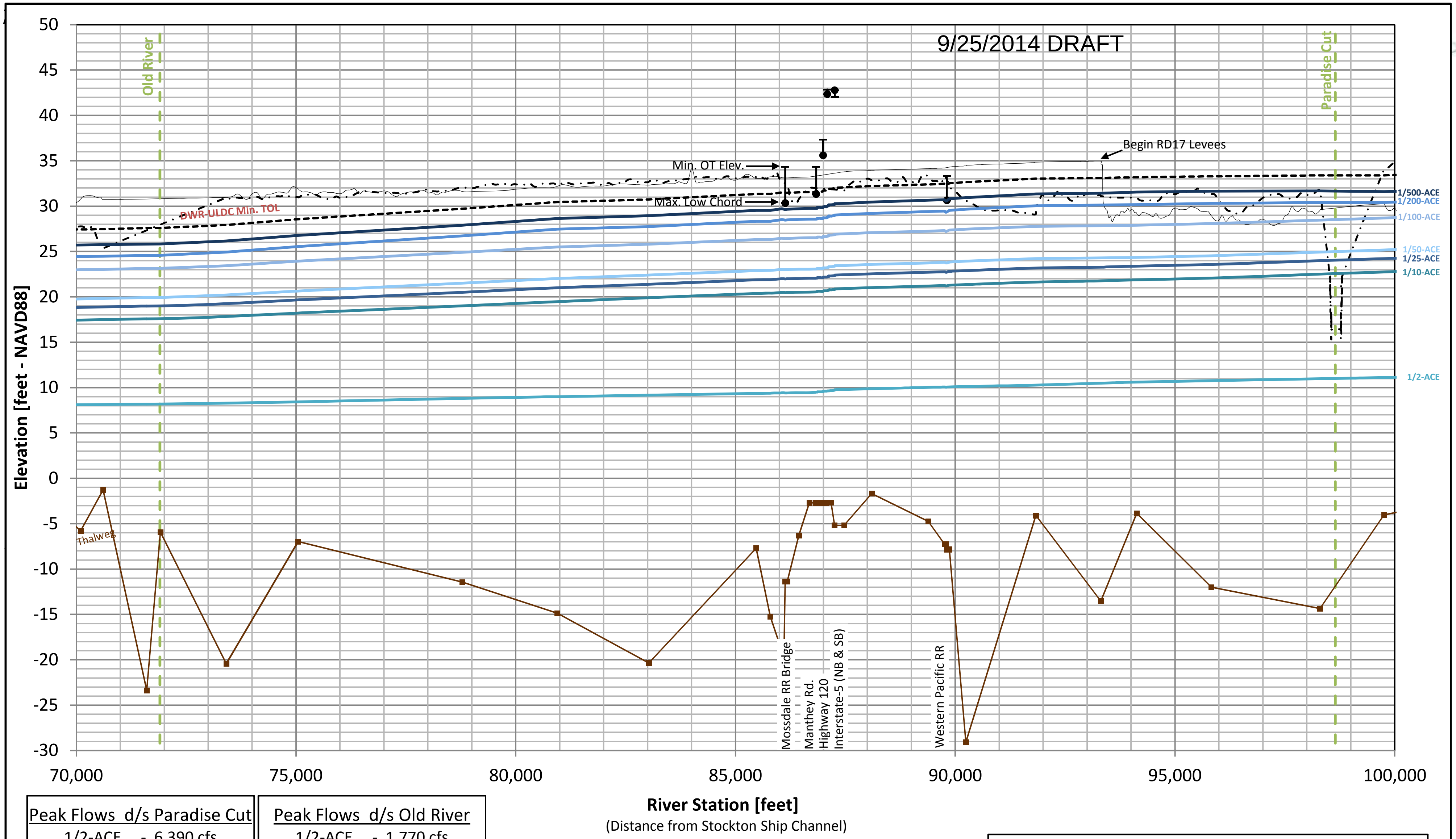
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
French Camp Slough to Old River
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 25B



Peak Flows d/s Paradise Cut
1/2-ACE - 6,390 cfs
1/10-ACE -25,200cfs
1/25-ACE - 28,900 cfs
1/50-ACE - 31,600 cfs
1/100-ACE - 43,000 cfs
1/200-ACE - 47,300 cfs
1/500-ACE - 49,400 cfs

Peak Flows d/s Old River
1/2-ACE - 1,770 cfs
1/10-ACE - 7,800 cfs
1/25-ACE - 9,100 cfs
1/50-ACE - 10,100 cfs
1/100-ACE - 13,900 cfs
1/200-ACE - 15,800 cfs
1/500-ACE - 17,000 cfs

Top of Levee - RB
Top of Levee - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

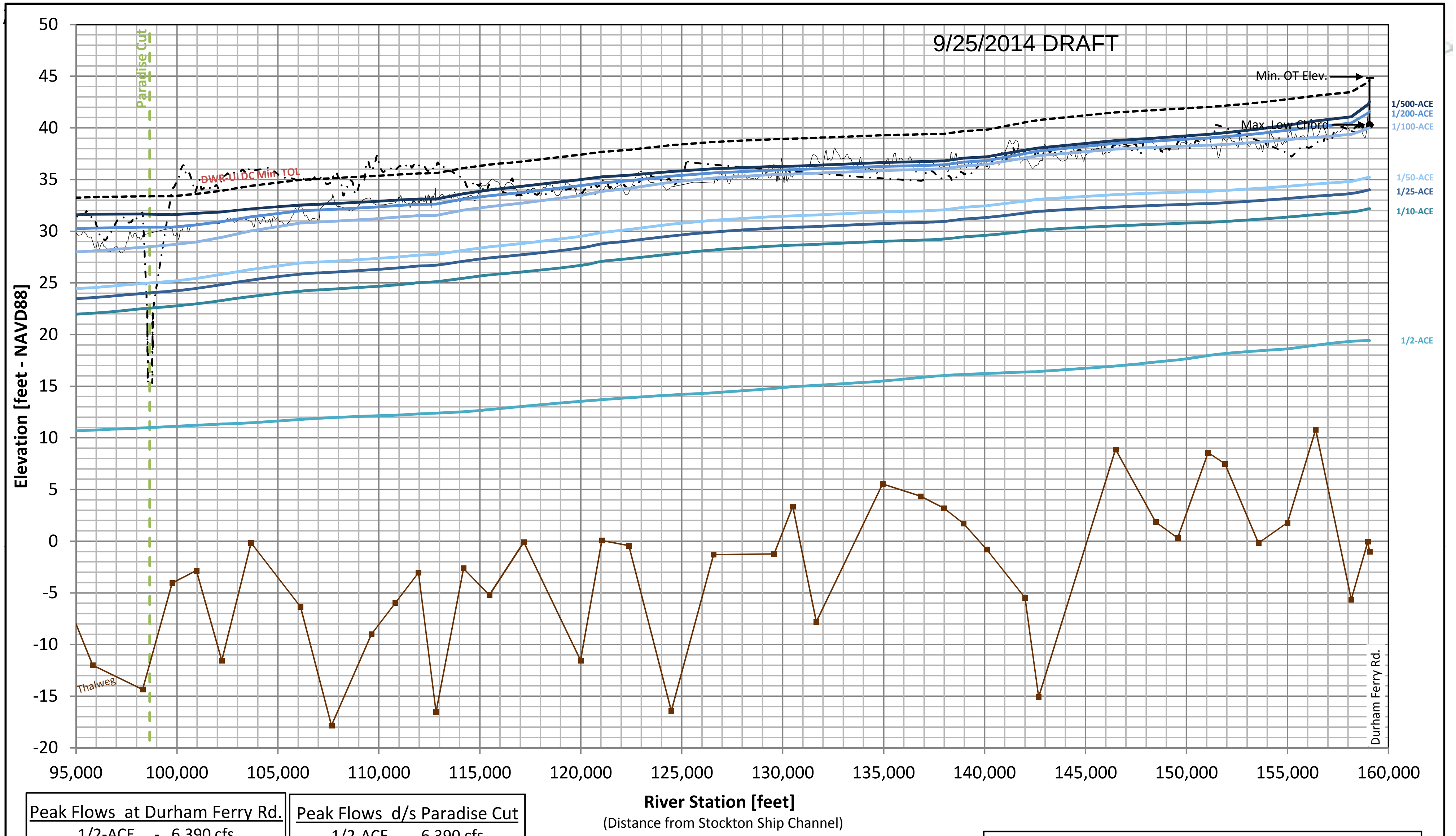
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
Old River to Paradise Cut
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 25C



Peak Flows at Durham Ferry Rd.
1/2-ACE - 6,390 cfs
1/10-ACE - 35,100 cfs
1/25-ACE - 42,300 cfs
1/50-ACE - 47,600 cfs
1/100-ACE - 78,200 cfs
1/200-ACE - 124,500 cfs
1/500-ACE - 165,100 cfs

Peak Flows d/s Paradise Cut
1/2-ACE - 6,390 cfs
1/10-ACE - 25,200cfs
1/25-ACE - 28,900 cfs
1/50-ACE - 31,600 cfs
1/100-ACE - 43,000 cfs
1/200-ACE - 47,300 cfs
1/500-ACE - 49,400 cfs

——— Top of Levee - RB
-·-·- Top of Levee - LB
– + – Stream Junction
●——| Bridge (Max. Low Chord; Min. OT Elev.)

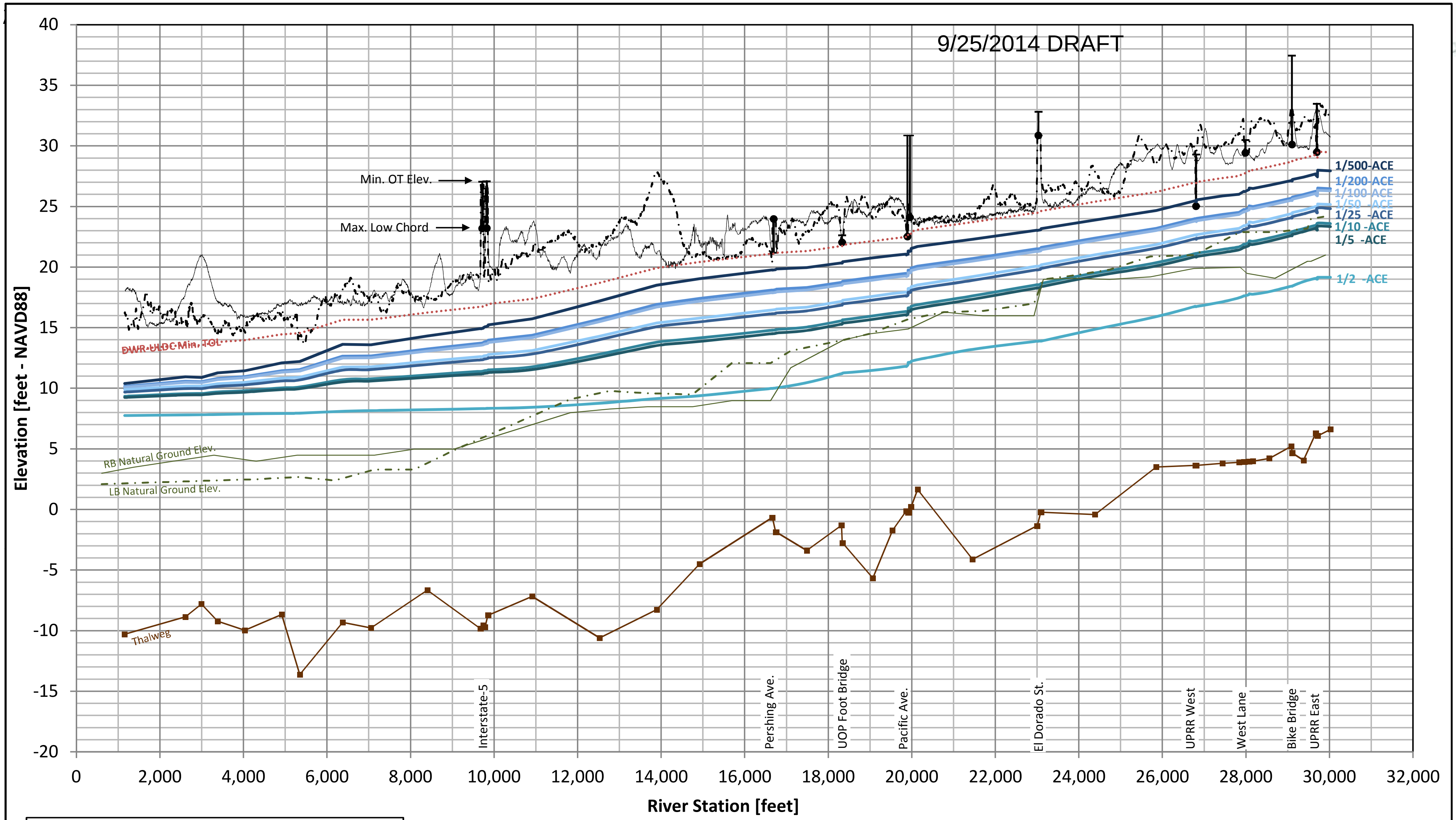
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
Paradise Cut to Durham Ferry Road
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 25D



Peak Flows in Reach	
1/2-ACE - 3,850 cfs	1/50-ACE - 12,850 cfs
1/5-ACE - 9,500 cfs	1/100-ACE - 15,360 cfs
1/10-ACE - 9,860 cfs	1/200-ACE - 15,750 cfs
1/25-ACE - 12,280 cfs	1/500-ACE - 19,130 cfs

—— Top of Levee - RB

-·-·- Top of Levee - LB

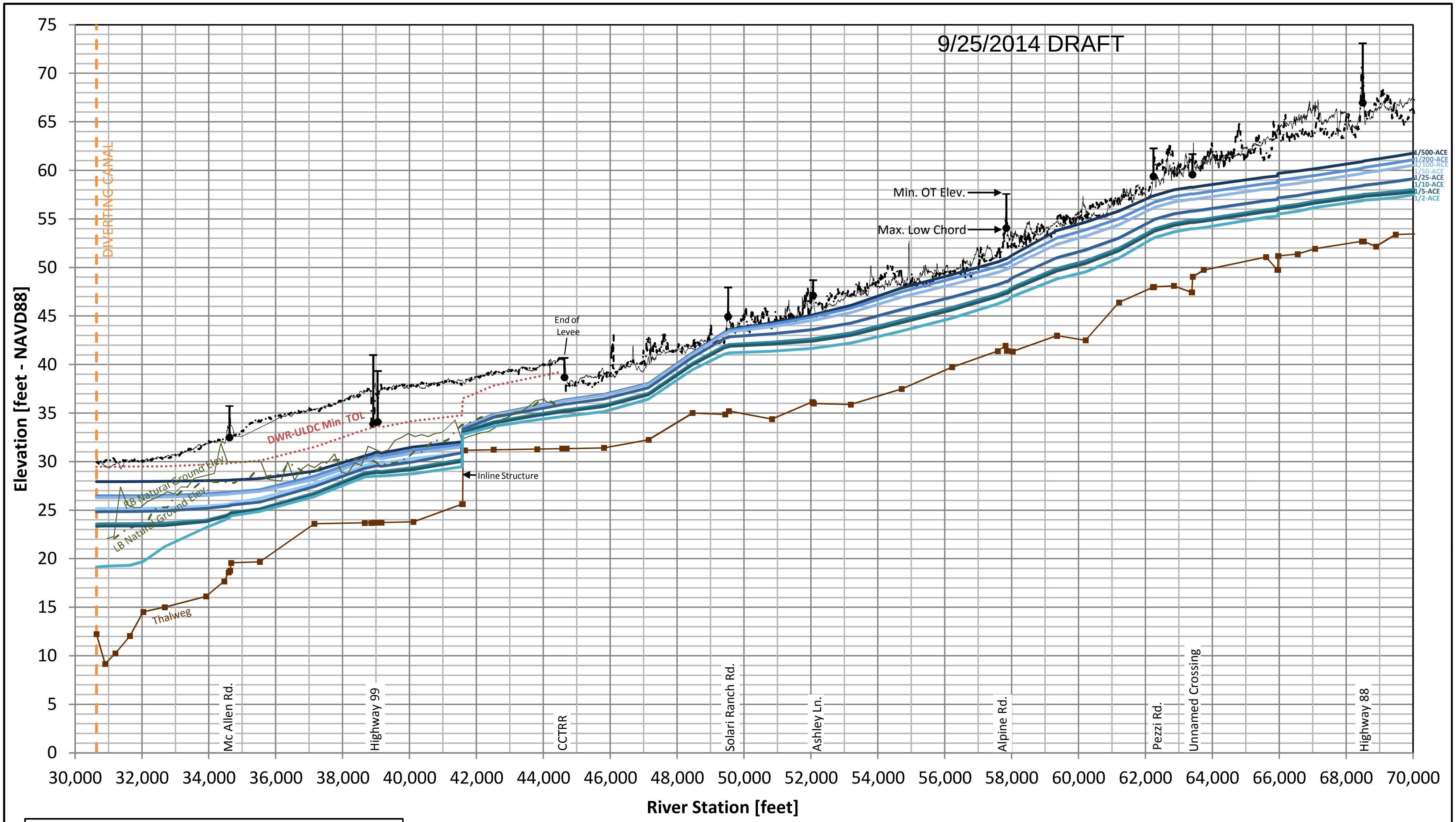
●—┘ Bridge (Max. Low Chord; Min. OT Elev.)

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Calaveras River
Downstream of Diverting Canal
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014



Peak Flows at Highway 88

1/2-ACE - 170 cfs	1/50-ACE - 340 cfs
1/5-ACE - 220 cfs	1/100-ACE - 490 cfs
1/10-ACE - 240 cfs	1/200-ACE - 570 cfs
1/25-ACE - 340 cfs	1/500-ACE - 680 cfs

── Top of Levee/Bank - RB

-·-·- Top of Levee/Bank - LB

- - - Stream Junction

●──┤ Bridge (Max. Low Chord; Min. OT Elev.)

September 18, 2013

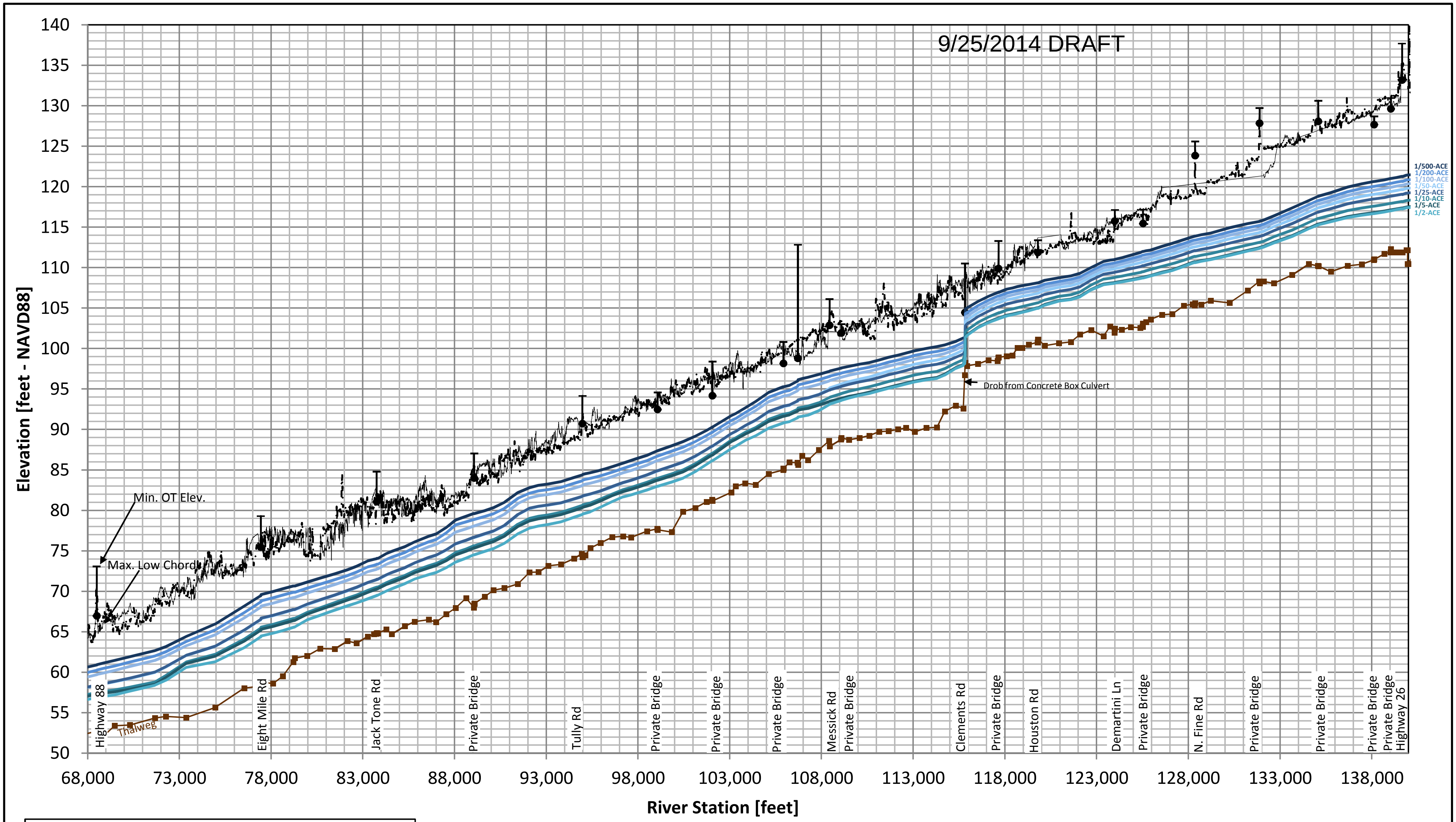
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Calaveras River
Stockton Diverting Canal to Hwy 88
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 27A



Peak Flows at Fine Rd (d/s of Podesta Reservoir)
1/2-ACE - 150 cfs
1/5-ACE - 160 cfs
1/10-ACE - 210 cfs
1/25-ACE - 270 cfs
1/50-ACE - 320 cfs
1/100-ACE - 370 cfs
1/200-ACE - 420 cfs
1/500-ACE - 480 cfs

——— Top of Levee/Bank - RB
-·-·- Top of Levee/Bank - LB
– – Stream Junction
●——| Bridge (Max. Low Chord; Min. OT Elev.)

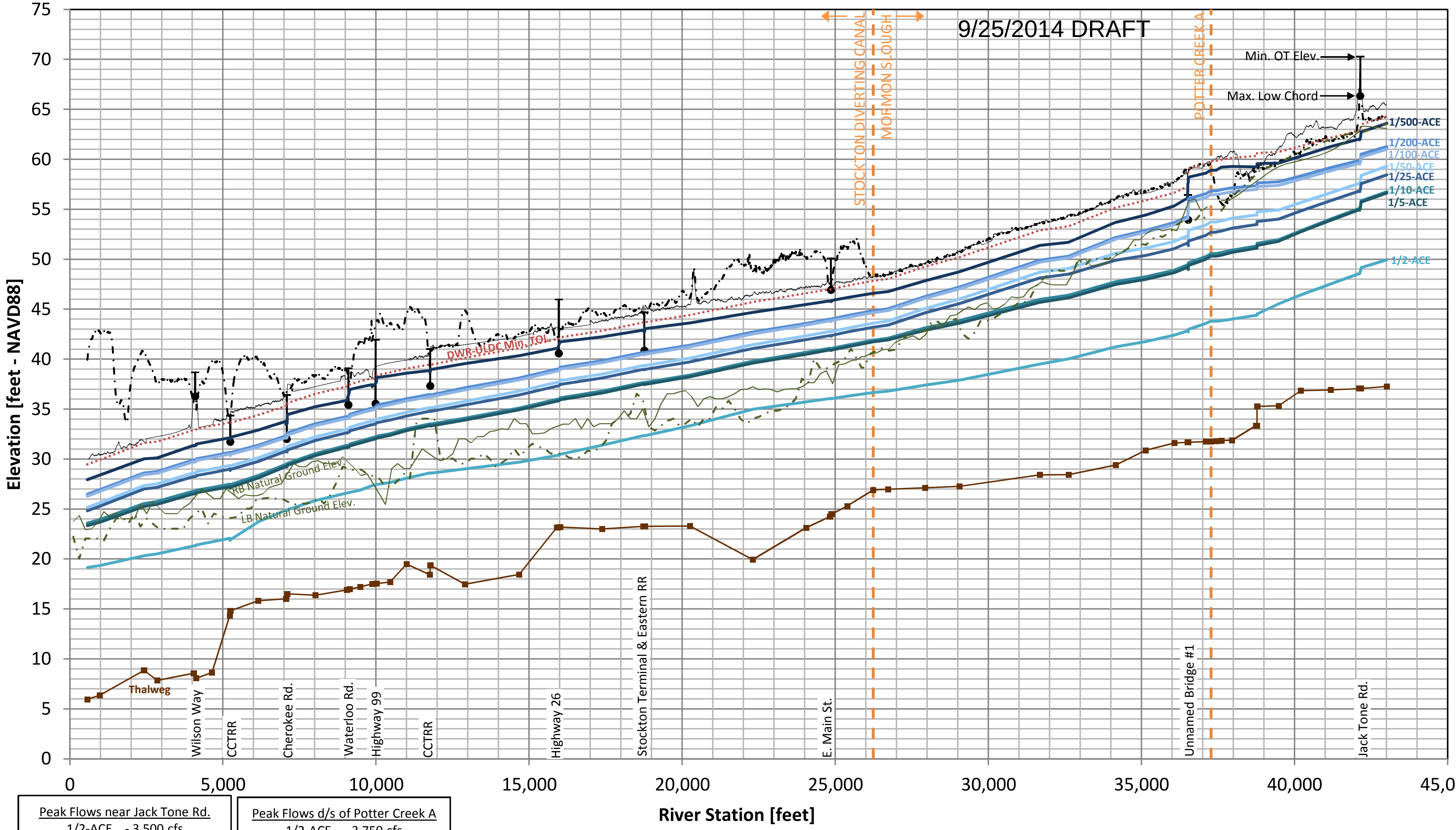
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Calaveras River
Hwy 88 to Bellota
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 27B



Peak Flows near Jack Tone Rd.	
1/2-ACE	- 3,500 cfs
1/5-ACE	- 9,400 cfs
1/10-ACE	- 9,500 cfs
1/25-ACE	- 11,300 cfs
1/50-ACE	- 12,500 cfs
1/100-ACE	- 13,800 cfs
1/200-ACE	- 14,050 cfs
1/500-ACE	- 17,450 cfs

Peak Flows d/s of Potter Creek A	
1/2-ACE	- 3,750 cfs
1/5-ACE	- 9,400 cfs
1/10-ACE	- 9,650 cfs
1/25-ACE	- 11,950 cfs
1/50-ACE	- 12,650 cfs
1/100-ACE	- 14,950 cfs
1/200-ACE	- 15,200 cfs
1-500-ACE	- 18,400 cfs

——— Top of Levee - RB

-·-·- Top of Levee - LB

- - - Stream Junction

●——┤ Bridge (Max. Low Chord; Min. OT Elev.)

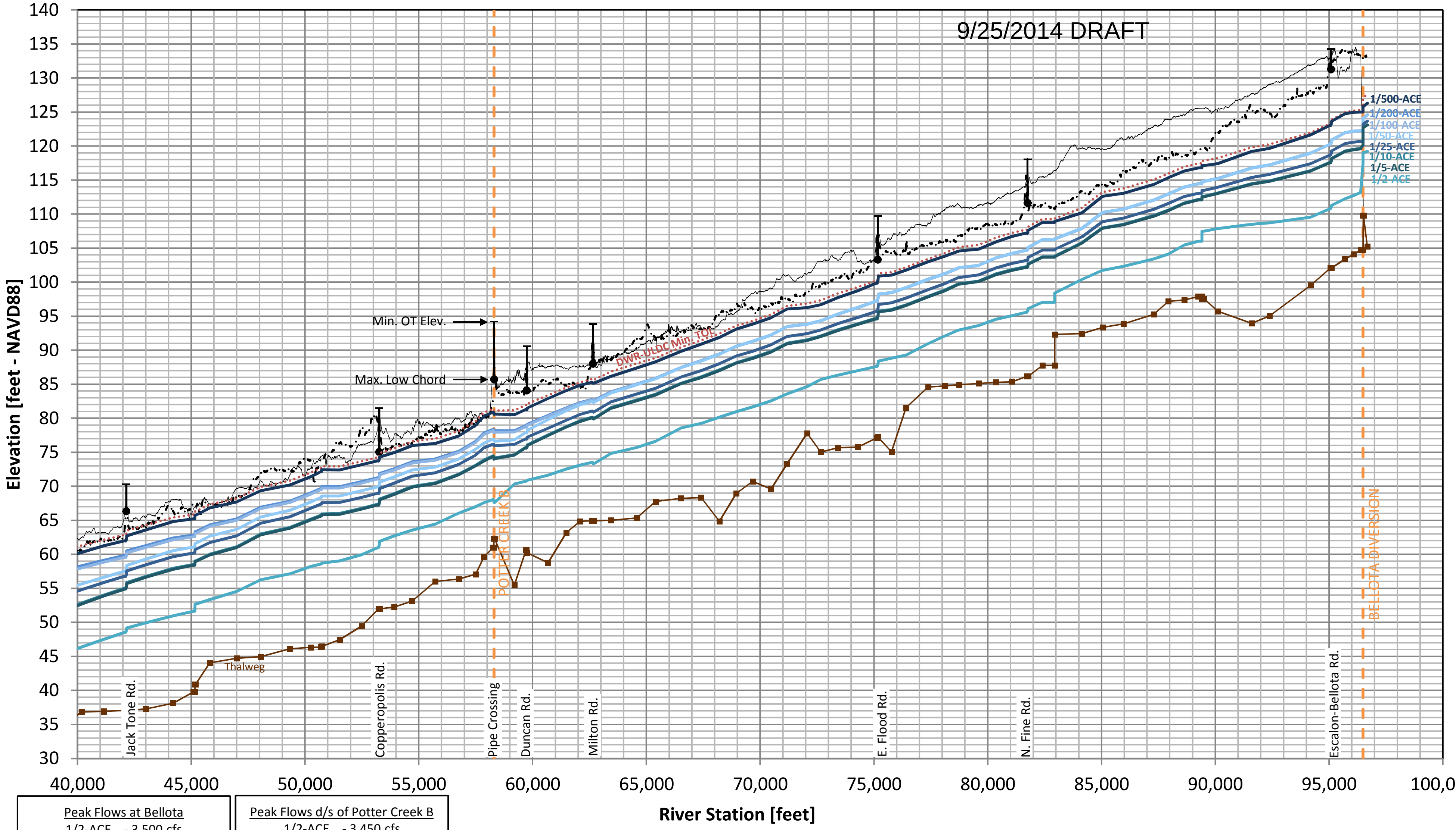
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STOCKTON DIVERTING CANAL / MORMON SLOUGH Downstream of Jack Tone Rd. 2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 28A



Peak Flows at Bellota
1/2-ACE - 3,500 cfs
1/5-ACE - 9,500 cfs
1/10-ACE - 9,550 cfs
1/25-ACE - 10,650 cfs
1/50-ACE - 12,500 cfs
1/100-ACE - 12,500 cfs
1/200-ACE - 12,500 cfs
1/500-ACE - 16,000 cfs

Peak Flows d/s of Potter Creek B
1/2-ACE - 3,450 cfs
1/5-ACE - 9,350 cfs
1/10-ACE - 9,500 cfs
1/25-ACE - 11,350 cfs
1/50-ACE - 12,500 cfs
1/100-ACE - 13,800 cfs
1/200-ACE - 14,050 cfs
1-500-ACE - 17,650 cfs

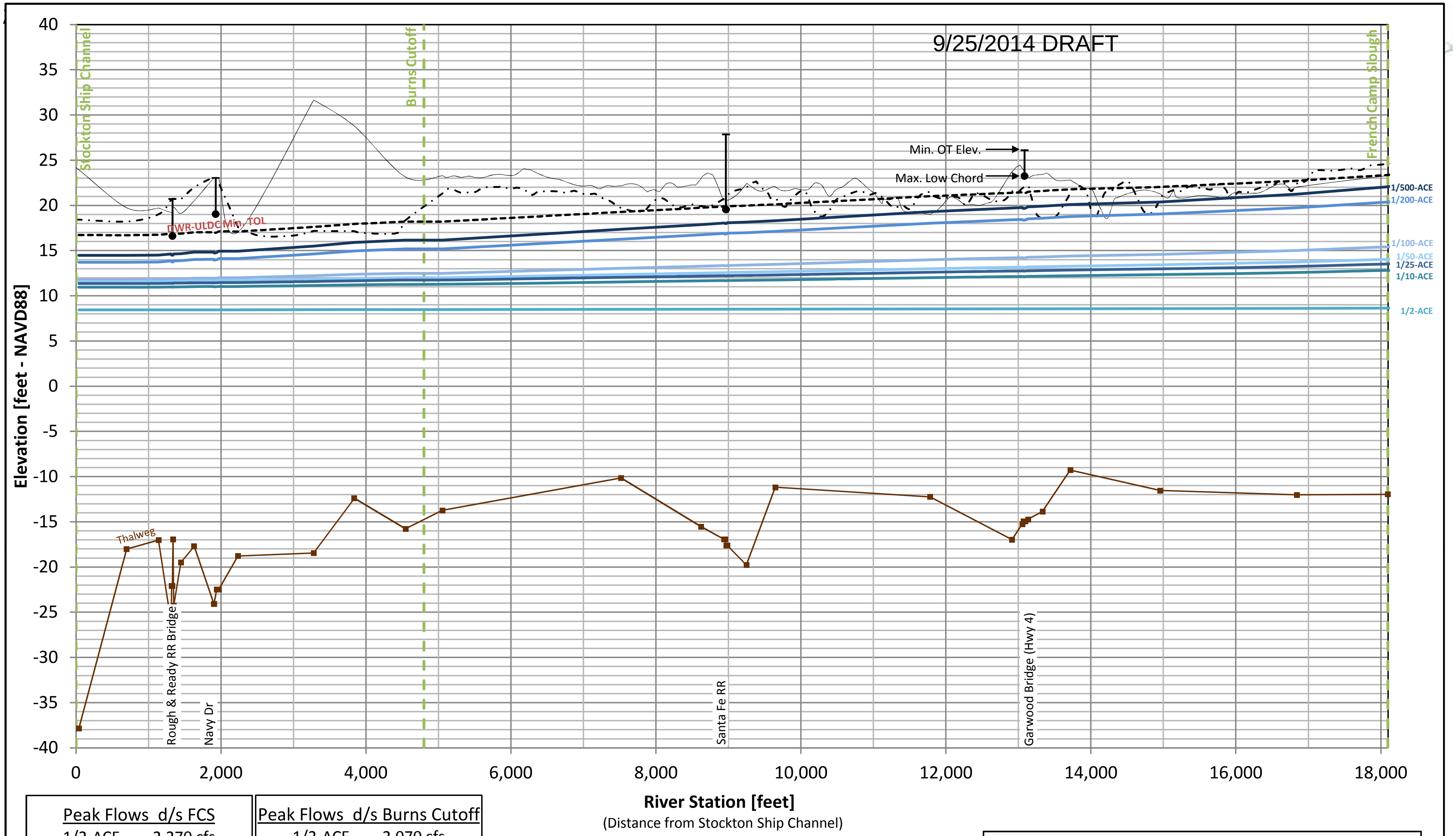
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STOCKTON DIVERTING CANAL /
MORMON SLOUGH
Upstream of Jack Tone Rd.
2010 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 28B



Peak Flows d/s FCS
1/2-ACE - 2,270 cfs
1/10-ACE - 9,780 cfs
1/25-ACE - 11,040 cfs
1/50-ACE - 12,200 cfs
1/100-ACE - 15,800 cfs
1/200-ACE - 27,900 cfs
1/500-ACE - 34,600 cfs

Peak Flows d/s Burns Cutoff
1/2-ACE - 2,070 cfs
1/10-ACE - 8,720 cfs
1/25-ACE - 9,800 cfs
1/50-ACE - 10,800 cfs
1/100-ACE - 13,900 cfs
1/200-ACE - 24,100 cfs
1/500-ACE - 27,300 cfs

Top of Levee - RB
Top of Levee - LB
Stream Junction
Bridge (Max. Low Chord; Min. OT Elev.)

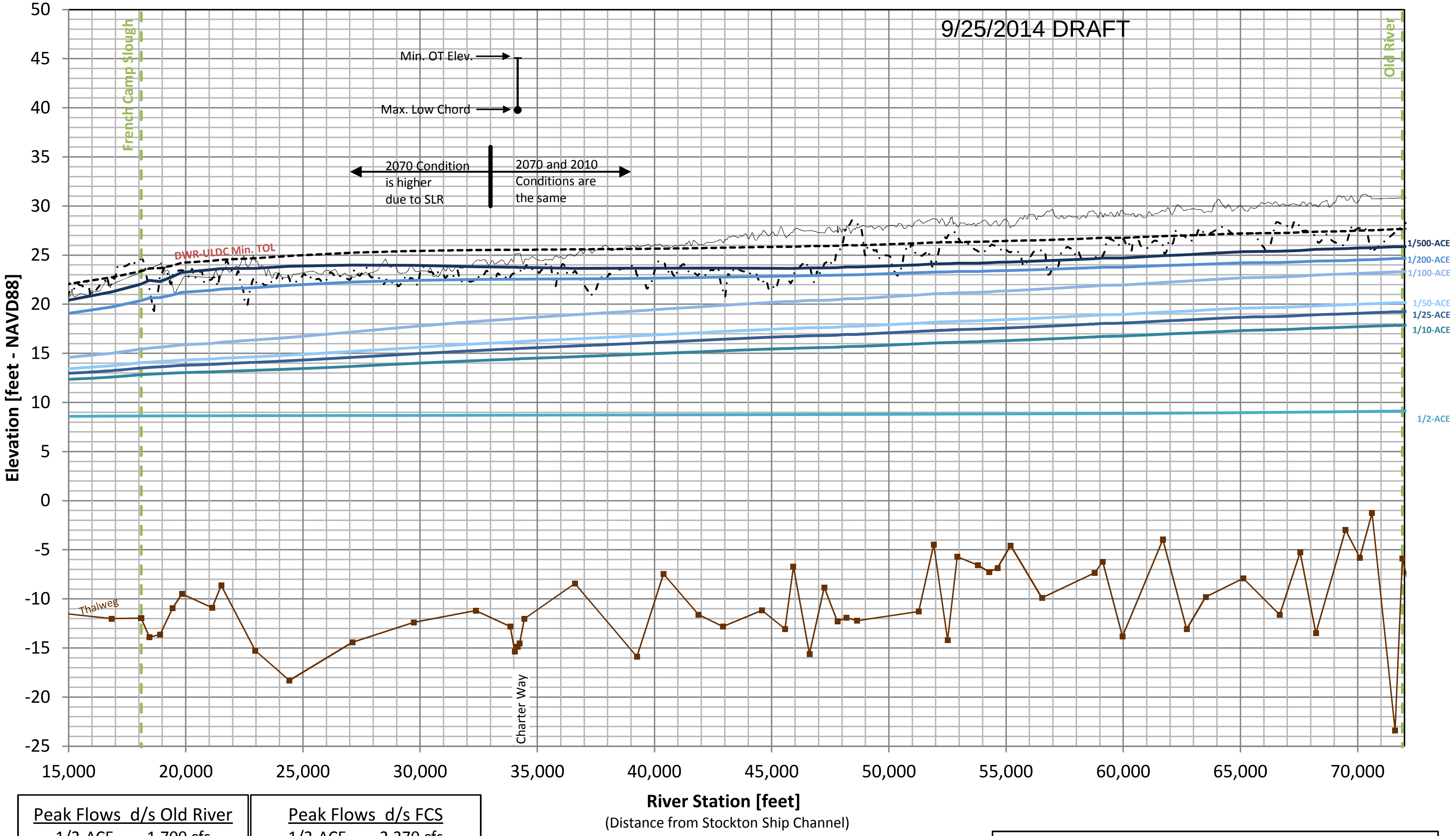
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
Stockton Ship Channel to French Camp Slough
2070 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

PLATE 29A



Peak Flows d/s Old River
1/2-ACE - 1,700 cfs
1/10-ACE - 7,700 cfs
1/25-ACE - 9,000 cfs
1/50-ACE - 10,000 cfs
1/100-ACE - 13,800 cfs
1/200-ACE - 15,700 cfs
1/500-ACE - 16,900 cfs

Peak Flows d/s FCS
1/2-ACE - 2,270 cfs
1/10-ACE - 9,780 cfs
1/25-ACE - 11,040 cfs
1/50-ACE - 12,200 cfs
1/100-ACE - 15,800 cfs
1/200-ACE - 27,900 cfs
1/500-ACE - 34,600 cfs

—— Top of Levee - RB
-·-·- Top of Levee - LB
- -+- - Stream Junction
●—— Bridge (Max. Low Chord; Min. OT Elev.)

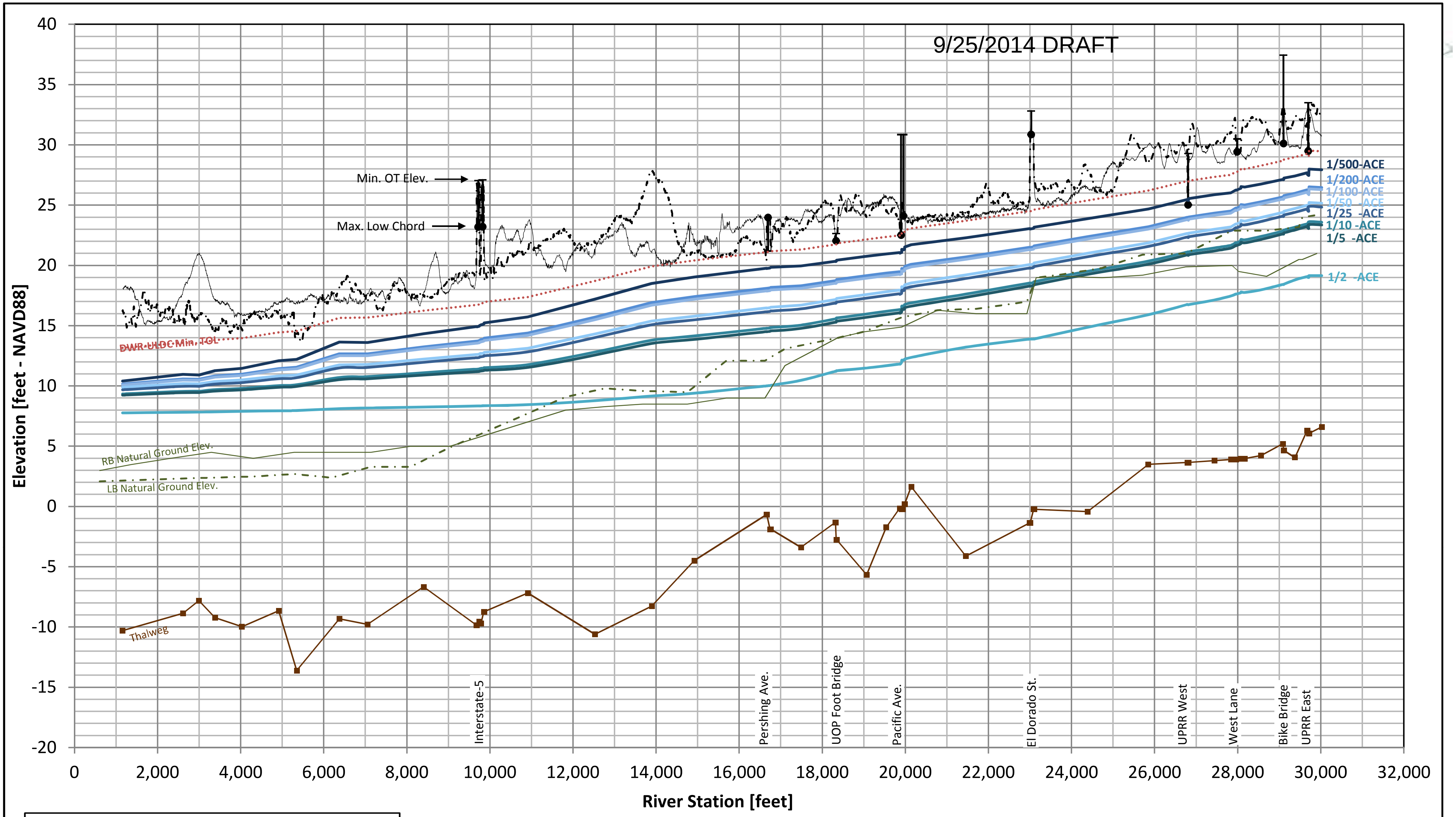
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

San Joaquin River Main Stem
French Camp Slough to Old River
2070 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014

FIGURE 29B



Peak Flows in Reach

1/2-ACE	3,850 cfs	1/50-ACE	12,850 cfs
1/5-ACE	9,500 cfs	1/100-ACE	15,360 cfs
1/10-ACE	9,860 cfs	1/200-ACE	15,750 cfs
1/25-ACE	12,280 cfs	1/500-ACE	19,130 cfs

─── Top of Levee - RB

-·-·- Top of Levee - LB

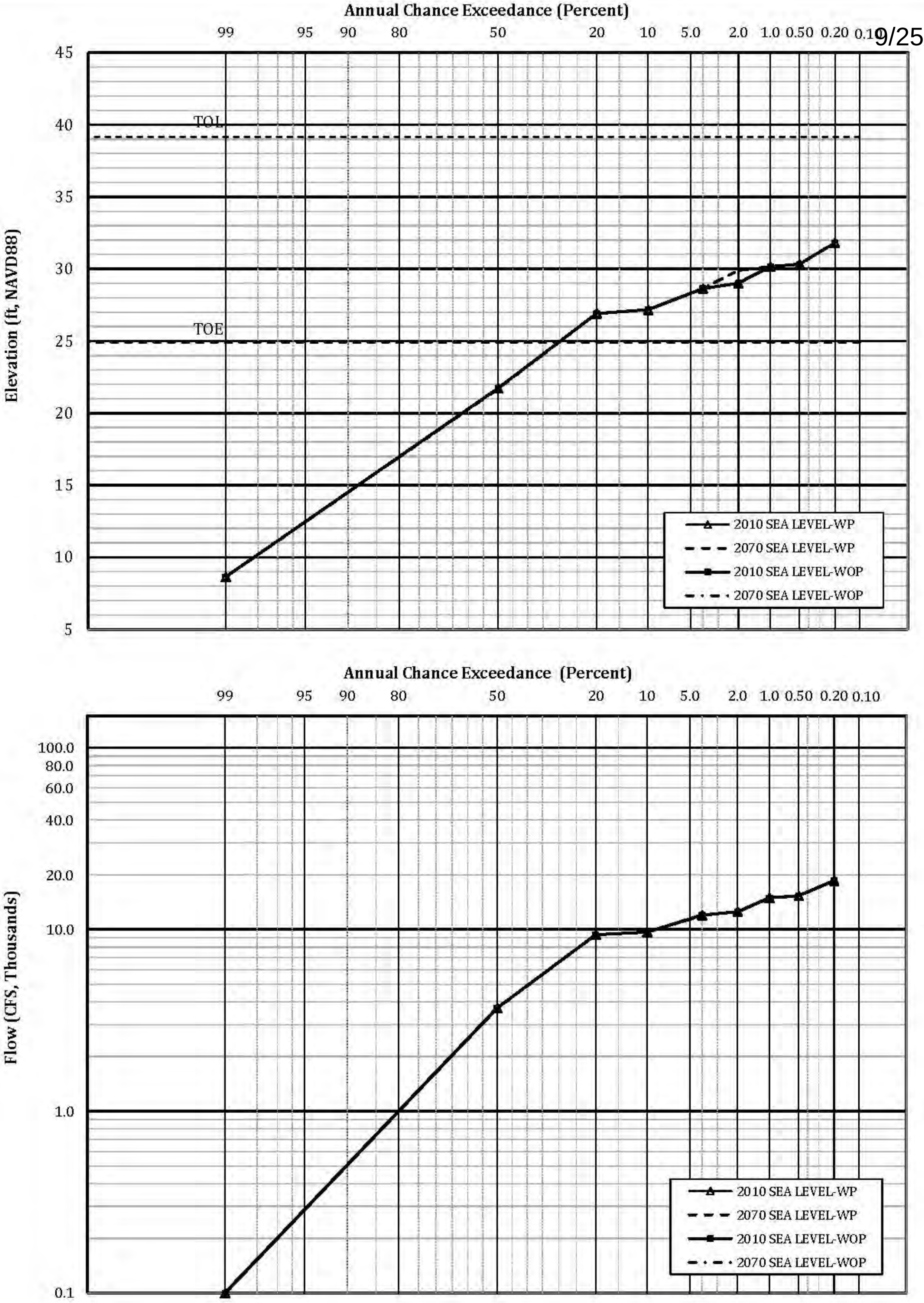
●──┘ Bridge (Max. Low Chord; Min. OT Elev.)

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

Calaveras River
Downstream of Diverting Canal
2070 Without-Project Water Surface Profiles

United States Army Corps of Engineers
Sacramento District

JUN 2014 PLATE 30



NOTES:

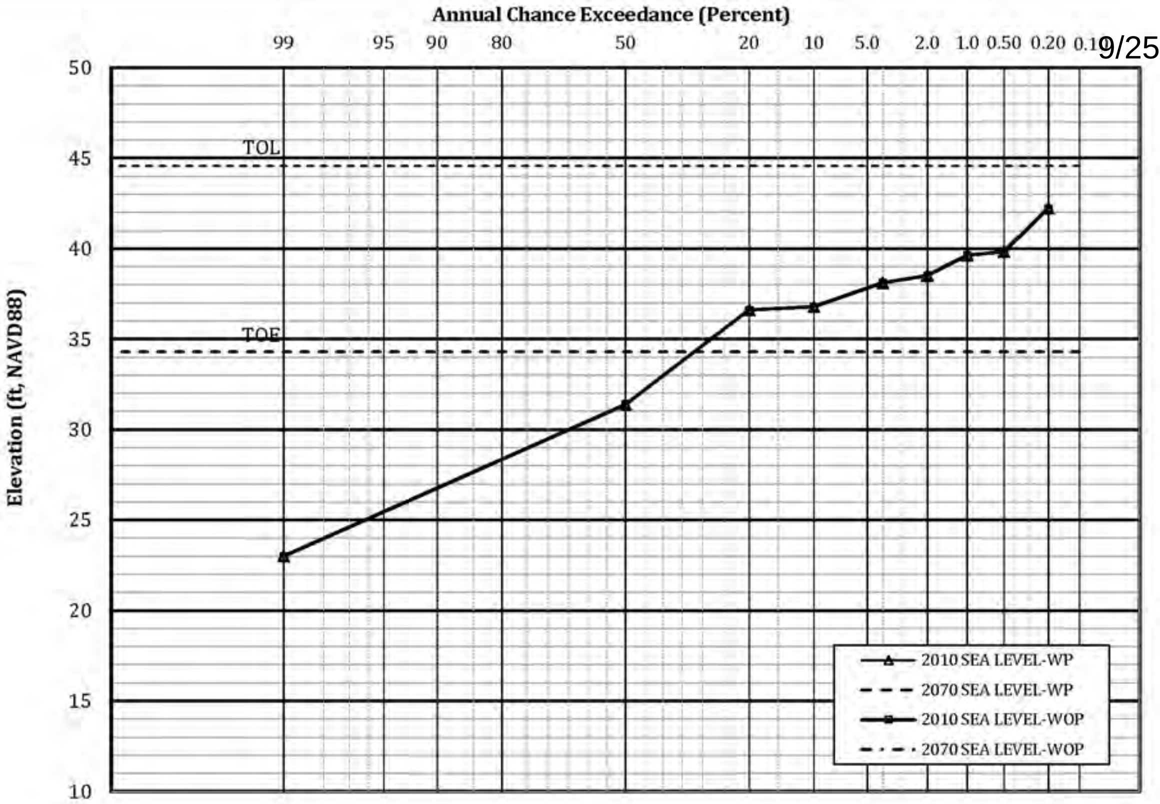
-Curves based on without- and with-project HEC-RAS simulations, where:
Without-Project (WOP) = No Action Alternative
With-Project (WP) = RD17 levee heights adjusted, where necessary,
to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)
-Stages and Flows for Index Point F-SL1 are from Stockton
Diverting Canal at RS 4644
-TOL - Top of levee elevation
-TOE - Approx. elevation of natural floodplain adjacent to levee

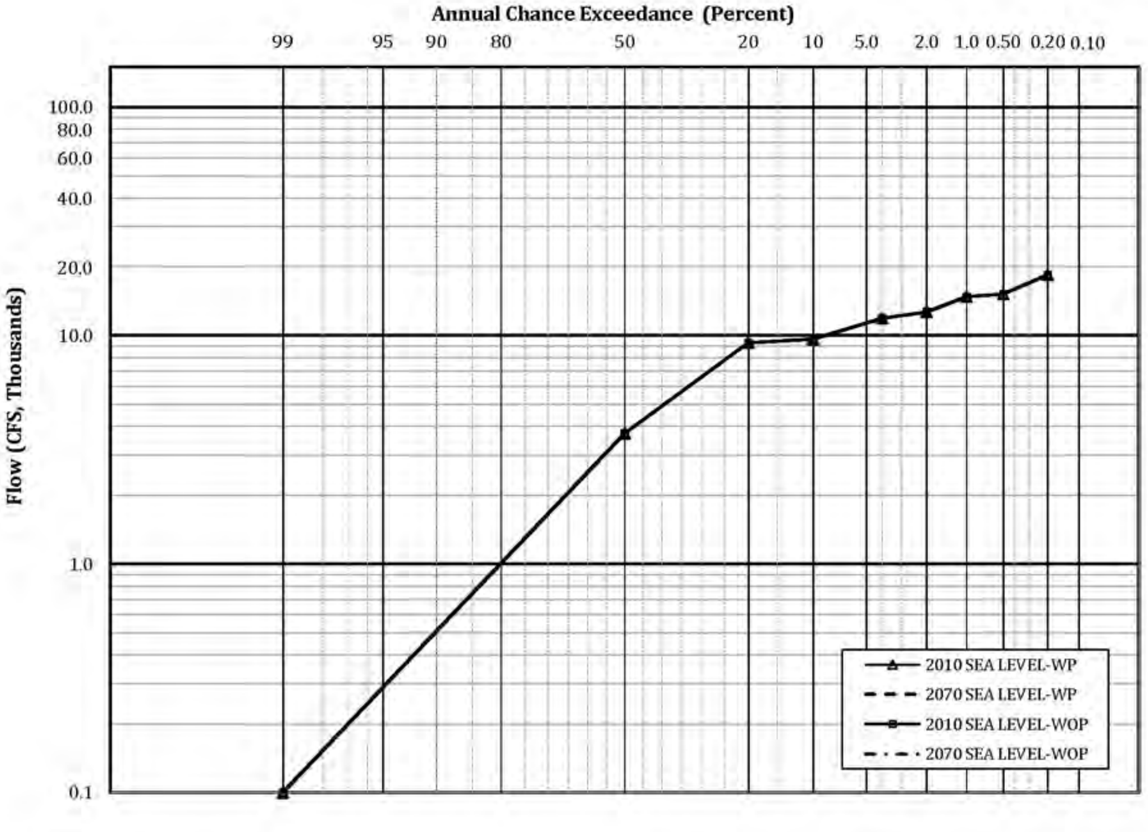
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-SL1

United States Army Corps of Engineers
Sacramento District

9/25/2014 DRAFT





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:
Without-Project (WOP) = No Action Alternative
With-Project (WP) = RD17 levee heights adjusted, where necessary, to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Stages and Flows for Index Point F-SL2 are from Stockton Diverting Canal at RS 17391

-TOL - Top of levee elevation

-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

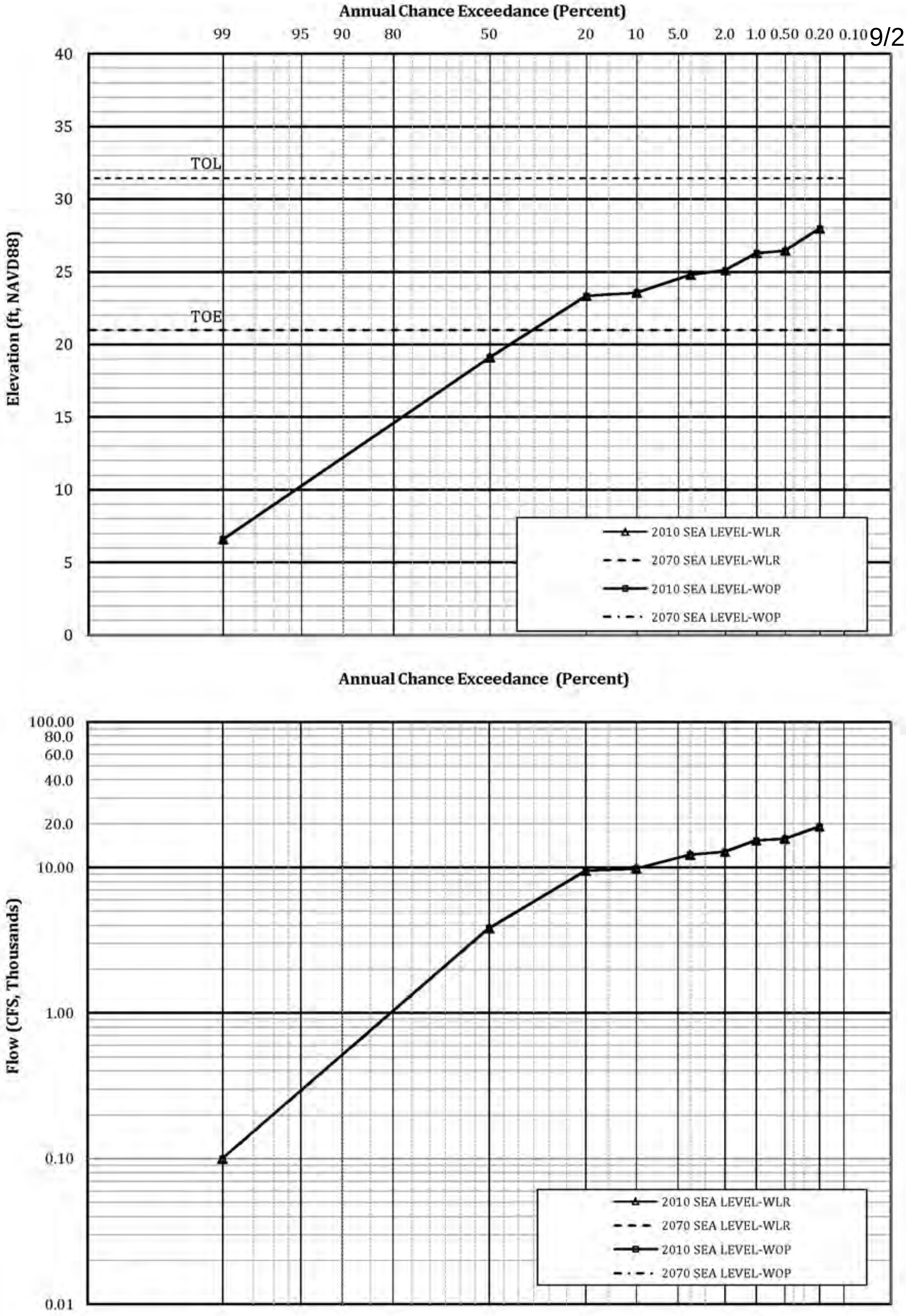
STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-SL2

United States Army Corps of Engineers
Sacramento District

JUN 2014

FIGURE 31B

9/25/2014 DRAFT



NOTES:

Curves based on HEC-RAS simulations

Without-Project (WOP) = No Action Alternative

With-Levee-Raise (WLR) = All levee heights in study area increased to meet California SB5 design requirement for 0.5% (1/200) ACE Mean Stage plus 3 feet feet) with 2070 climate conditions.

Stages amd Flows based on Calaveras River at RS 30024.

Top of Levee = TOL

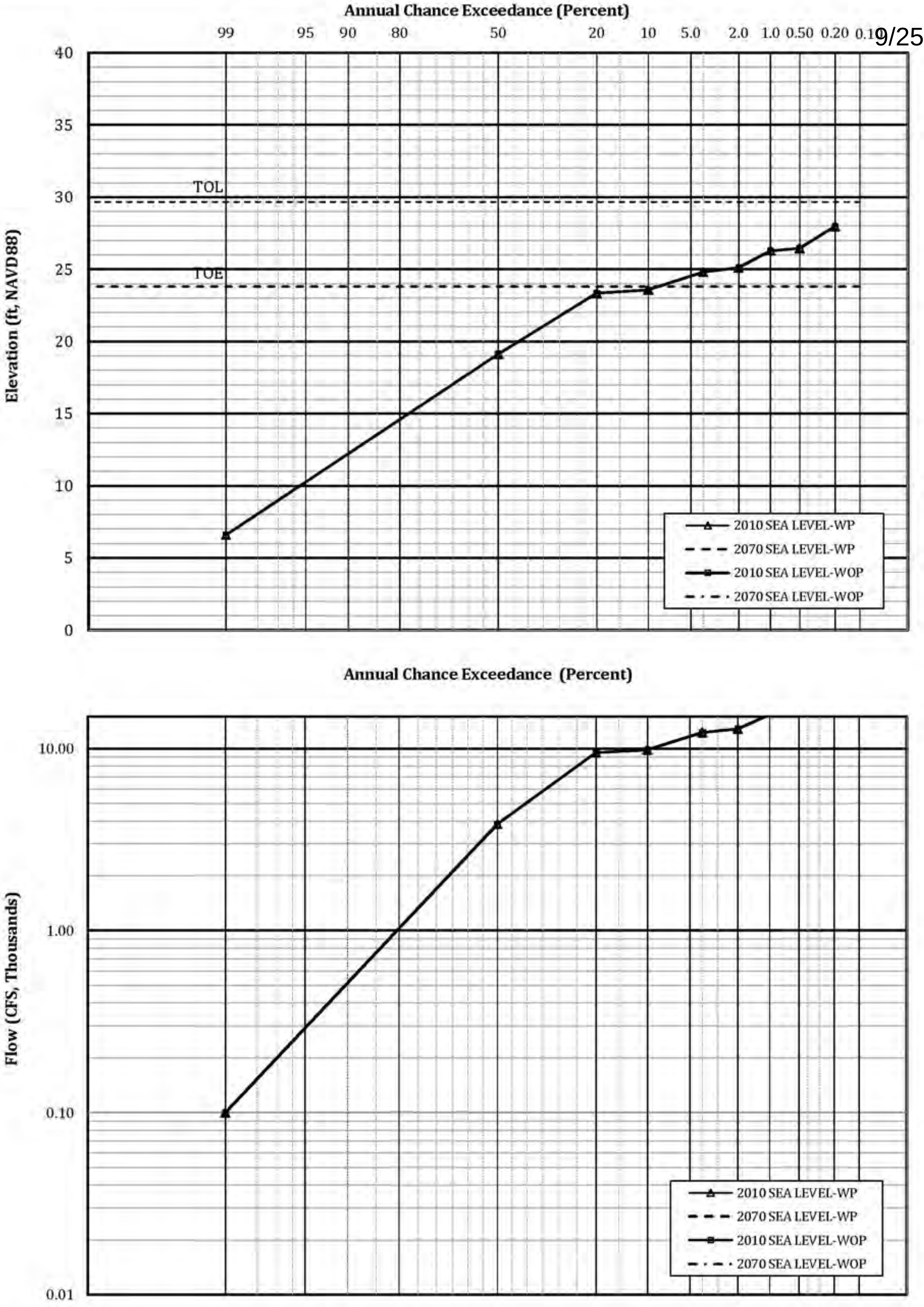
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
INDEX POINT F-CL2

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014

FIGURE 31C



NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:
Without-Project (WOP) = No Action Alternative
With-Project (WP) = RD17 levee heights adjusted, where necessary,
to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Stages and Flows for Index Point CR2 are from the Calaveras River at RS 30639

-TOL - Top of levee elevation

-TOE - Approx. elevation of natural floodplain adjacent to levee

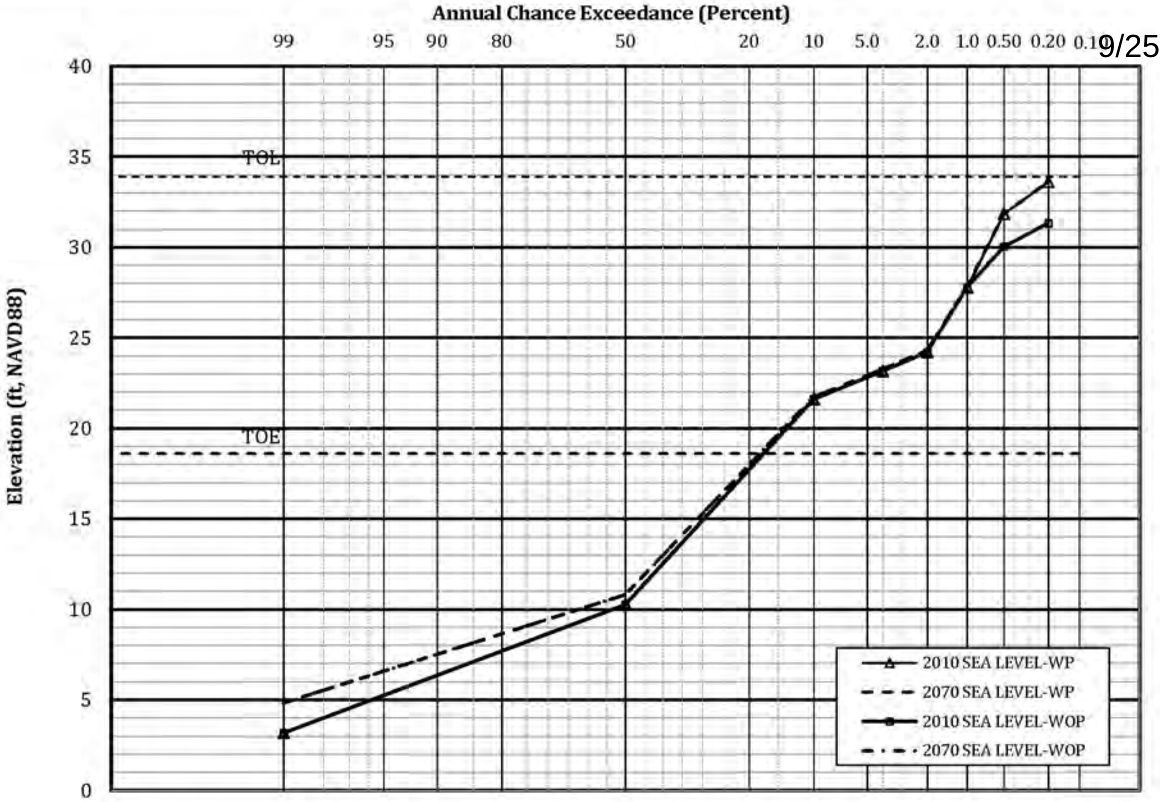
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

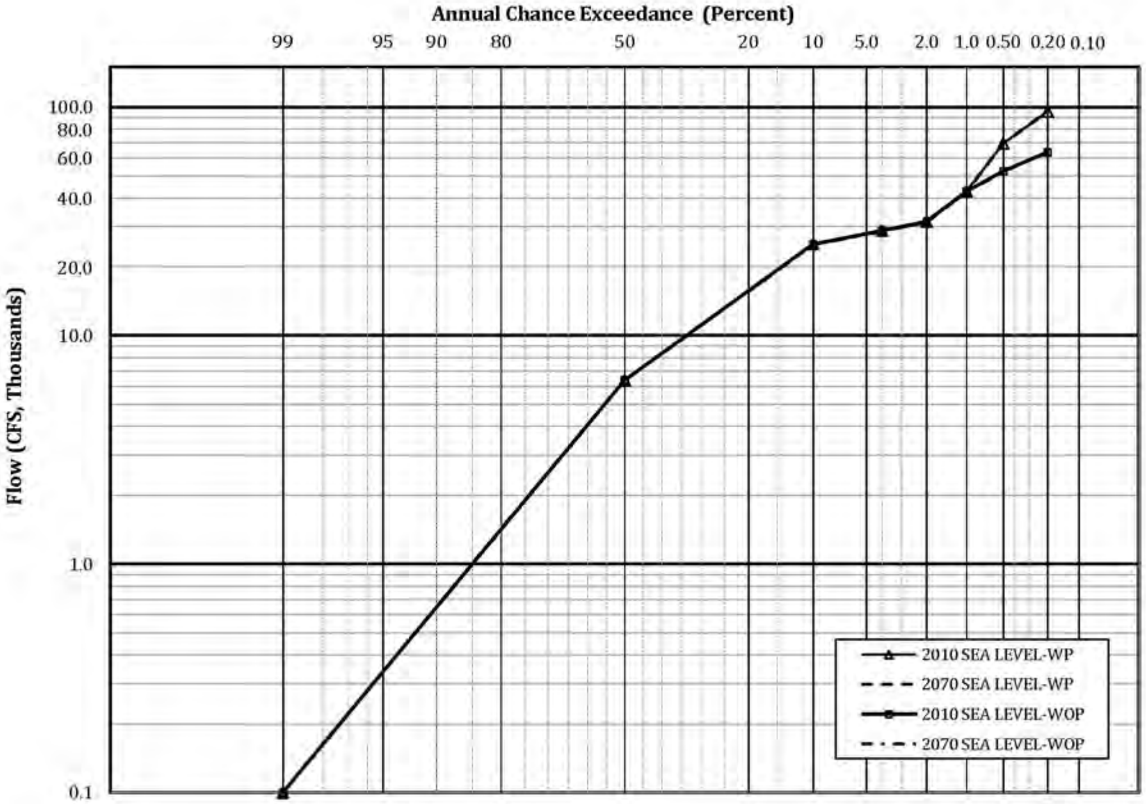
STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-CR2

United States Army Corps of Engineers
Sacramento District

JUN 24 2013

FIGURE 31D





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

With-Project (WP) = RD17 levee heights adjusted, where necessary, to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Index Point F-LR4 located on San Joaquin River at RS 57.05

-TOL - Top of levee elevation

-TOE - Approx. elevation of natural floodplain adjacent to levee

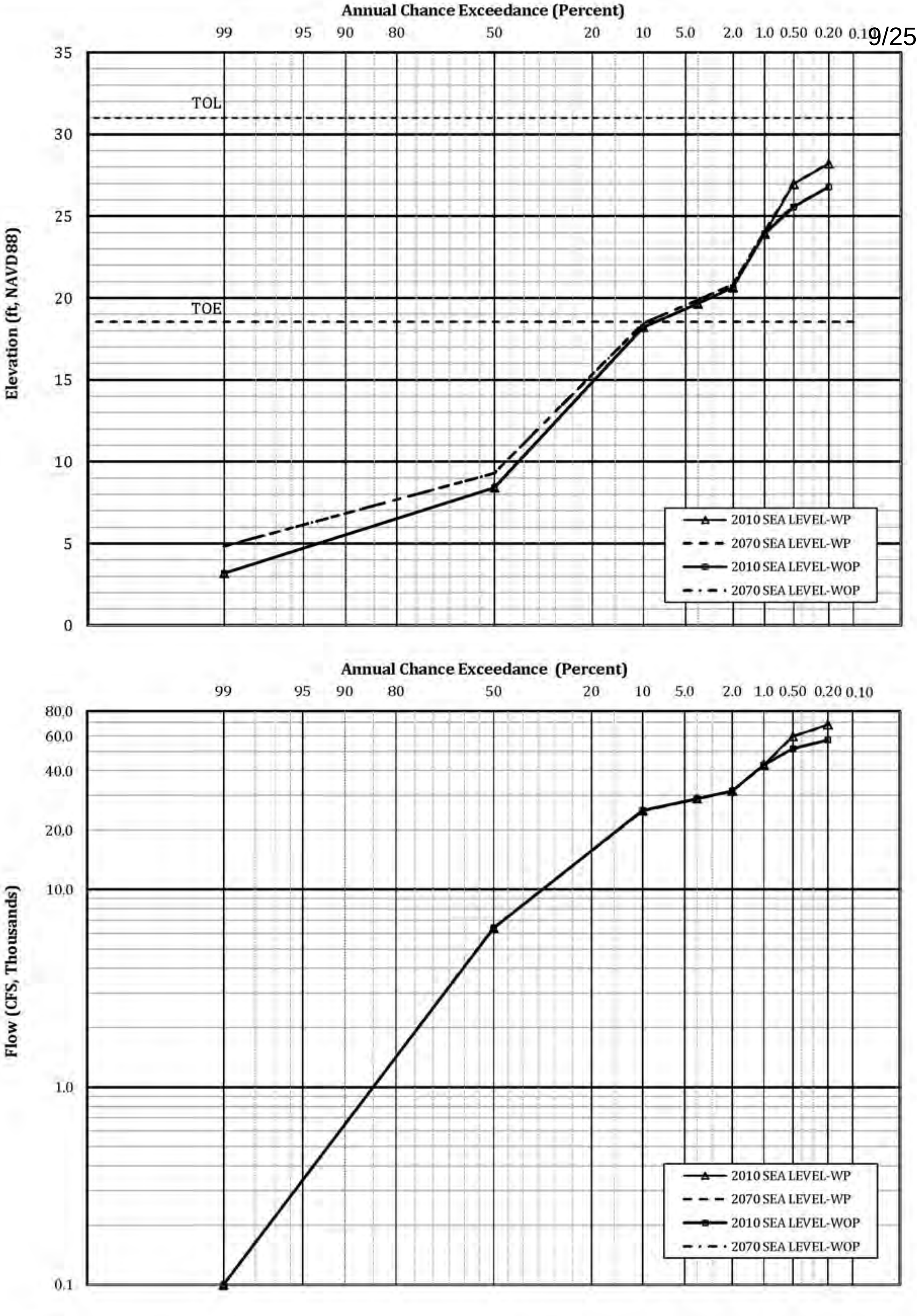
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-LR4

United States Army Corps of Engineers
Sacramento District

JUN 2014

FIGURE 31E



NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

With-Project (WP) = RD17 levee heights adjusted, where necessary, to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Index Point F-LR3 located on San Joaquin River at RS 53.89

-TOL - Top of levee elevation

-TOE - Approx. elevation of natural floodplain adjacent to levee

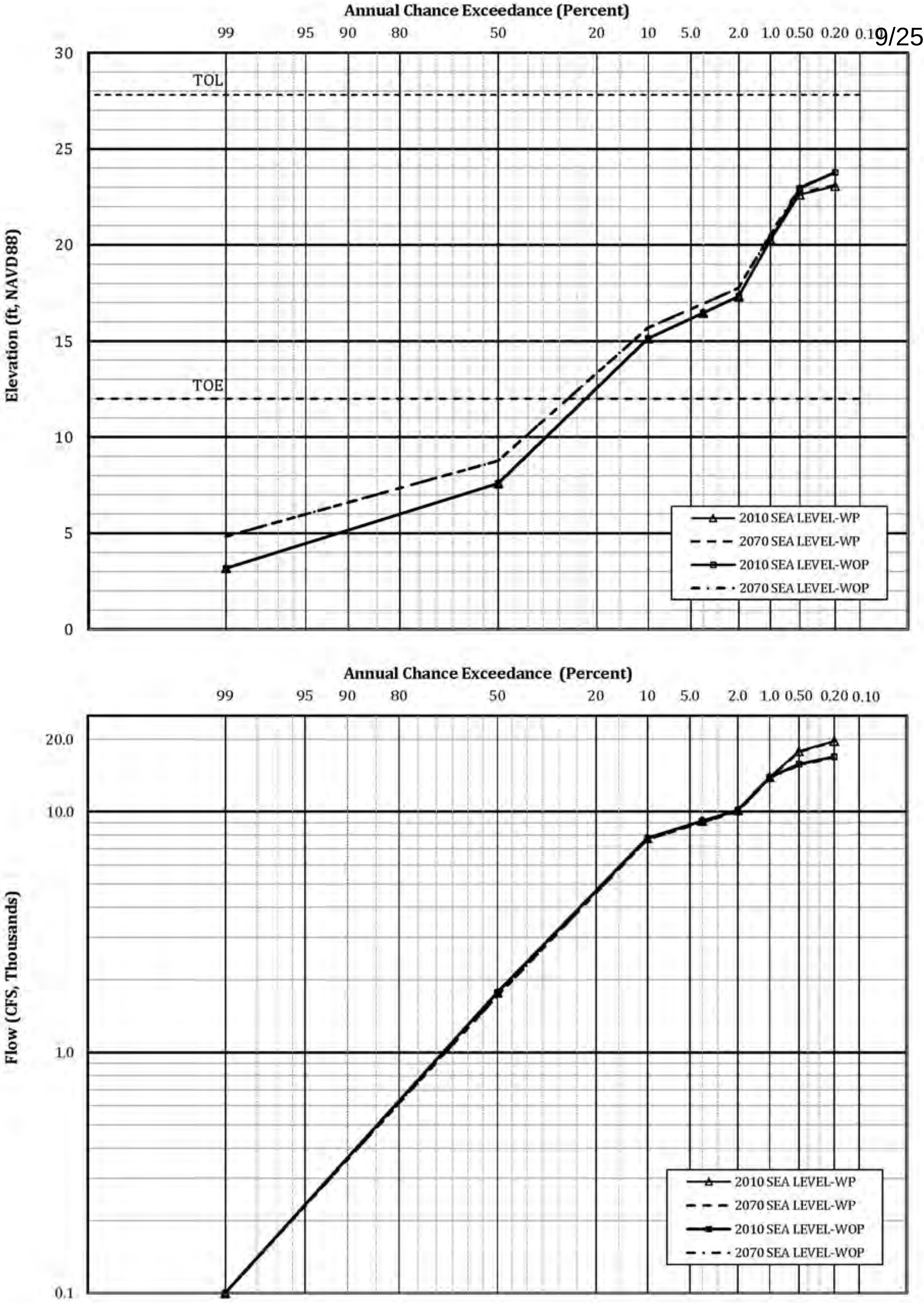
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-LR3

United States Army Corps of Engineers
Sacramento District

JUN 2014

FIGURE 31F



NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:
Without-Project (WOP) = No Action Alternative
With-Project (WP) = RD17 levee heights adjusted, where necessary,
to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)
-Index Point F-LR2 located on San Joaquin River at RS 48.89
-TOL - Top of levee elevation
-TOE - Approx. elevation of natural floodplain adjacent to levee

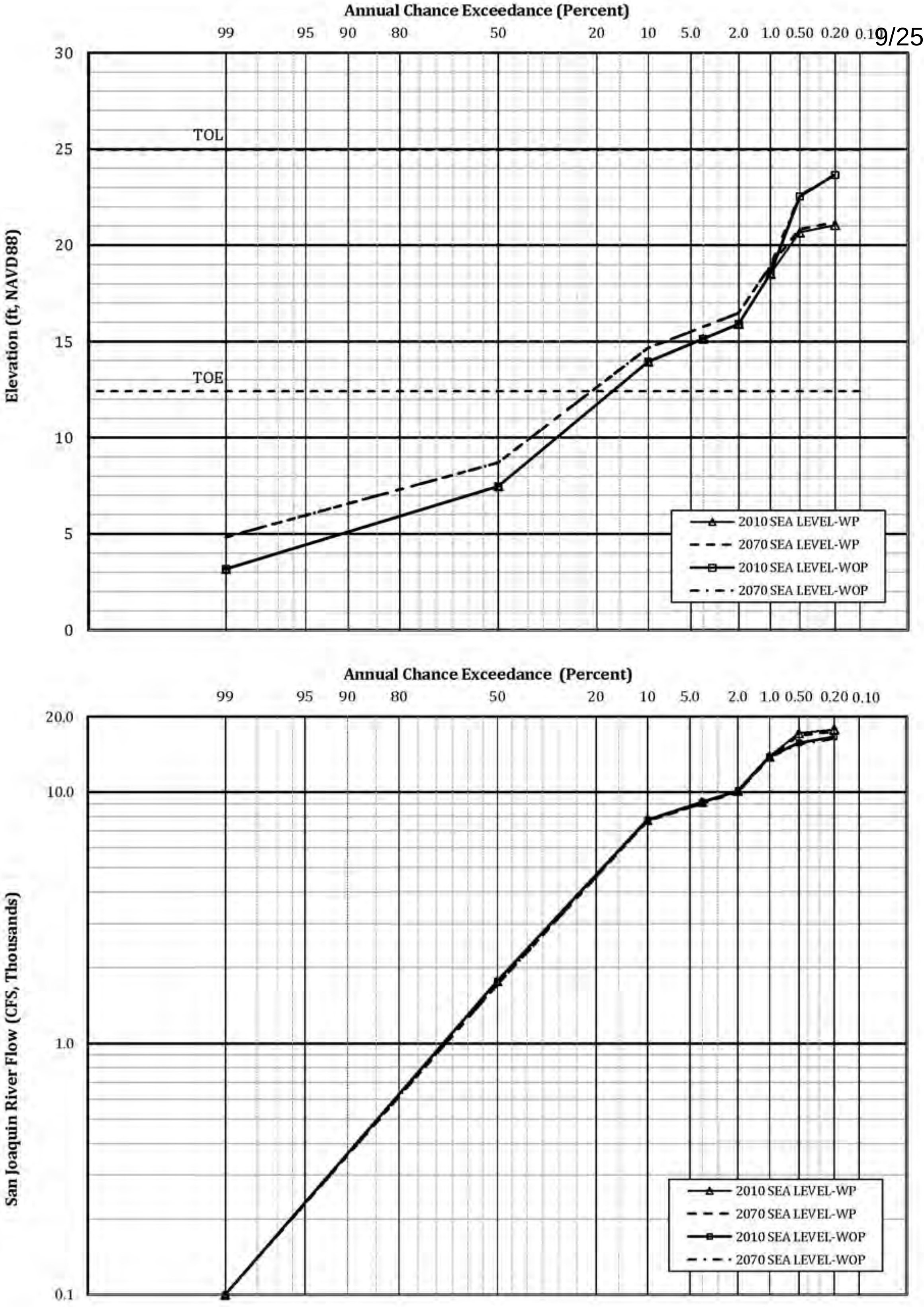
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-LR2

United States Army Corps of Engineers
Sacramento District

JUN 2014

9/25/2014 DRAFT



NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

With-Project (WP) = RD17 levee heights adjusted, where necessary, to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Index Point F-LR1 based on conditions on San Joaquin River at RS 46.61

-TOL - Top of levee elevation

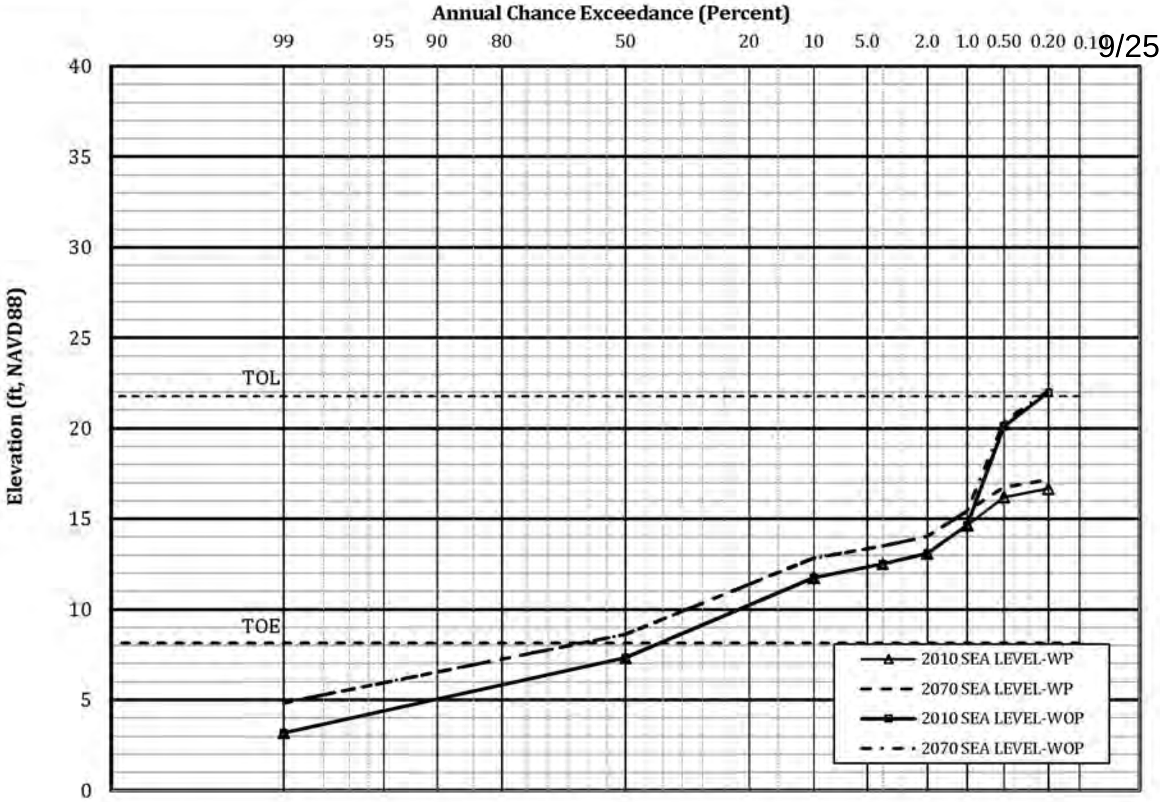
-TOE - Approx. elevation of natural floodplain adjacent to levee

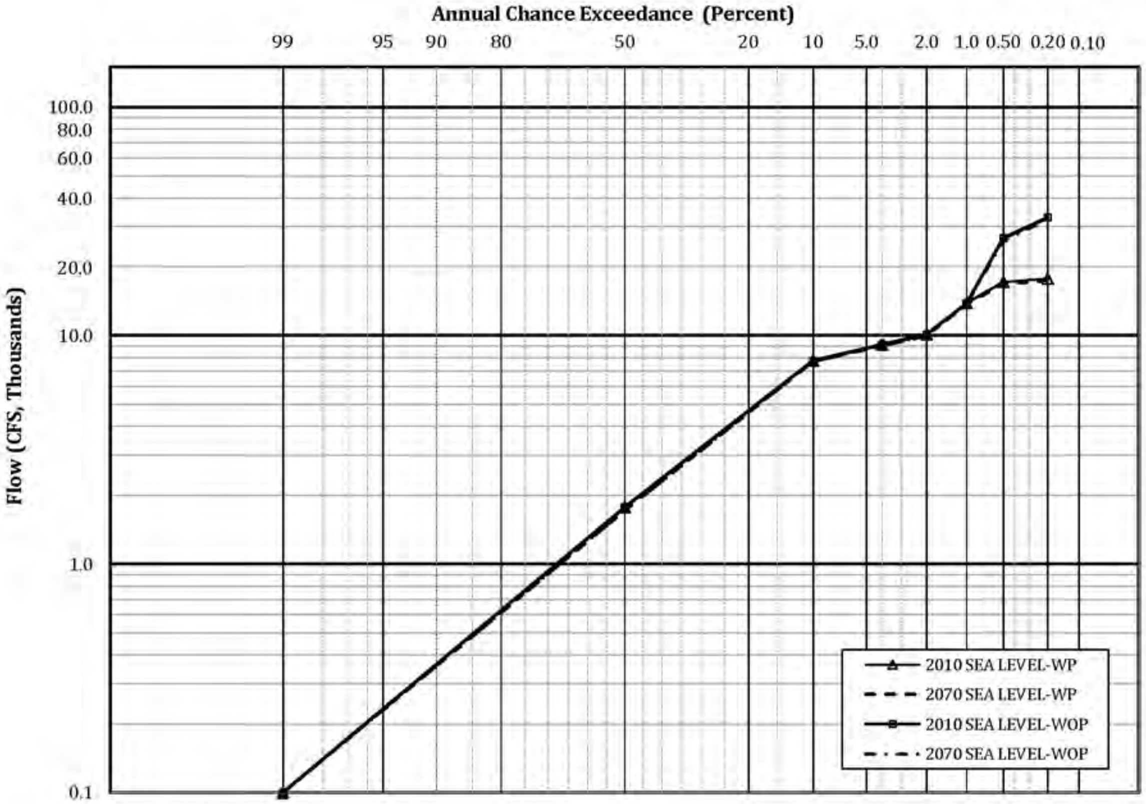
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-LR1

United States Army Corps of Engineers
Sacramento District

JUN 2014





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

With-Project (WP) = RD17 levee heights adjusted, where necessary, to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Stages and Flows for Index Point FL1 are from San Joaquin River at RS 43.1

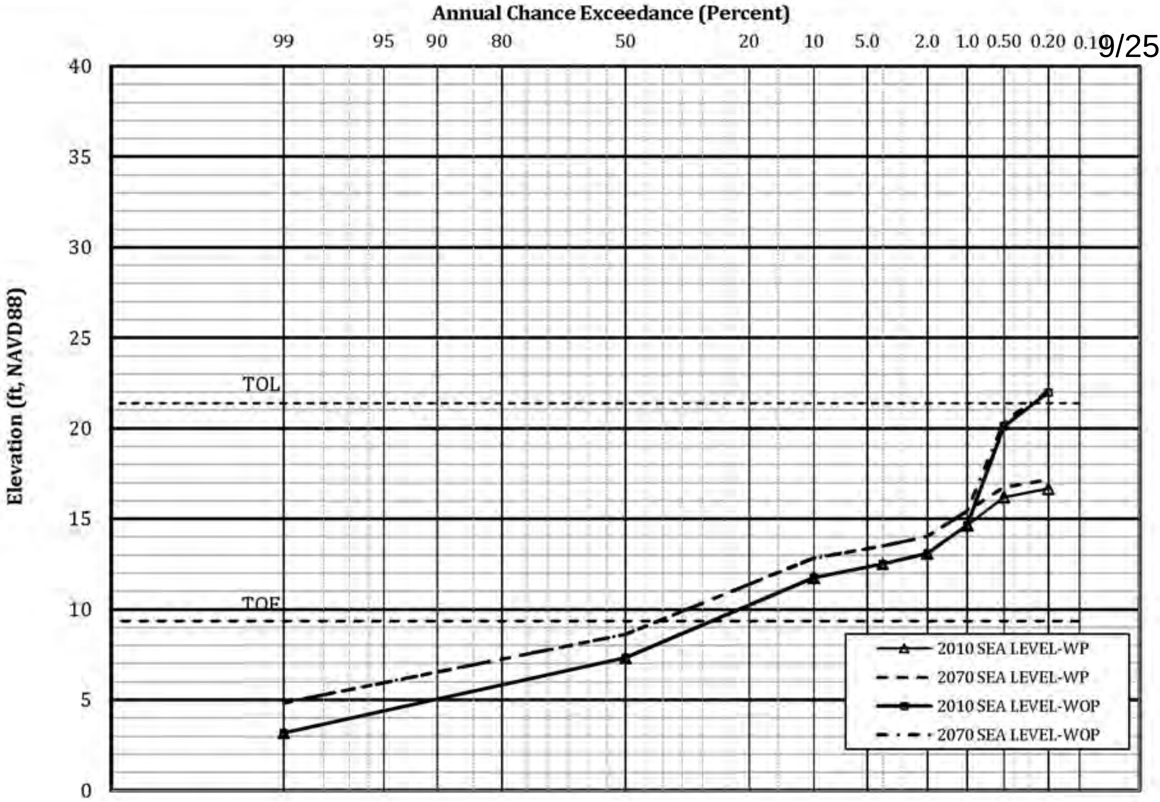
-TOL - Top of levee elevation

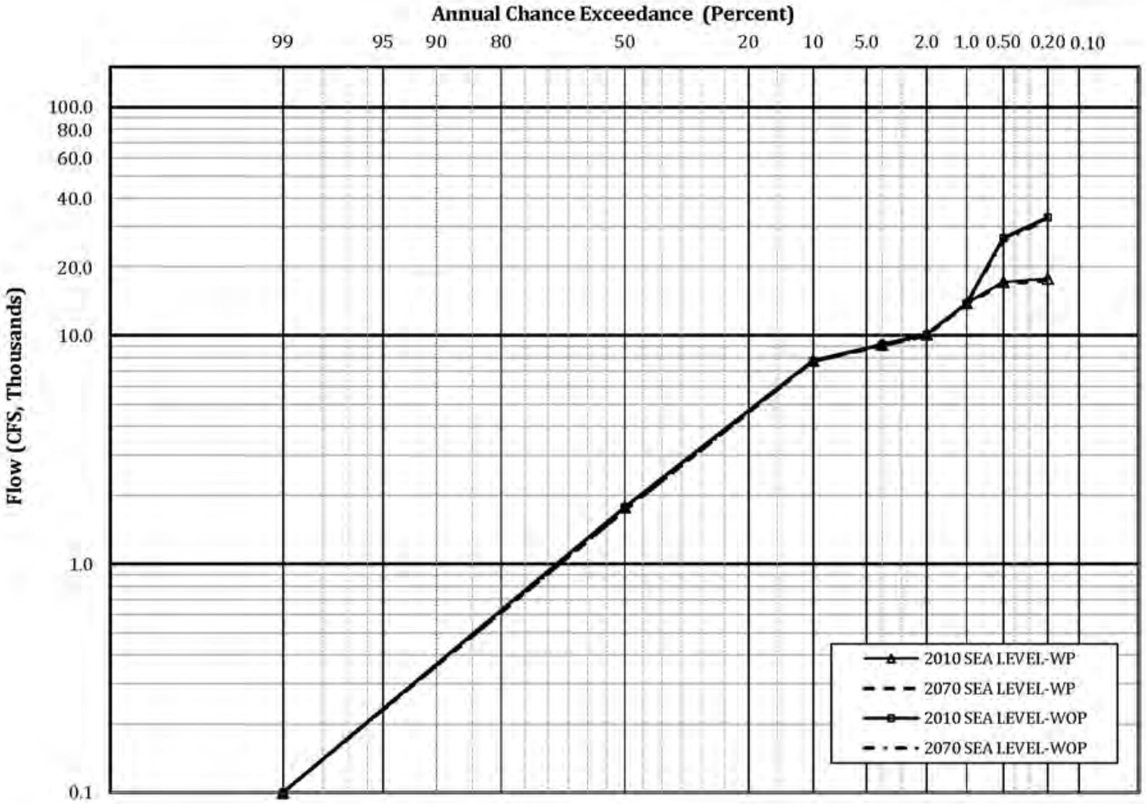
-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
FR1

United States Army Corps of Engineers
Sacramento District





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

With-Project (WP) = RD17 levee heights adjusted, where necessary, to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Stages and Flows for Index Point FL1 are from San Joaquin River at RS 43.1

-TOL - Top of levee elevation

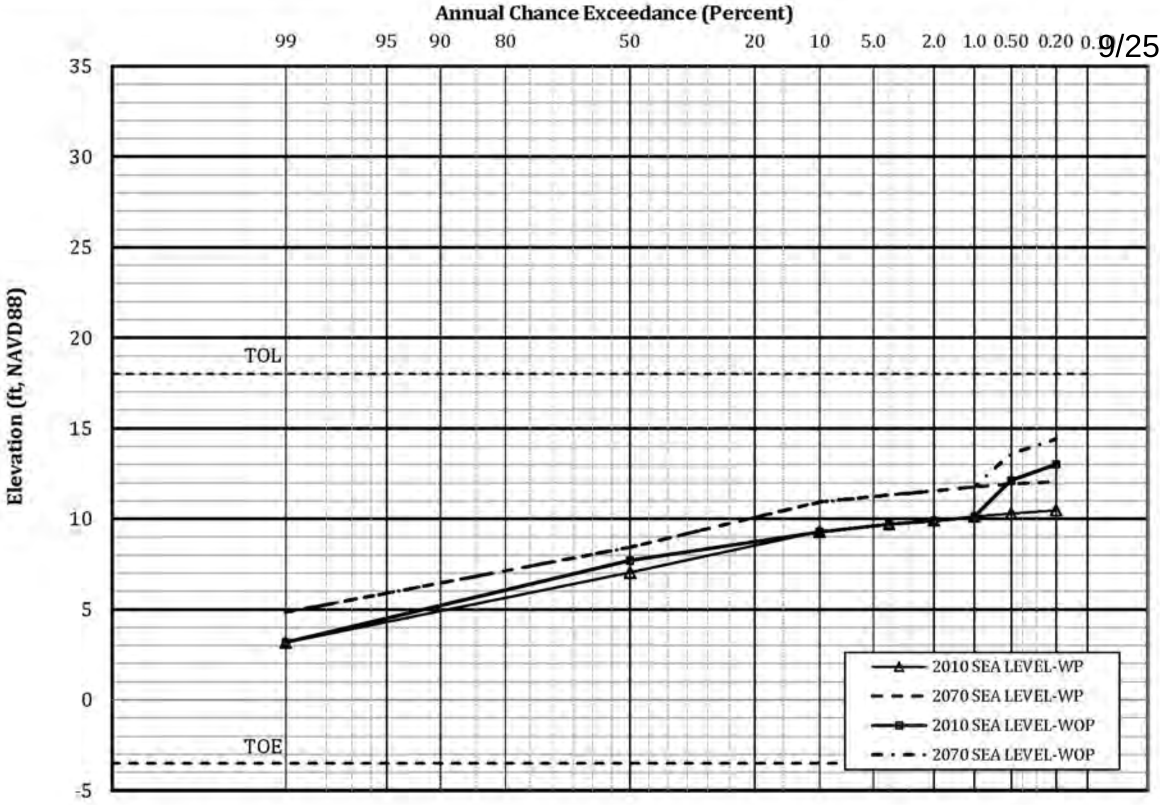
-TOE - Approx. elevation of natural floodplain adjacent to levee

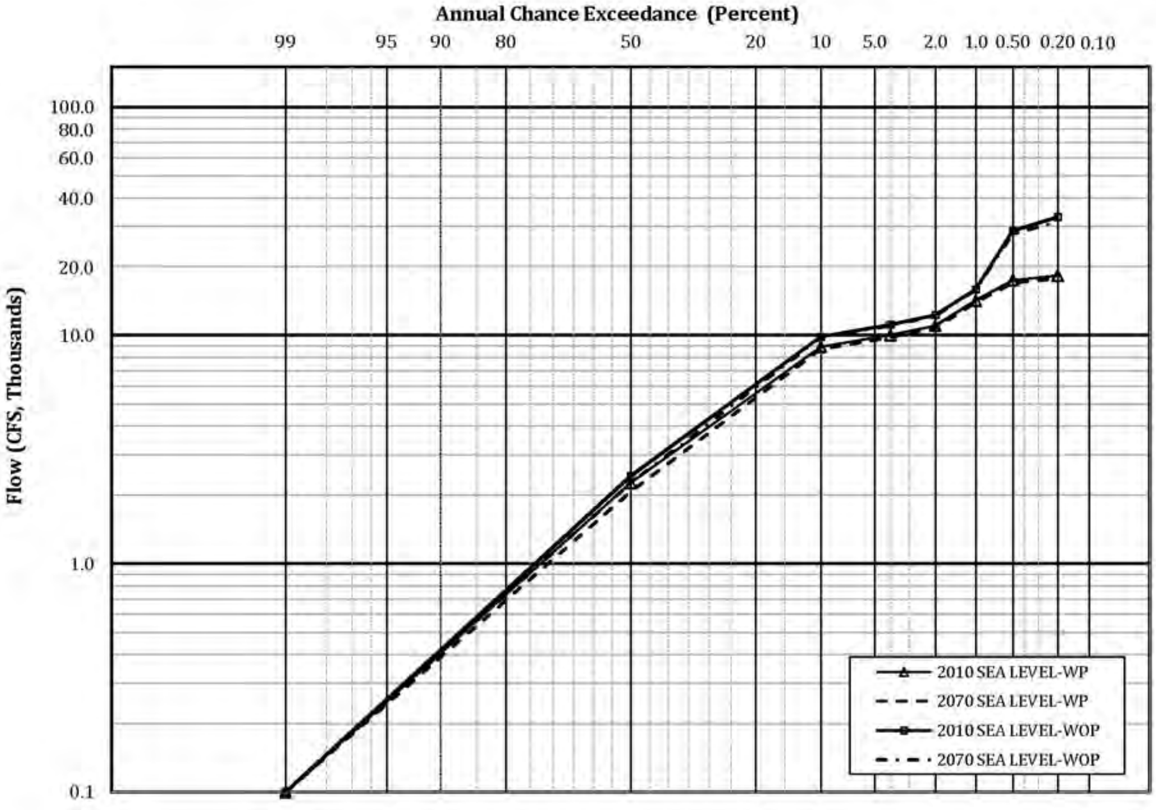
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
FL1

United States Army Corps of Engineers
Sacramento District

JUN 2014





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

With-Project (WP) = RD17 levee heights adjusted, where necessary, to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Index Point is located on the Ship Channel at RS 37.83

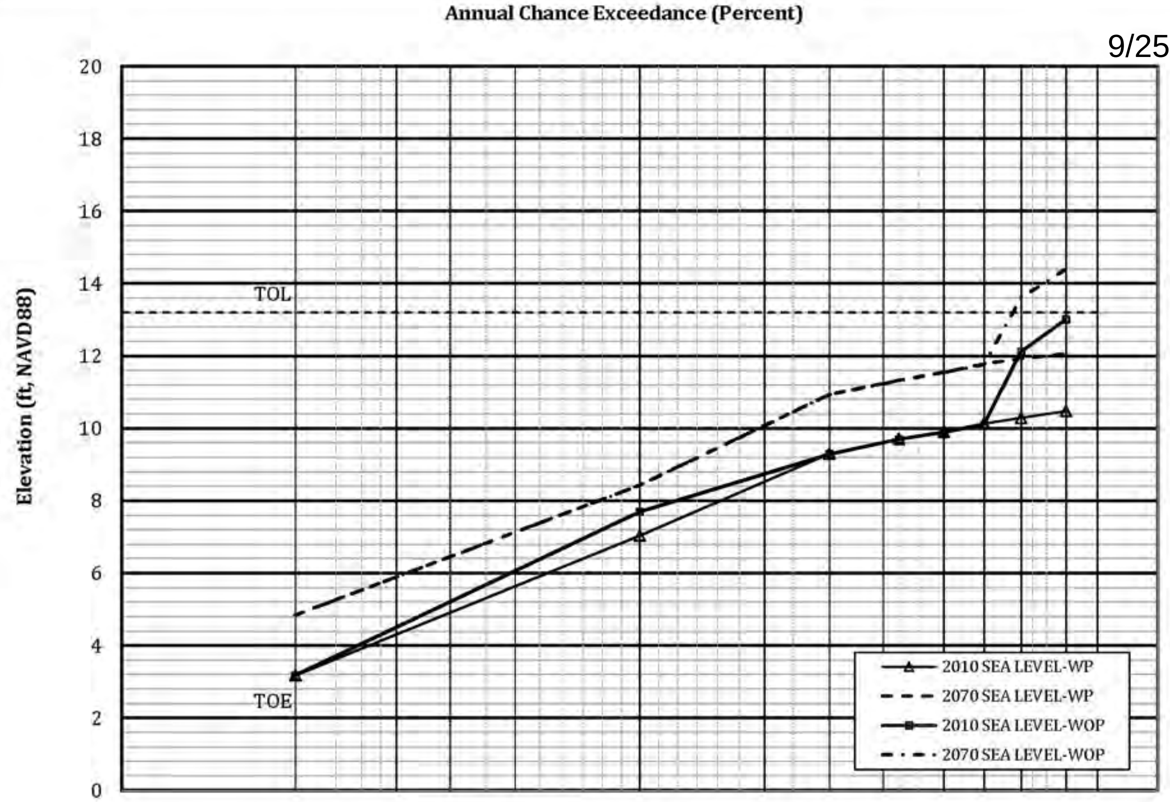
-TOE - Approx. elevation of natural floodplain adjacent to left bank levee levee toe estimated where land flattens closest to levee within Rough and Ready Island in line with HEC-RAS cross section

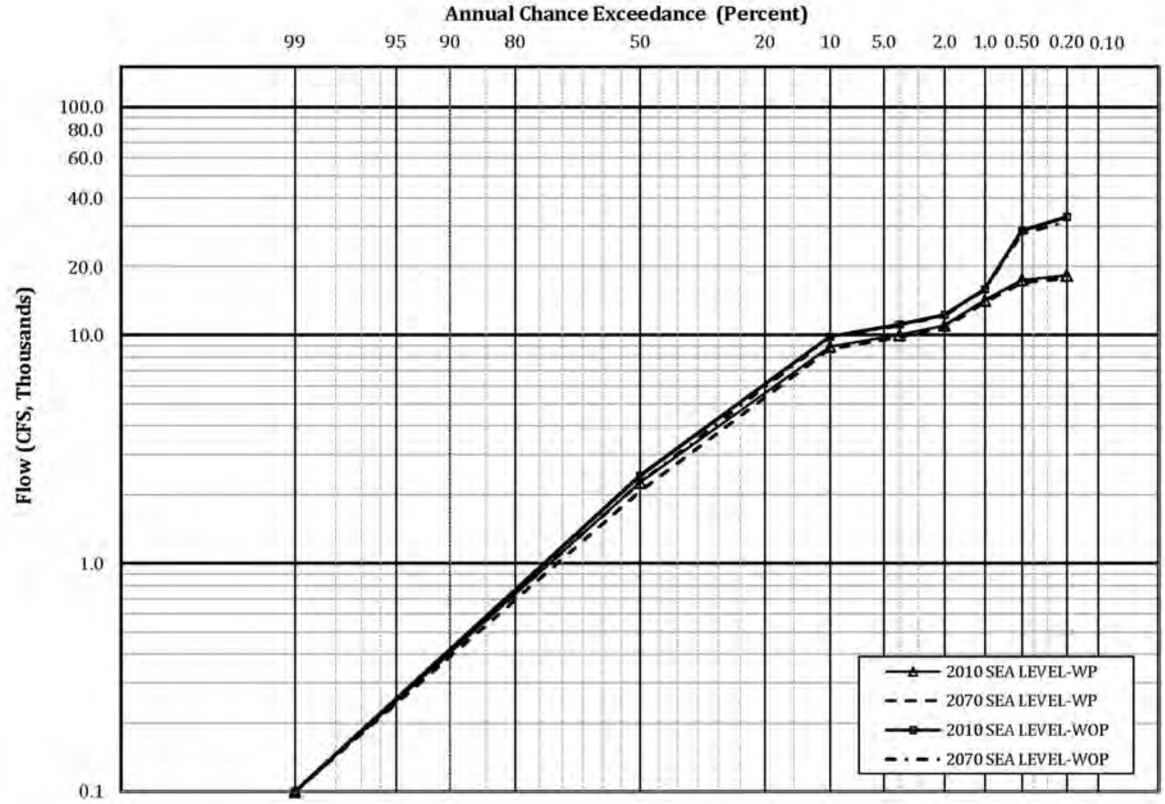
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-D-BS

United States Army Corps of Engineers
Sacramento District

9/25/2014 DRAFT





NOTES:

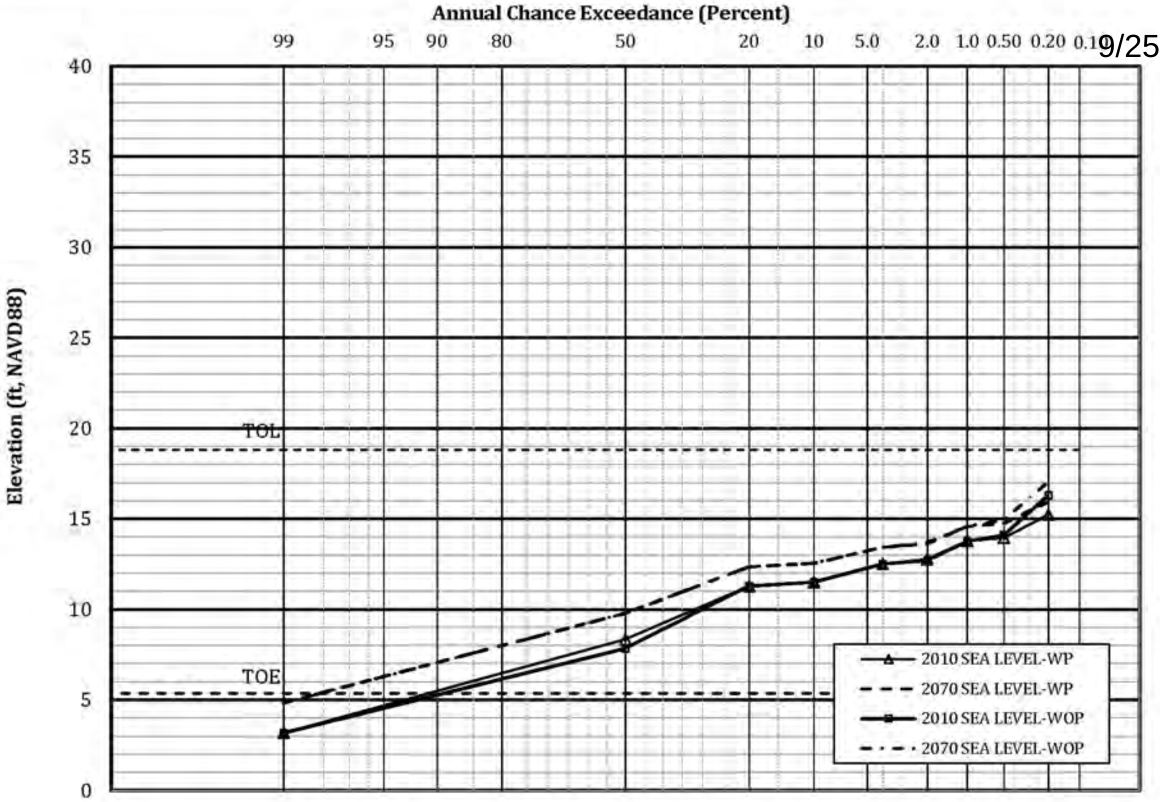
-Curves based on without- and with-project HEC-RAS simulations, where:
Without-Project (WOP) = No Action Alternative
With-Project (WP) = RD17 levee heights adjusted, where necessary,
to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)
-Stage and Flow data is from the Ship Channel at RS 37.83
-TOE - Approx. elevation of natural floodplain adjacent to left bank levee
levee toe estimated where land flattens closest to levee within Rough
and Ready Island in line with HEC-RAS cross section

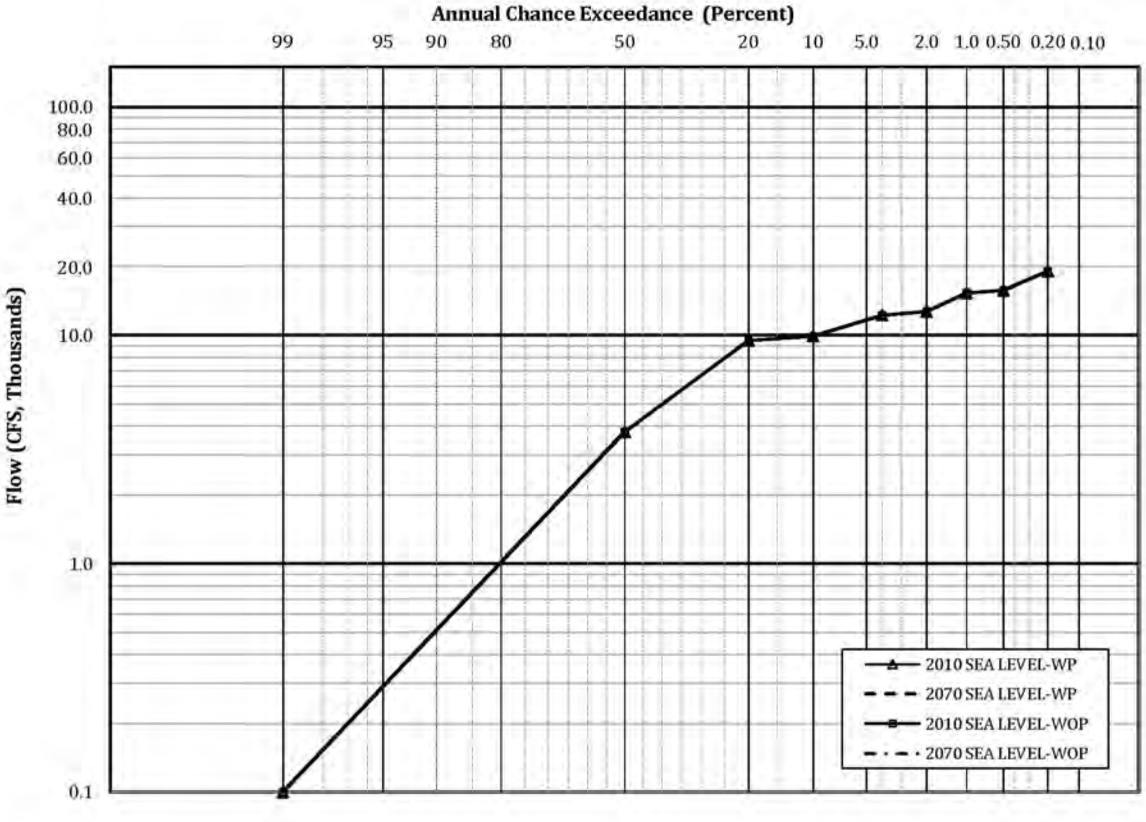
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-D3

United States Army Corps of Engineers
Sacramento District

JUN 2014





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:
Without-Project (WOP) = No Action Alternative
With-Project (WP) = RD17 levee heights adjusted, where necessary,
to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Stages and Flows for Index Point F-D4 are from the Calaveras River at RS 9862

-TOL - Top of levee elevation

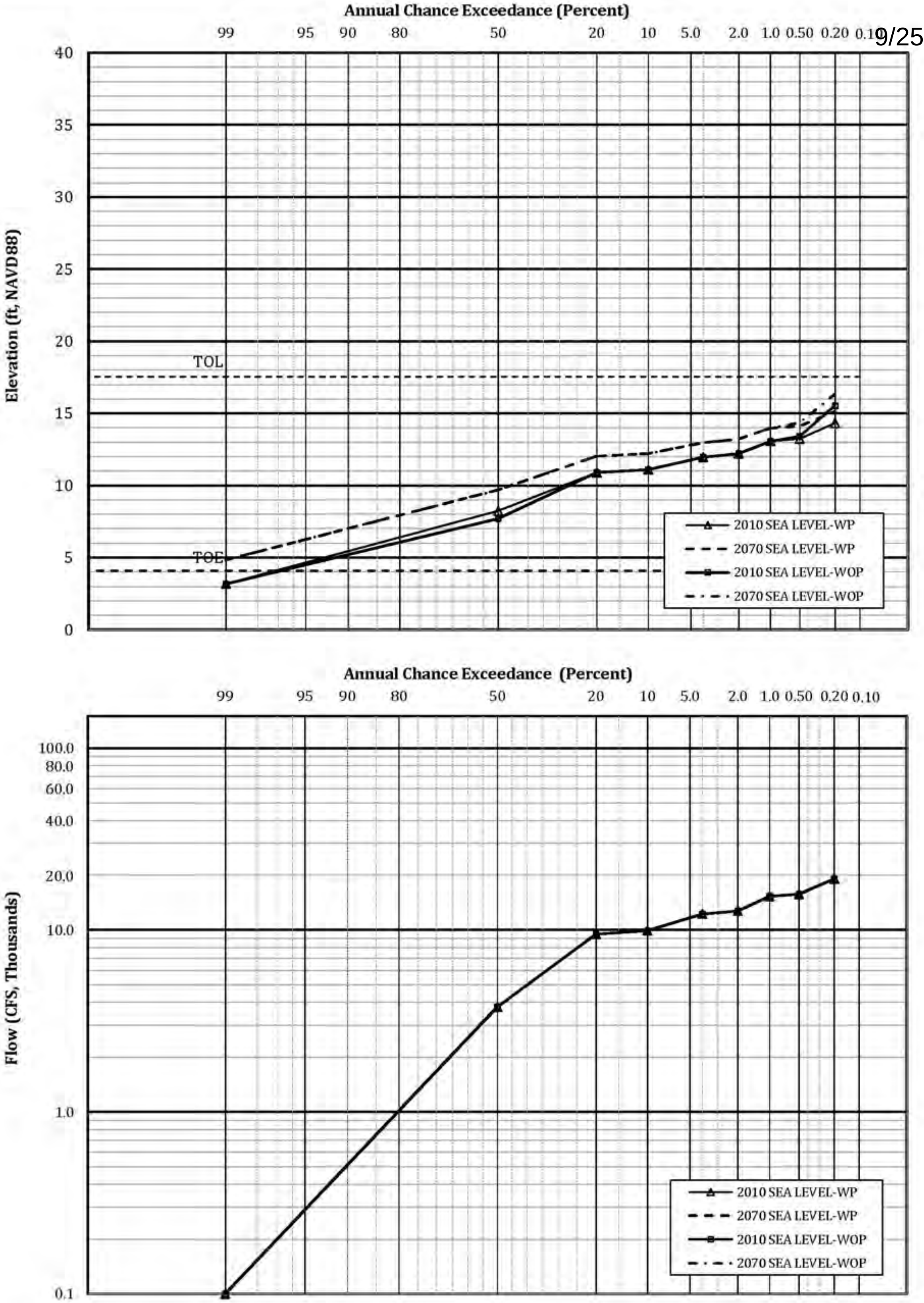
-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-D4

United States Army Corps of Engineers
Sacramento District

JUN 2014



NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

With-Project (WP) = RD17 levee heights adjusted, where necessary, to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Stages and Flows for Index Point F-D5 are from the Calaveras River at RS 8401

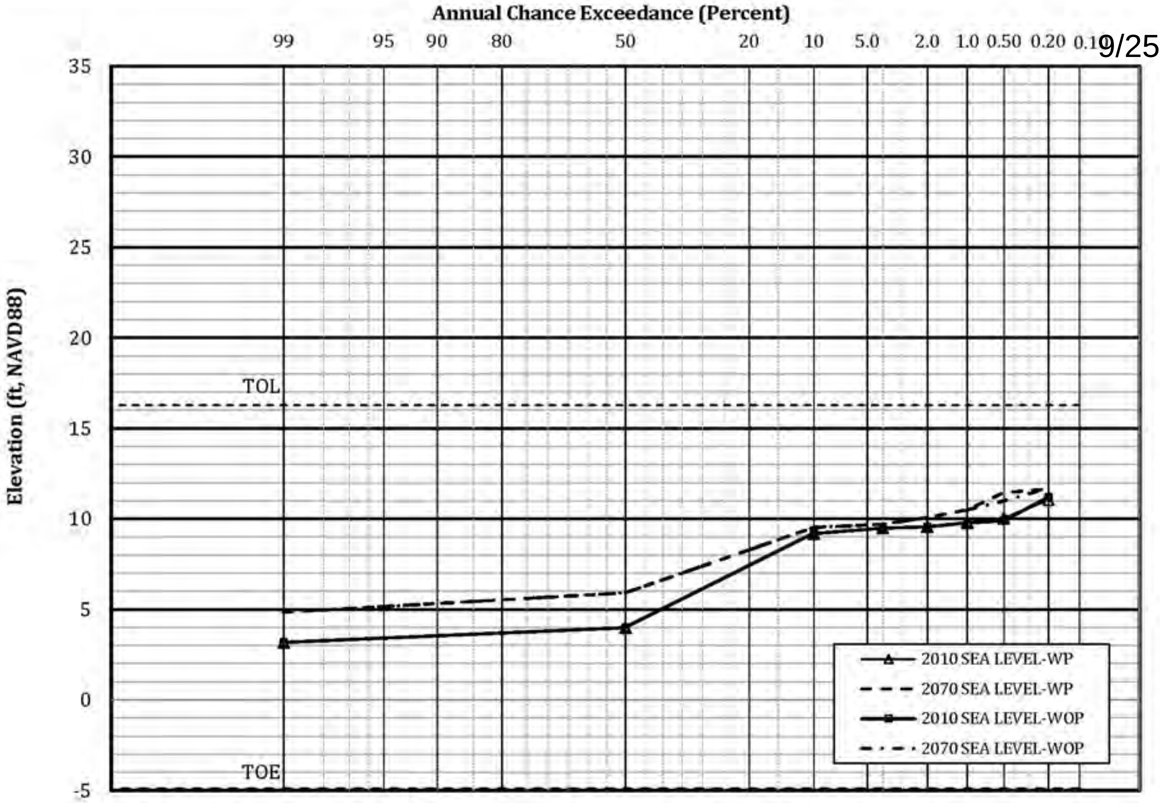
-TOL - Top of levee elevation

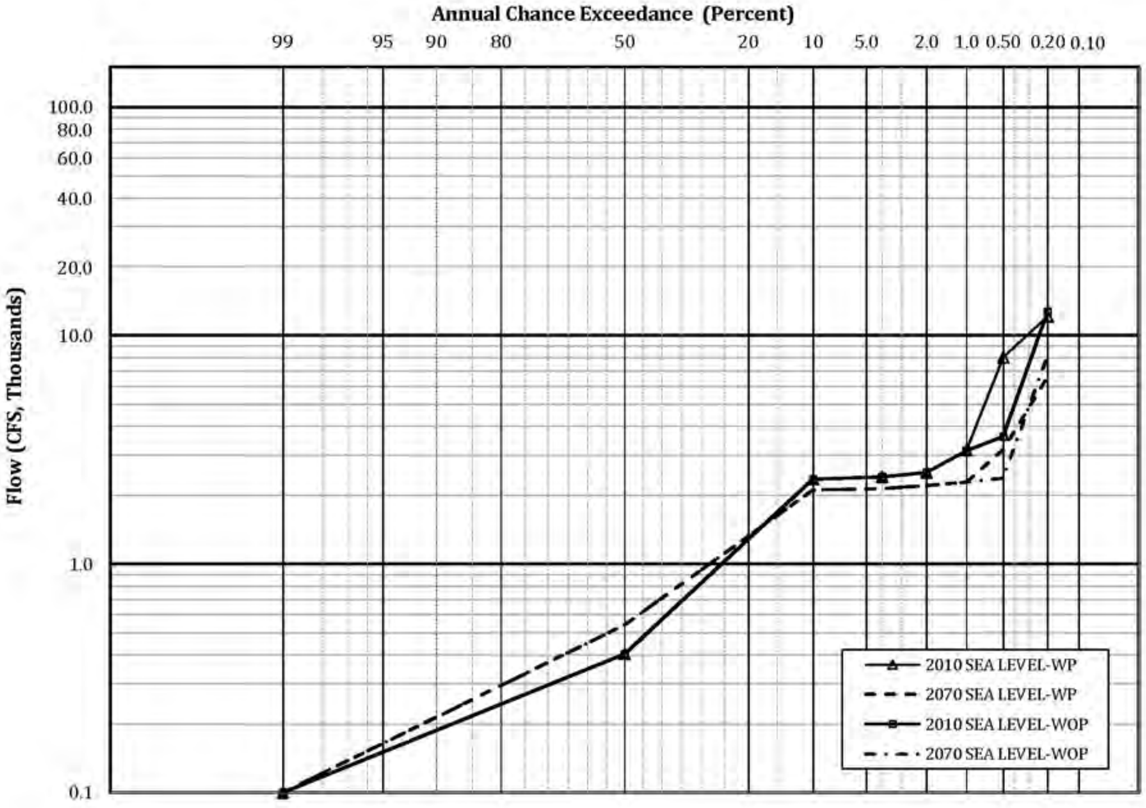
-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
F-D5

United States Army Corps of Engineers
Sacramento District





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

With-Project (WP) = RD17 levee heights adjusted, where necessary, to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Stages and Flows for Index Point Middle River @ Borden Hwy are from Middle River at RS 15.923

-TOL - Top of levee elevation

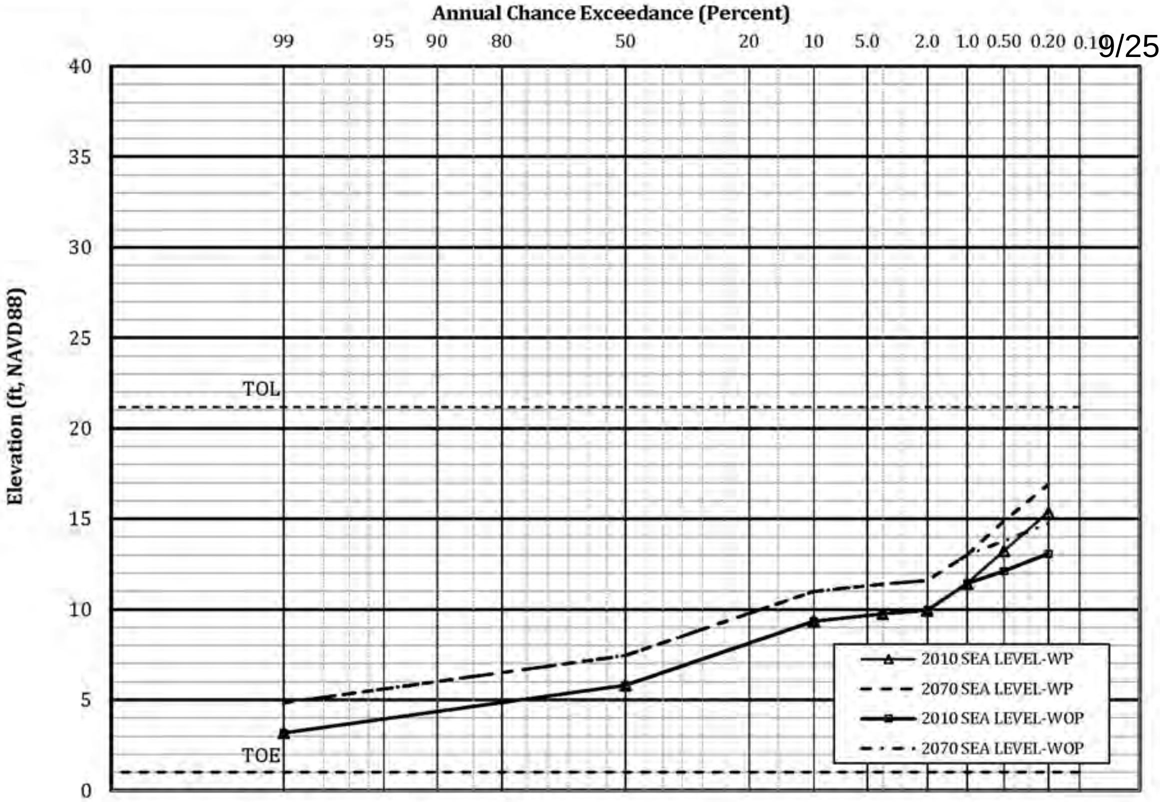
-TOE - Approx. elevation of natural floodplain adjacent to levee

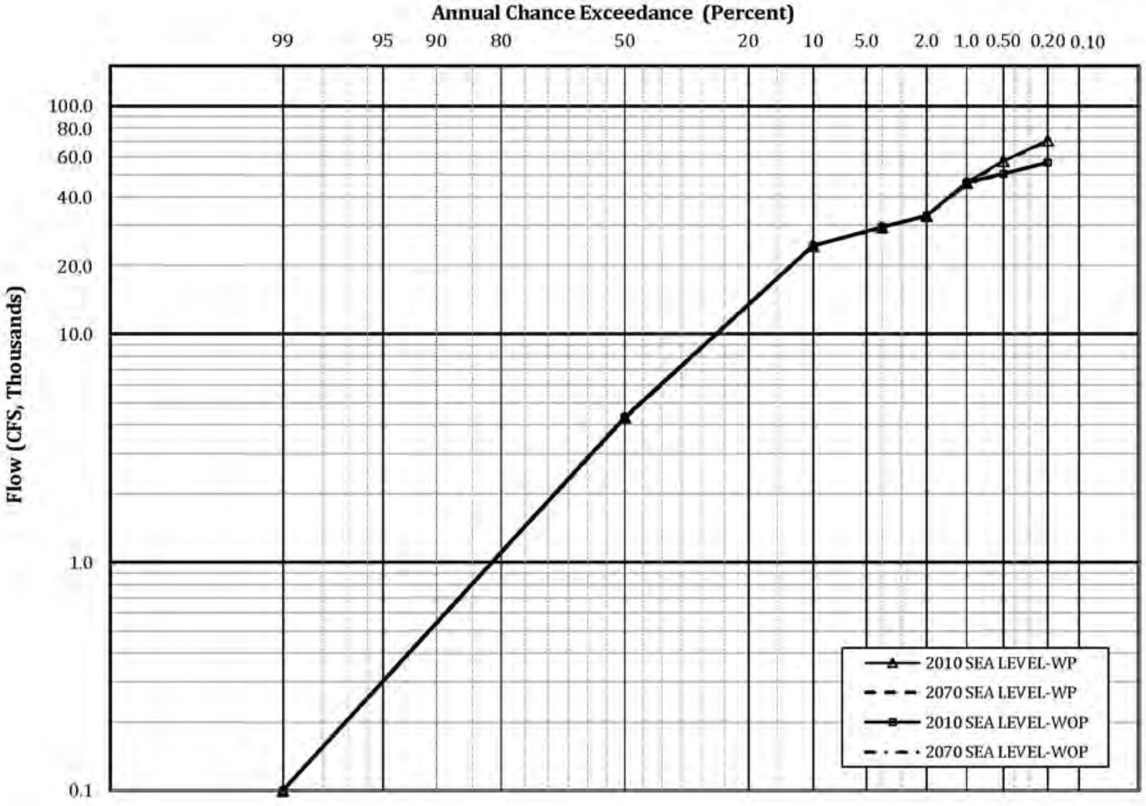
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE FREQUENCY CURVES AT INDEX POINT MIDDLE RIVER AT BORDEN HWY

United States Army Corps of Engineers
Sacramento District

JUN 2014





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

With-Project (WP) = RD17 levee heights adjusted, where necessary, to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Stages and Flows for Index Point Old River @ Clifton Court are from Old River at RS 20.092

-TOL - Top of levee elevation

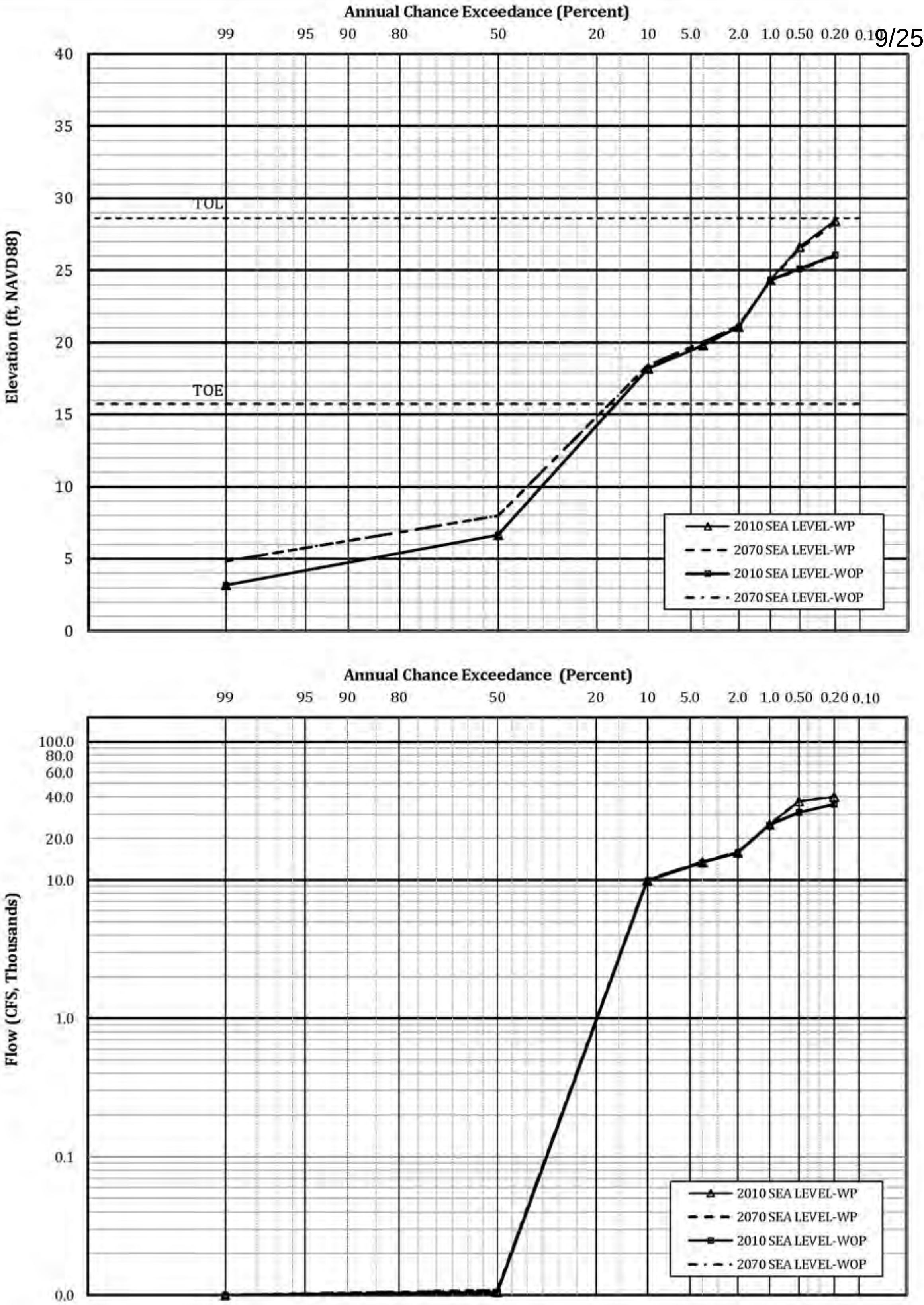
-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
OLD RIVER AT CLIFTON COURT

United States Army Corps of Engineers
Sacramento District

JUN 2014



NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:
Without-Project (WOP) = No Action Alternative
With-Project (WP) = RD17 levee heights adjusted, where necessary, to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Stages and Flows for Index Point Paradise Cut at I-5 are from Paradise Cut Reach 35 at RS 6.033

-TOL - Top of levee elevation

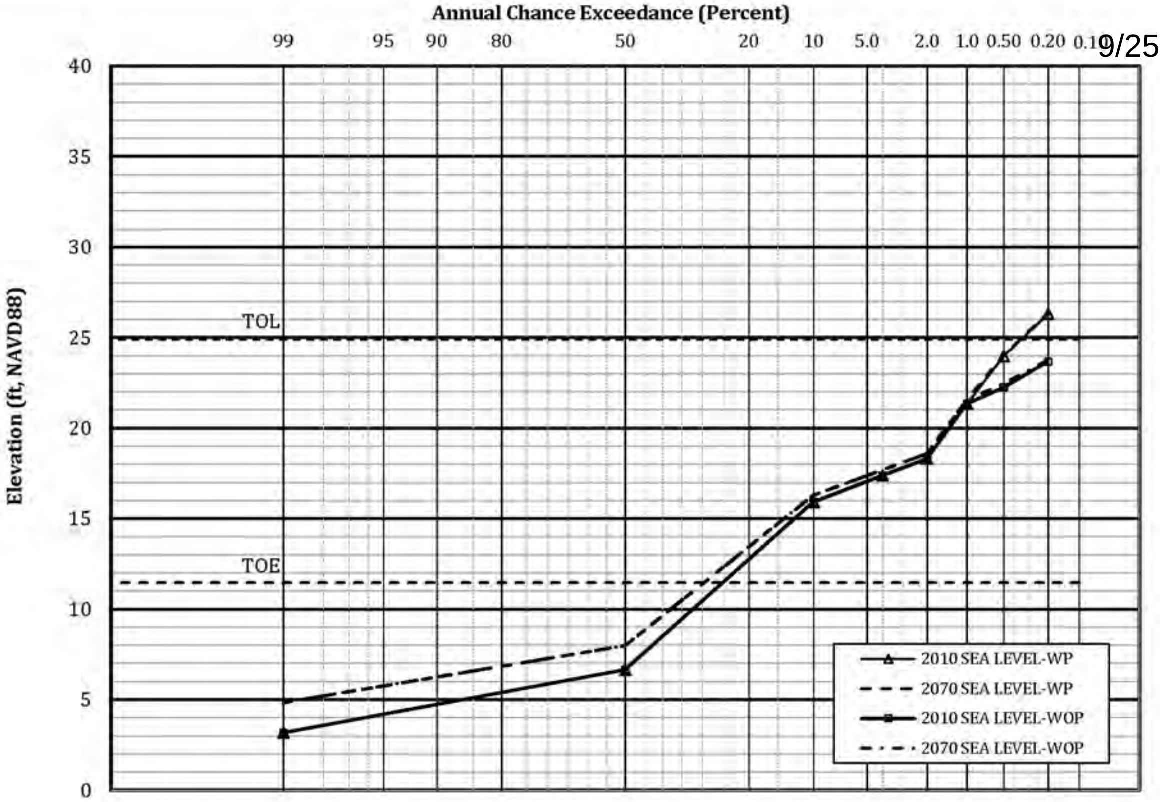
-TOE - Approx. elevation of natural floodplain adjacent to levee

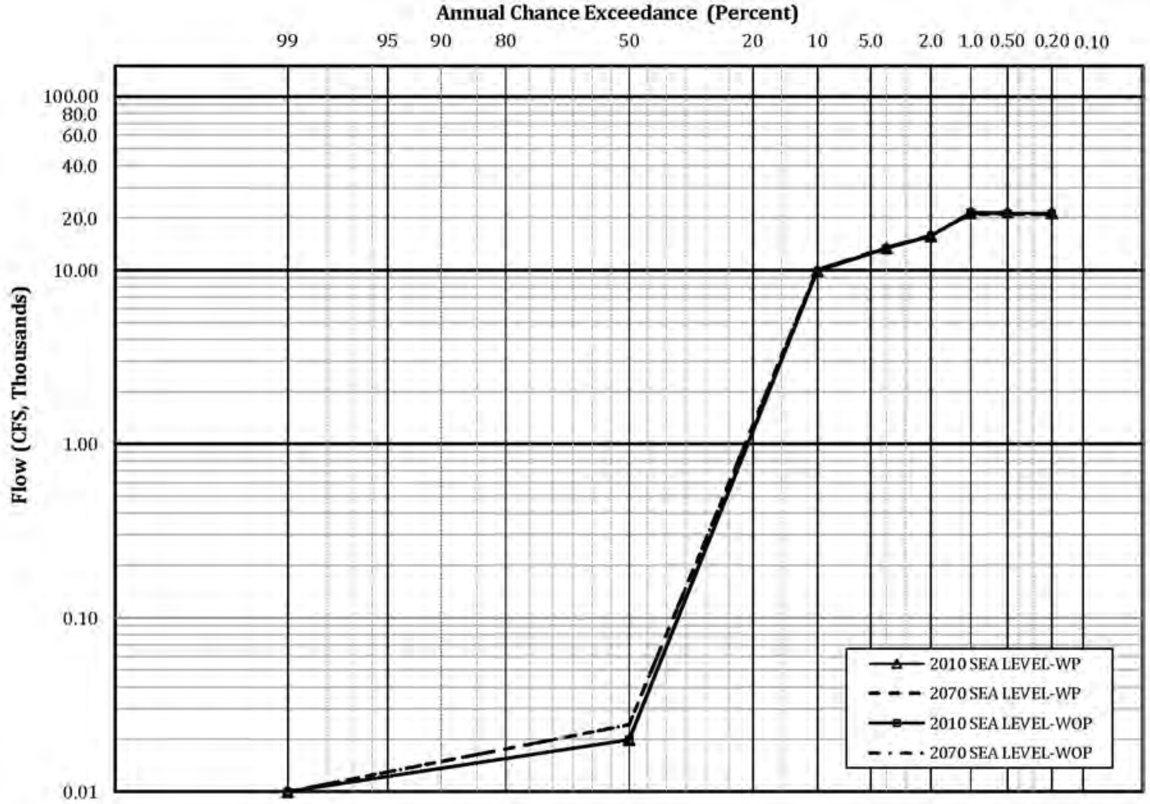
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
PARADISE CUT AT I-5

United States Army Corps of Engineers
Sacramento District

JUN 2014





NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:

Without-Project (WOP) = No Action Alternative

With-Project (WP) = RD17 levee heights adjusted, where necessary, to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)

-Stages and Flows for Index Point Paradise Cut at Paradise Rd are from Paradise Cut Reach 35 at RS 2.893

-TOL - Top of levee elevation

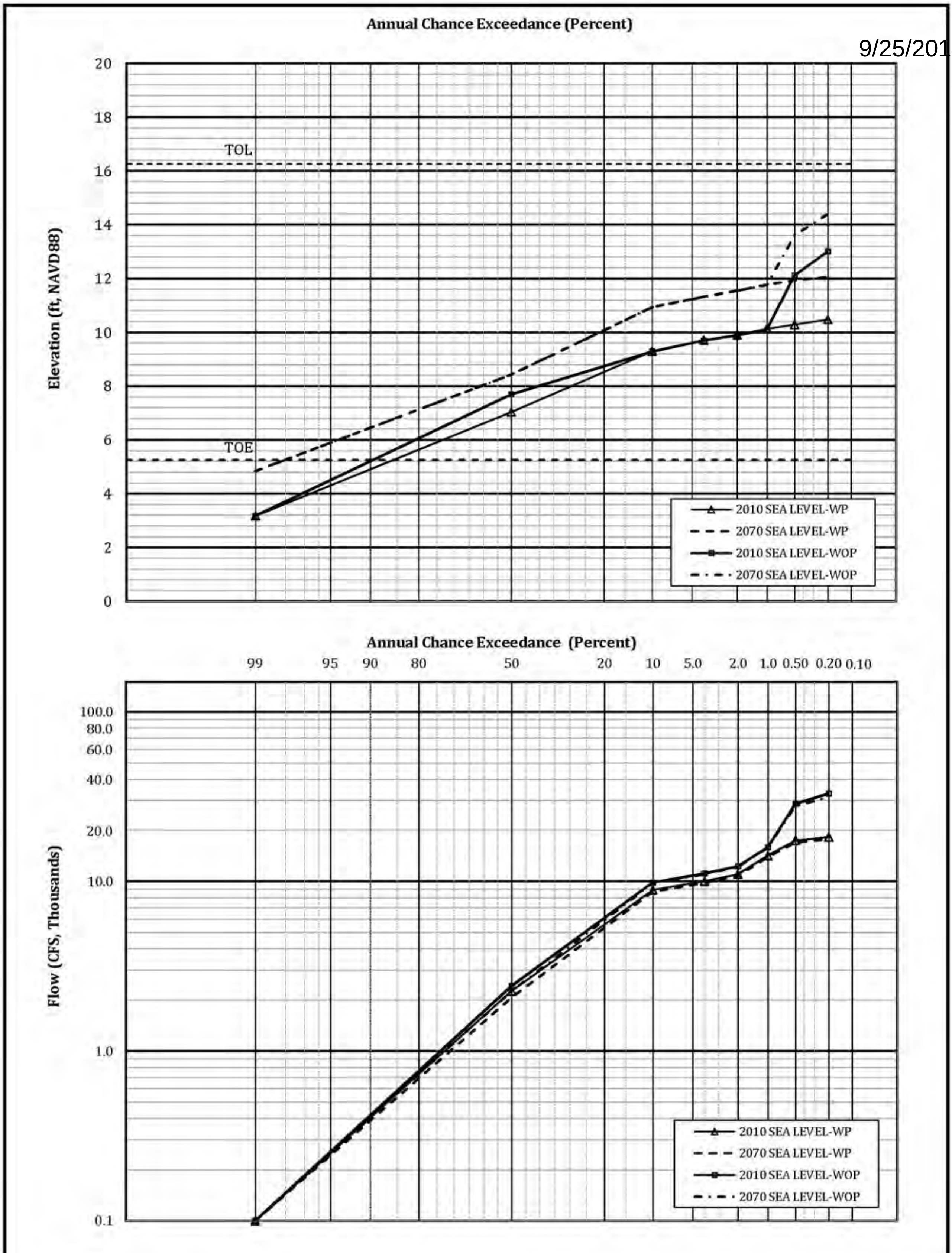
-TOE - Approx. elevation of natural floodplain adjacent to levee

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
PARADISE CUT AT PARADISE RD

United States Army Corps of Engineers
Sacramento District

JUN 2014



NOTES:

-Curves based on without- and with-project HEC-RAS simulations, where:
Without-Project (WOP) = No Action Alternative
With-Project (WP) = RD17 levee heights adjusted, where necessary,
to meet the minimum 200yr SB5 requirement (200yr mean WSE + 3-feet)
-Index Point San Joaquin River is located on the Ship Channel at RS 37.83
-TOE - Approx. elevation of natural floodplain adjacent to left bank levee
levee toe estimated where land flattens closest to levee within Rough
and Ready Island in line with HEC-RAS cross section

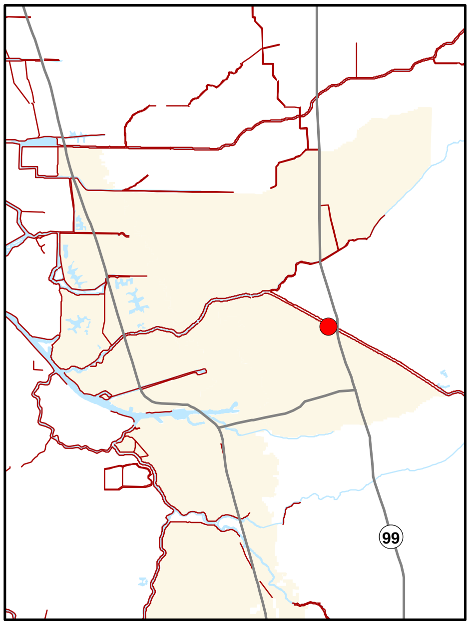
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIUM FEASIBILITY STUDY

STAGE AND DISCHARGE
FREQUENCY CURVES
AT INDEX POINT
SJR BELOW BURNS CUTOFF

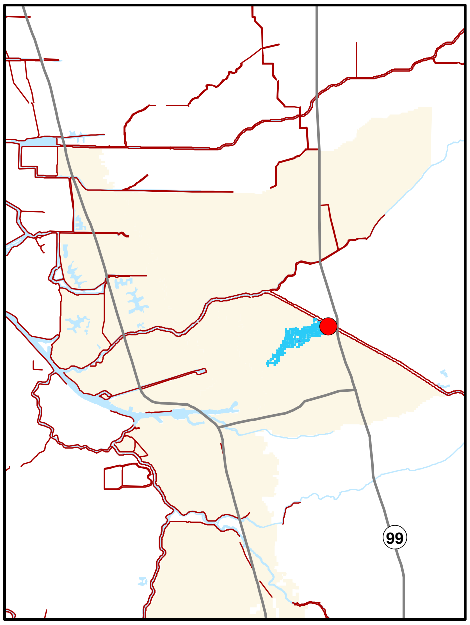
United States Army Corps of Engineers
Sacramento District

JUN 2014

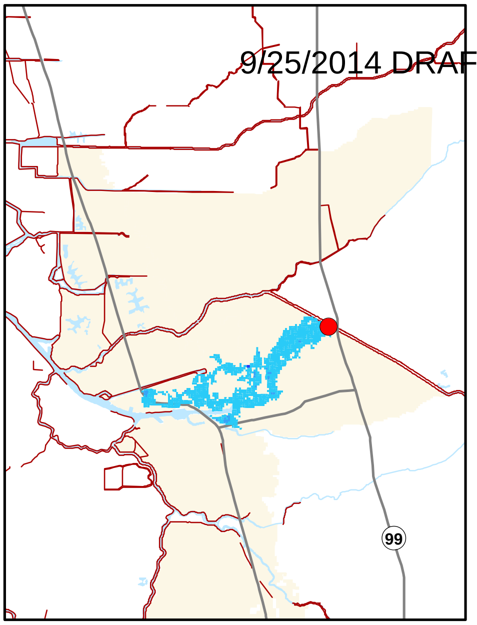
9/25/2014 DRAFT



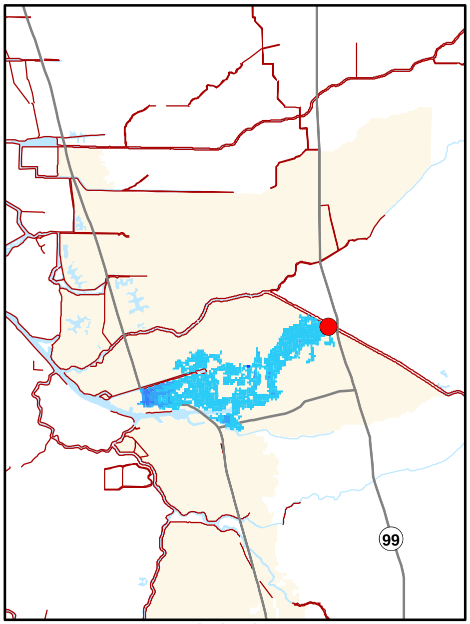
50% (1/2) ACE



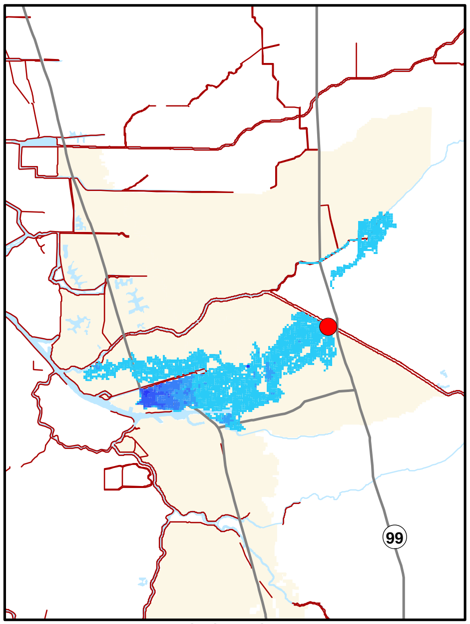
10% (1/10) ACE



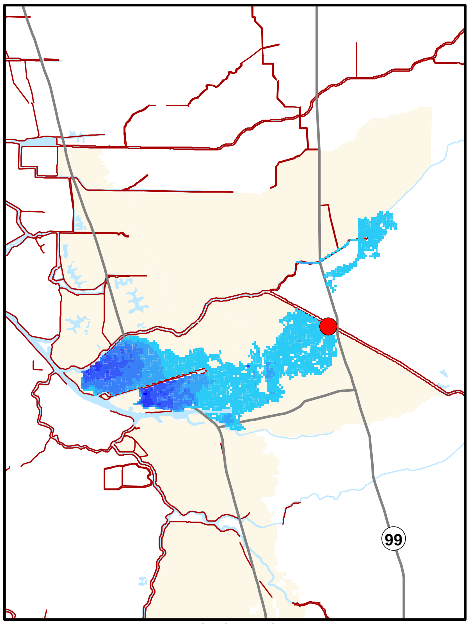
4% (1/25) ACE



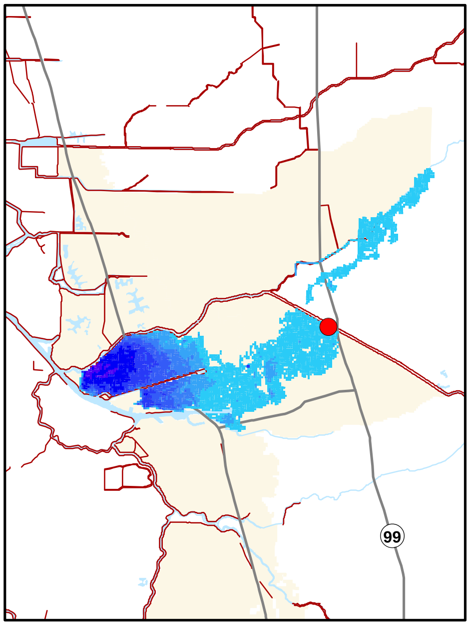
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-SL2

Depth (FT)

- 🟦 0.01 - 2.00
- 🟦 2.01 - 4.00
- 🟦 4.01 - 6.00
- 🟦 6.01 - 8.00
- 🟦 8.01 - 10.00
- 🟦 10.01 - 12.00
- 🟪 12.01 - 14.00
- 🟪 14.01 - 16.00
- 🟪 16.01 - 18.00
- 🟪 18.01 - 20.00
- 🟪 20.01 - 22.00
- 🟪 22.01 - 24.00
- ～ Levees (CLD, NLD)
- ■ LSJ Damage Areas

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

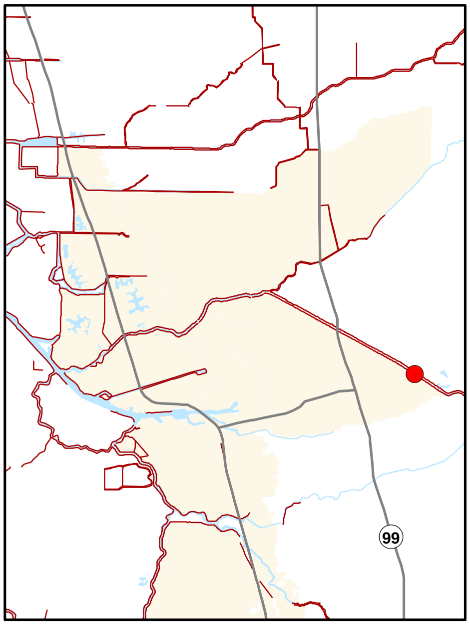
BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-SL1

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

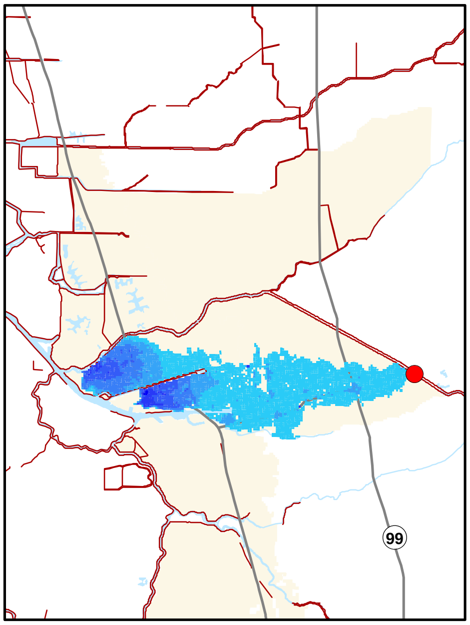
Prepared by C. Young

JUN 2014

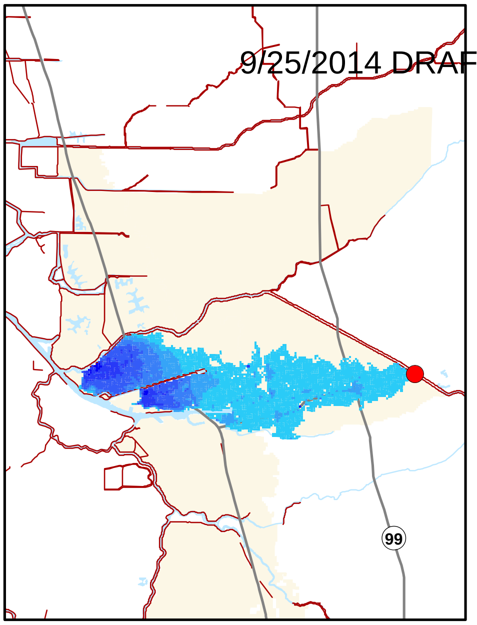
PLATE 33A



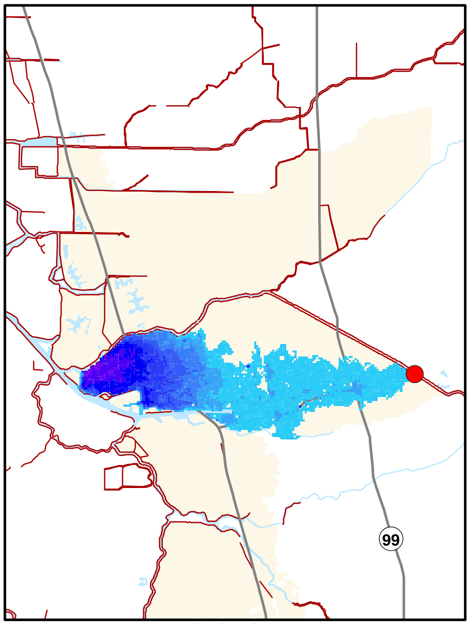
50% (1/2) ACE



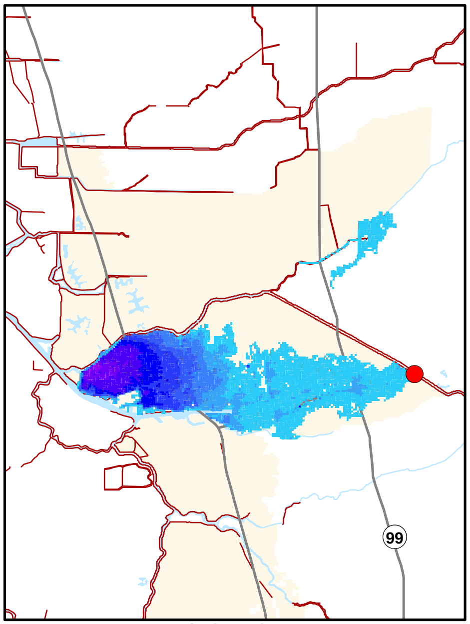
10% (1/10) ACE



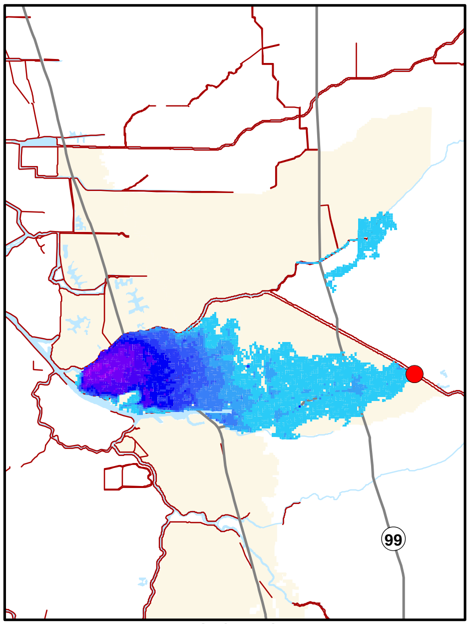
4% (1/25) ACE



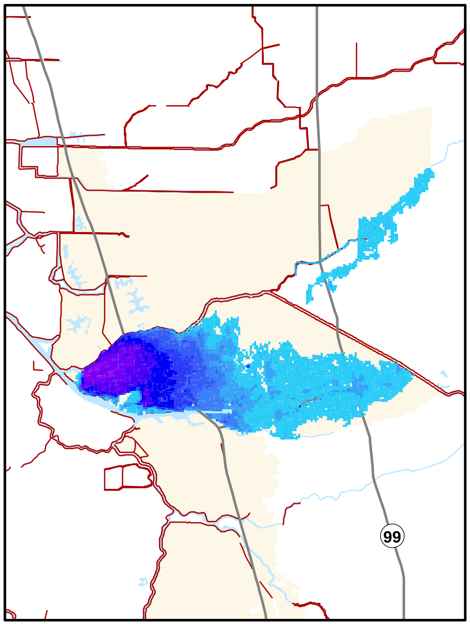
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-SL2

Depth (FT)

- ■ 0.01 - 2.00
- ■ 2.01 - 4.00
- ■ 4.01 - 6.00
- ■ 6.01 - 8.00
- ■ 8.01 - 10.00
- ■ 10.01 - 12.00
- ■ 12.01 - 14.00
- ■ 14.01 - 16.00
- ■ 16.01 - 18.00
- ■ 18.01 - 20.00
- ■ 20.01 - 22.00
- ■ 22.01 - 24.00
- ⁓ Levees (CLD, NLD)
- ■ LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

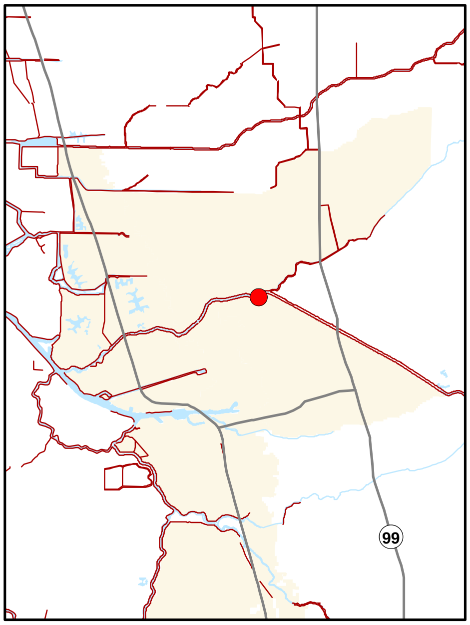
BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-SL2

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

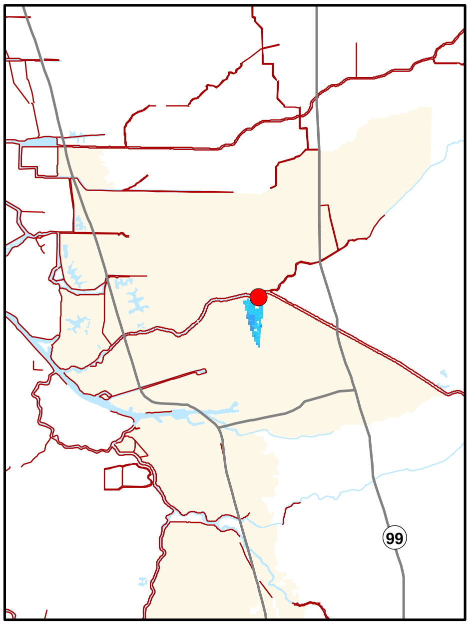
Prepared by C. Young

JUN 2014

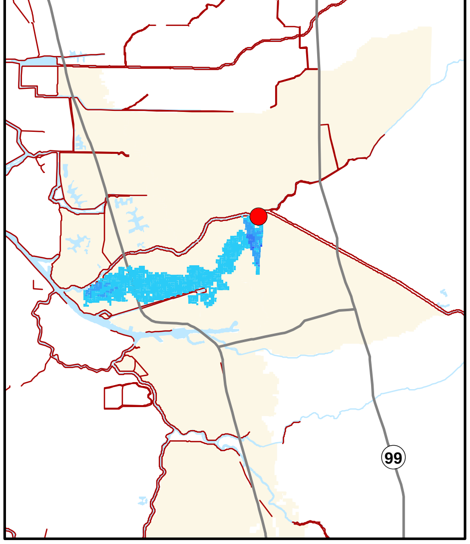
PLATE 33B



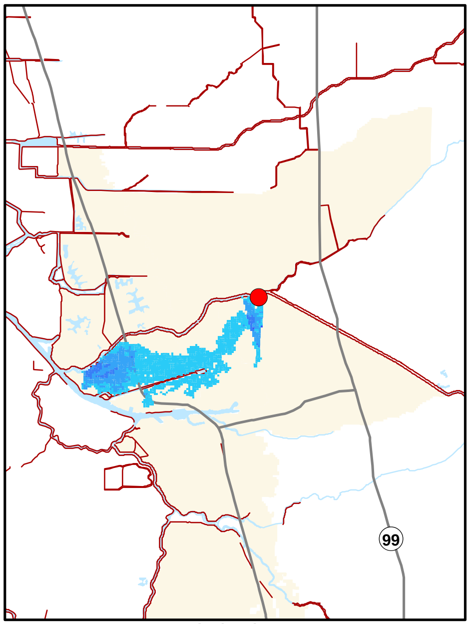
50% (1/2) ACE



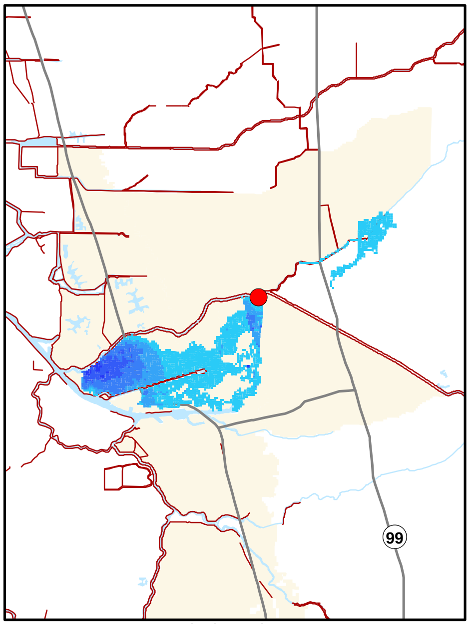
10% (1/10) ACE



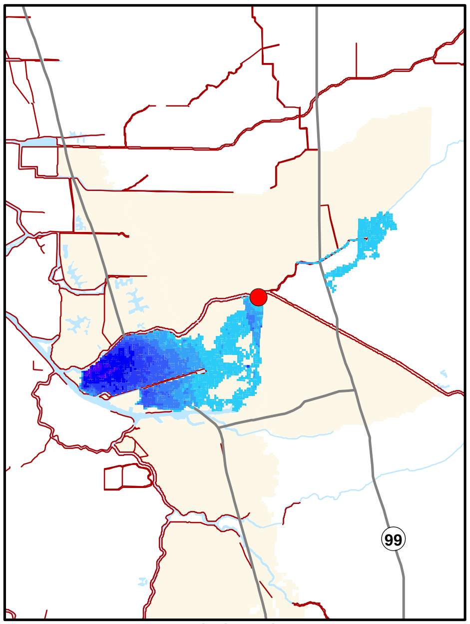
4% (1/25) ACE



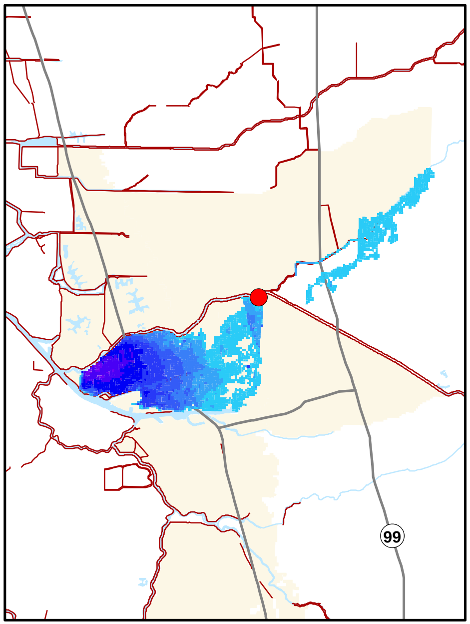
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-CL2

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00
- Levees (CLD, NLD)
- LSJ Damage Areas

0 5 Miles

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

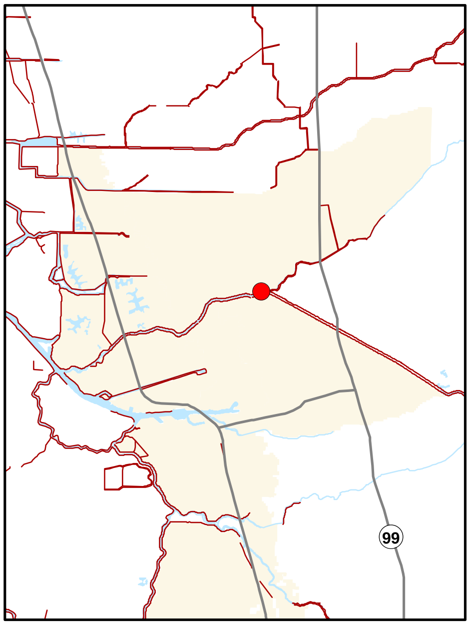
BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-CL2

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

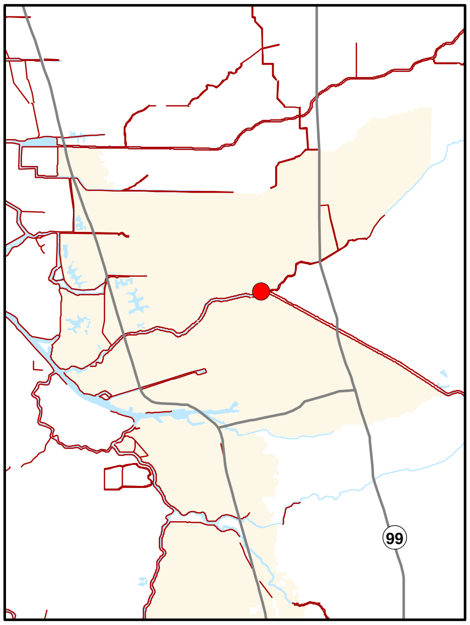
Prepared by C. Young

JUN 2014

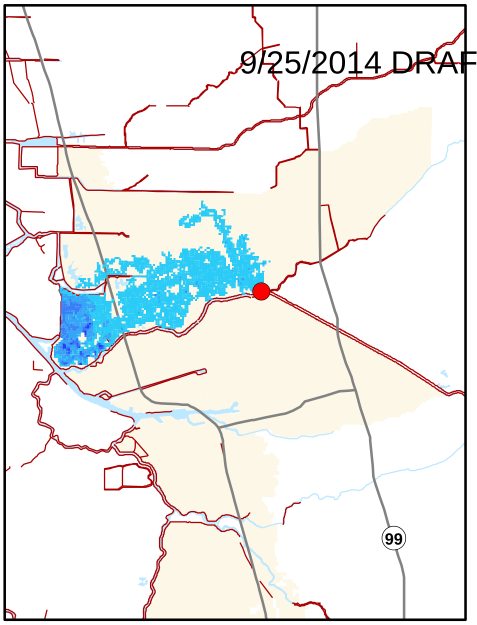
PLATE 33C



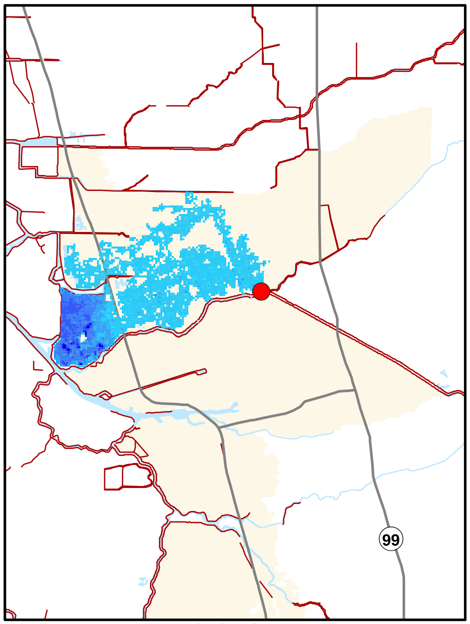
50% (1/2) ACE



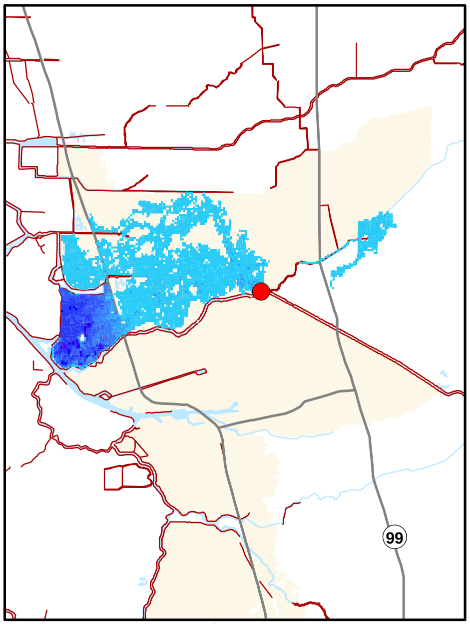
10% (1/10) ACE



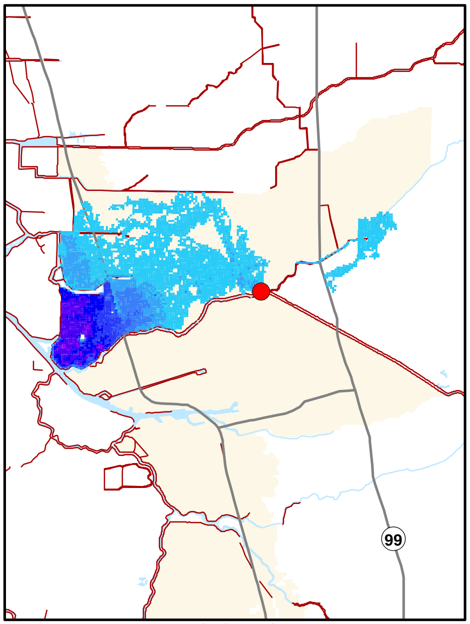
4% (1/25) ACE



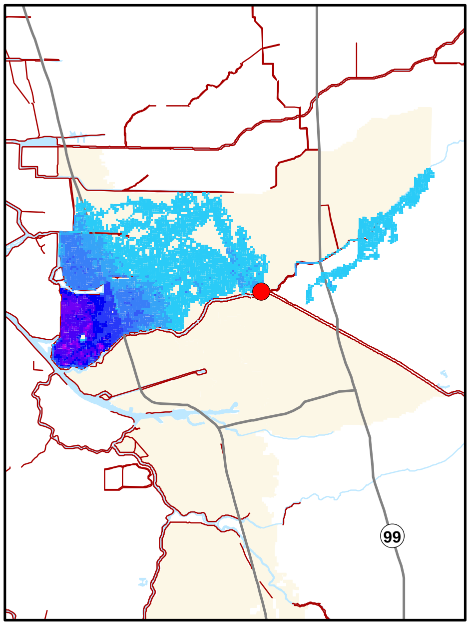
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-CR2

Depth (FT)

- ■ 0.01 - 2.00
- ■ 2.01 - 4.00
- ■ 4.01 - 6.00
- ■ 6.01 - 8.00
- ■ 8.01 - 10.00
- ■ 10.01 - 12.00
- ■ 12.01 - 14.00
- ■ 14.01 - 16.00
- ■ 16.01 - 18.00
- ■ 18.01 - 20.00
- ■ 20.01 - 22.00
- ■ 22.01 - 24.00
- — Levees (CLD, NLD)
- ■ LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

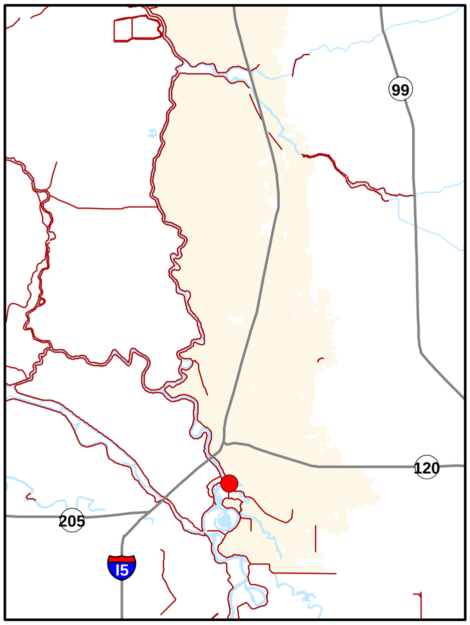
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-CR2

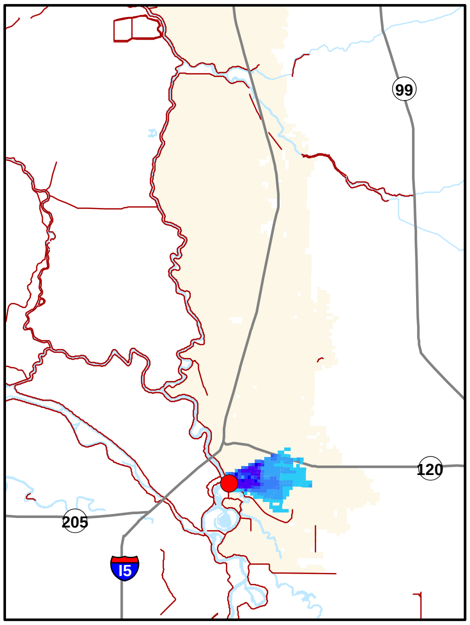
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

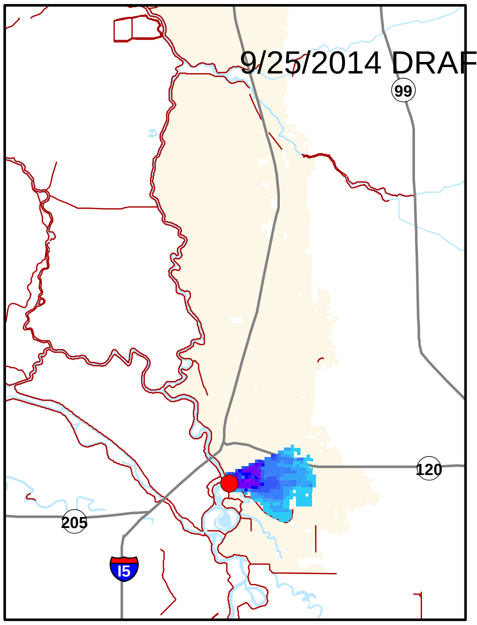
JUN 2014



50% (1/2) ACE

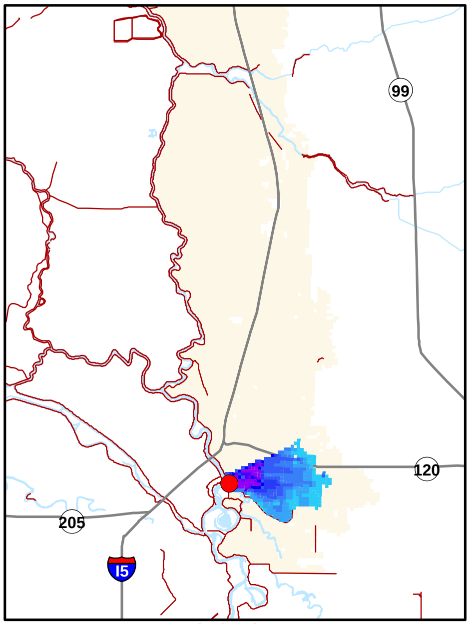


10% (1/10) ACE

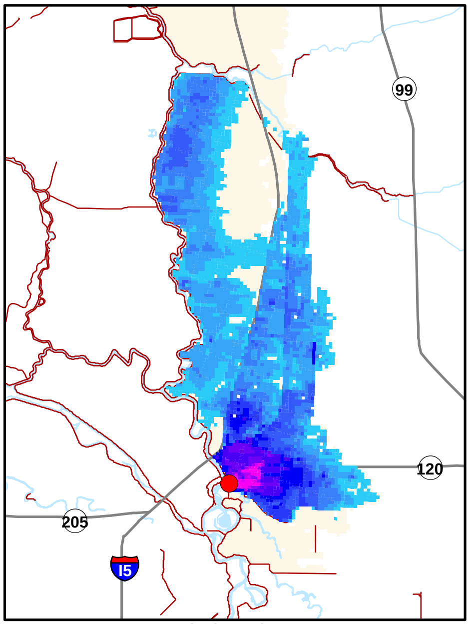


9/25/2014 DRAFT

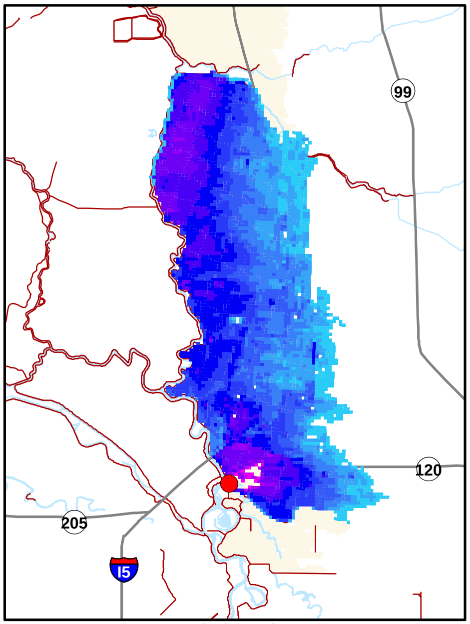
4% (1/25) ACE



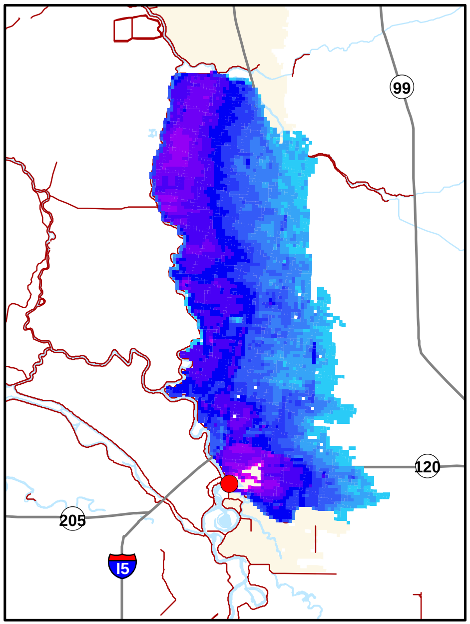
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-LR4

Depth (FT)

🟦 0.01 - 2.00
🟦 2.01 - 4.00
🟦 4.01 - 6.00
🟦 6.01 - 8.00
🟦 8.01 - 10.00
🟦 10.01 - 12.00
🟪 12.01 - 14.00
🟪 14.01 - 16.00
🟪 16.01 - 18.00
🟪 18.01 - 20.00
🟪 20.01 - 22.00
🟪 22.01 - 24.00
〜 Levees (CLD, NLD)
▭ LSJ Damage Areas

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-LR4

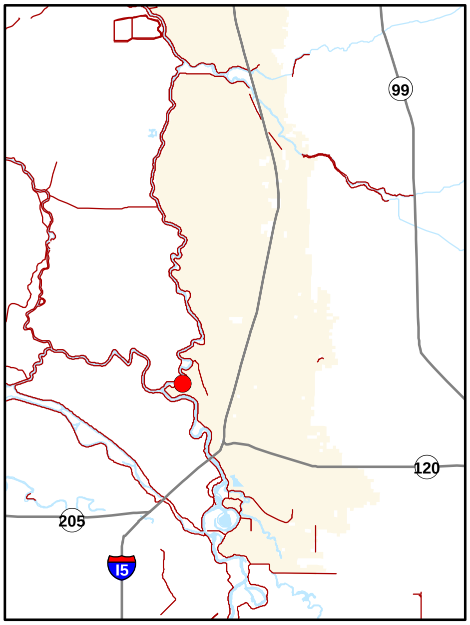
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

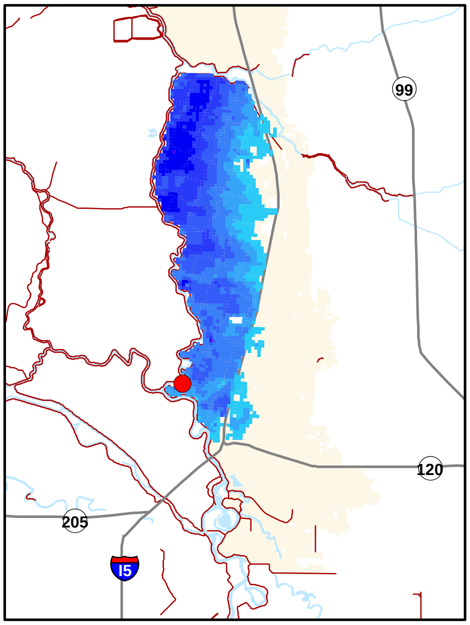
APR 2014

PLATE 33E

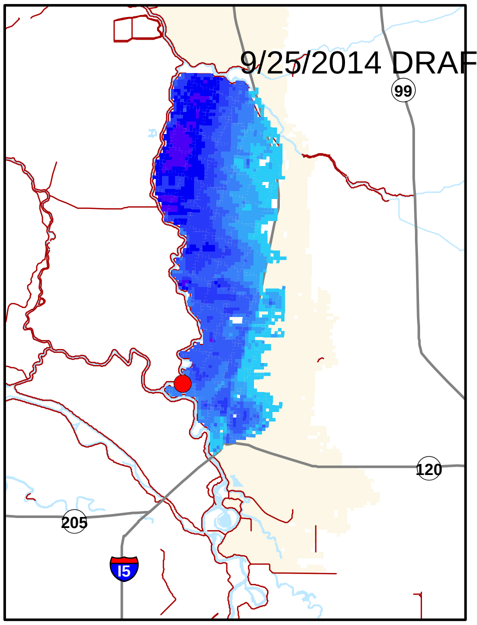
9/25/2014 DRAFT



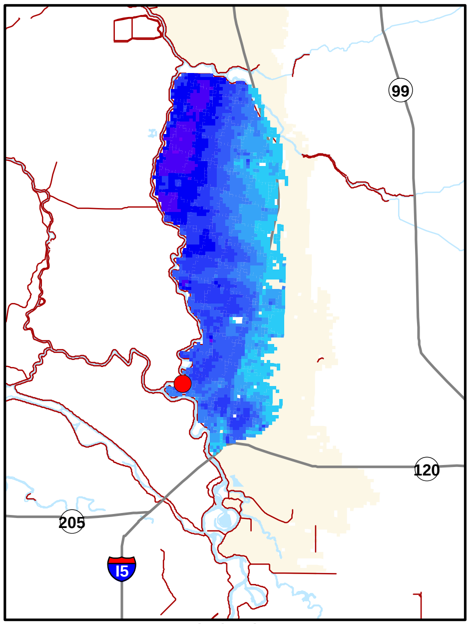
50% (1/2) ACE



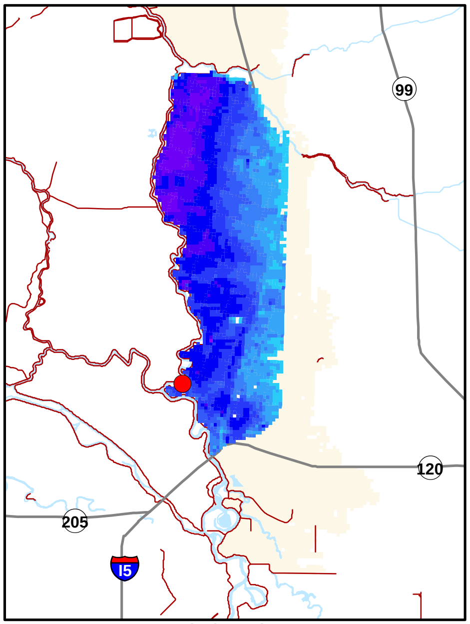
10% (1/10) ACE



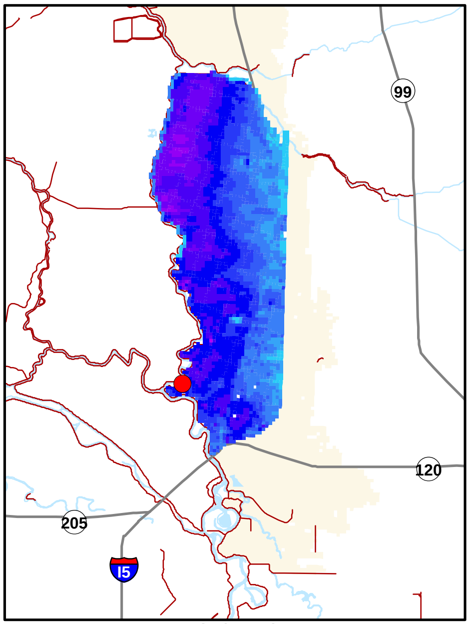
4% (1/25) ACE



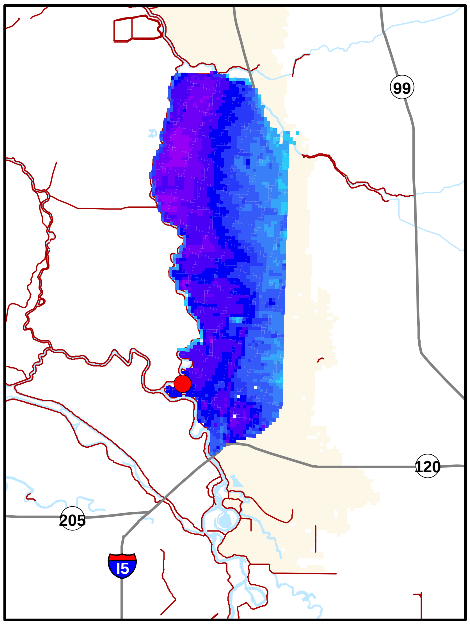
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-LR3

Depth (FT)

- ■ 0.01 - 2.00
- ■ 2.01 - 4.00
- ■ 4.01 - 6.00
- ■ 6.01 - 8.00
- ■ 8.01 - 10.00
- ■ 10.01 - 12.00
- ■ 12.01 - 14.00
- ■ 14.01 - 16.00
- ■ 16.01 - 18.00
- ■ 18.01 - 20.00
- ■ 20.01 - 22.00
- ■ 22.01 - 24.00
- ⌒ Levees (CLD, NLD)
- ■ LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-LR3

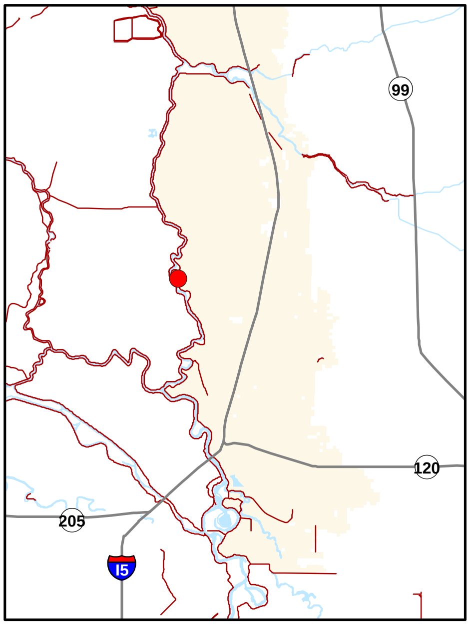
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

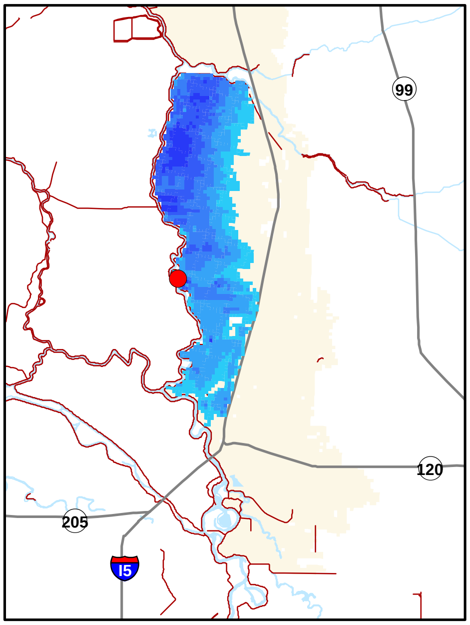
JUN 2014

PLATE 33F

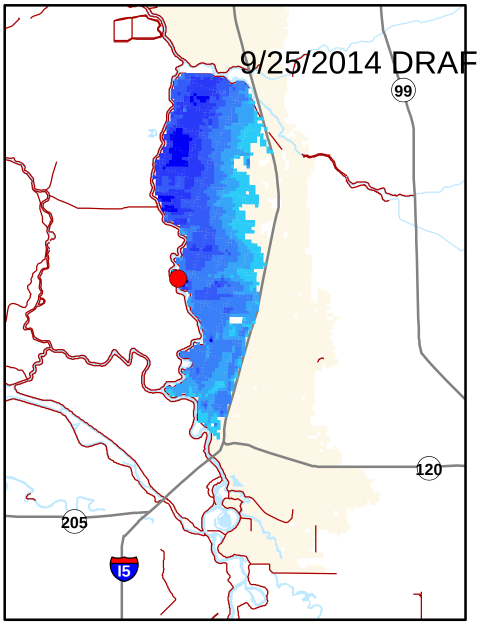
9/25/2014 DRAFT



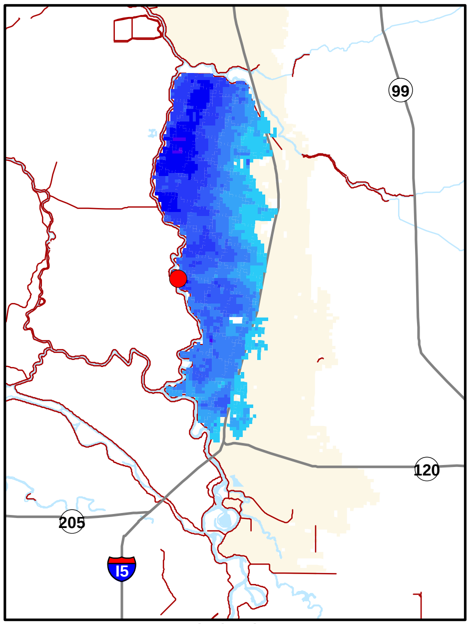
50% (1/2) ACE



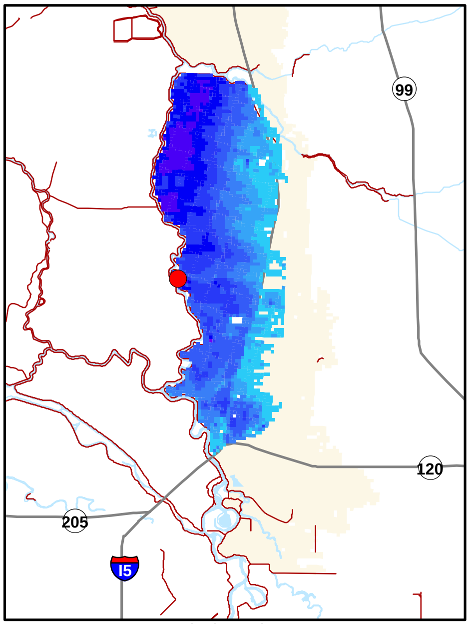
10% (1/10) ACE



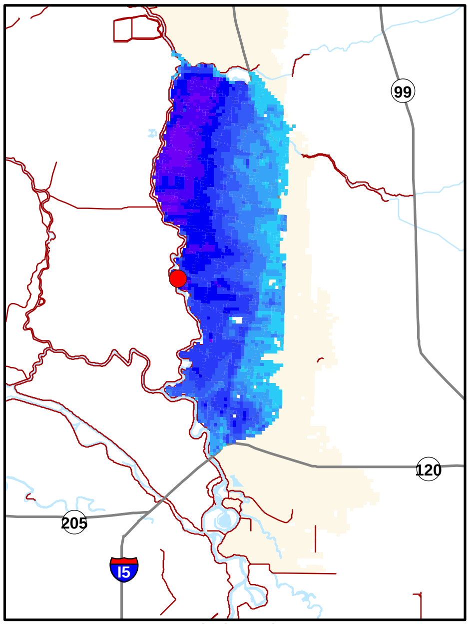
4% (1/25) ACE



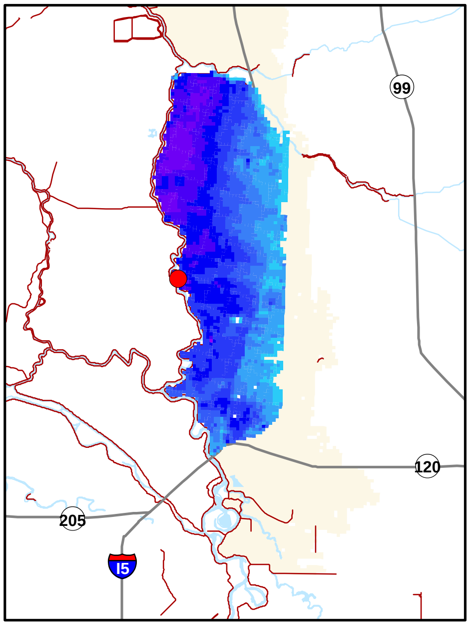
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-LR2

Depth (FT)

🟦	0.01 - 2.00
🟦	2.01 - 4.00
🟦	4.01 - 6.00
🟦	6.01 - 8.00
🟦	8.01 - 10.00
🟦	10.01 - 12.00
🟪	12.01 - 14.00
🟪	14.01 - 16.00
🟪	16.01 - 18.00
🟪	18.01 - 20.00
🟪	20.01 - 22.00
🟪	22.01 - 24.00
⌒	Levees (CLD, NLD)
▨	LSJ Damage Areas

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

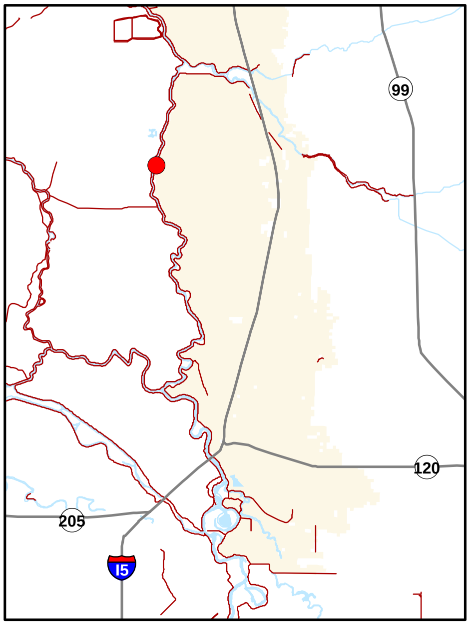
BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-LR2

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

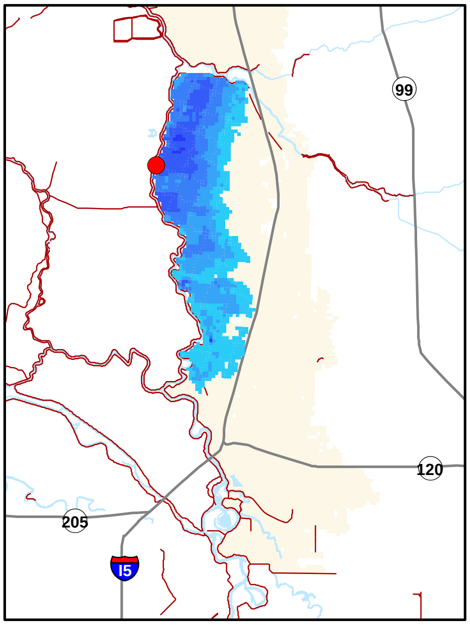
Prepared by C. Young

JUN 2014

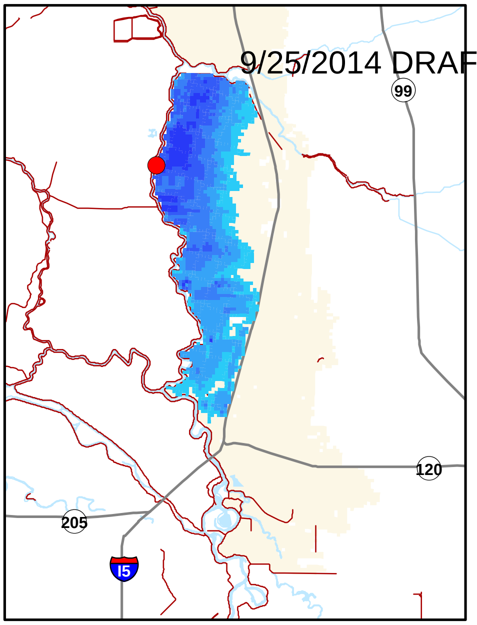
9/25/2014 DRAFT



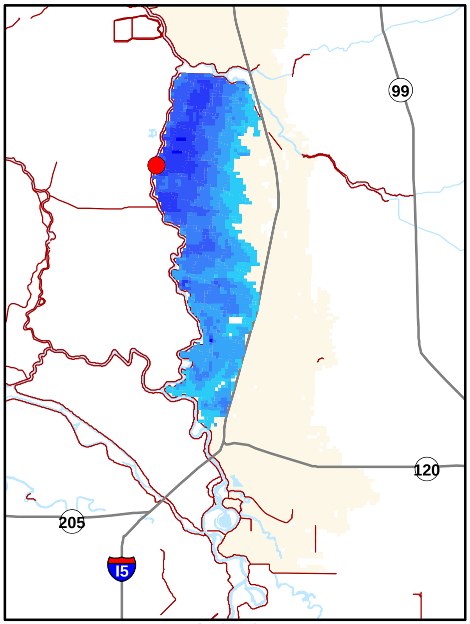
50% (1/2) ACE



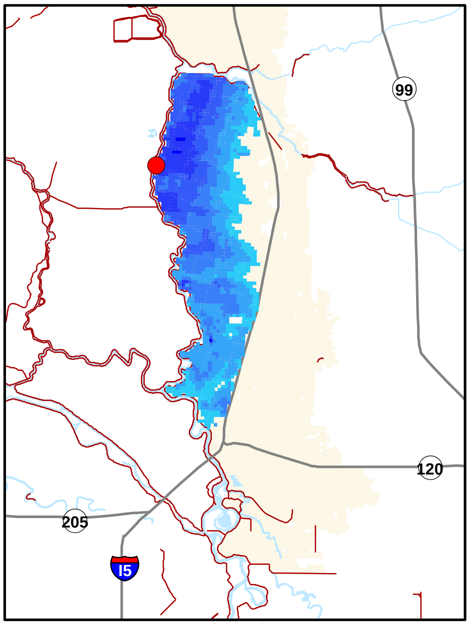
10% (1/10) ACE



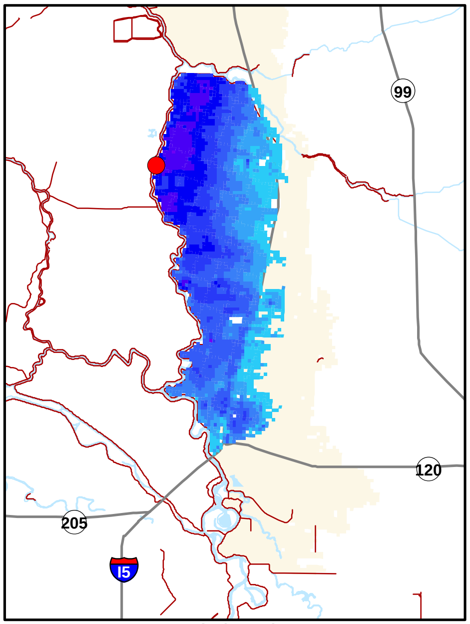
4% (1/25) ACE



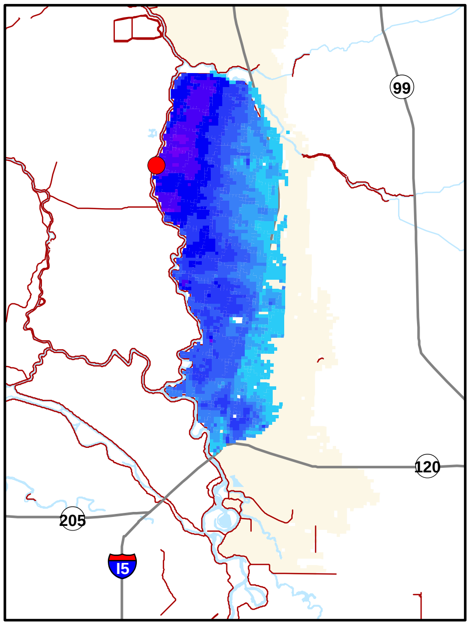
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-LR1

Depth (FT)

🟦 0.01 - 2.00
🟦 2.01 - 4.00
🟦 4.01 - 6.00
🟦 6.01 - 8.00
🟦 8.01 - 10.00
🟦 10.01 - 12.00
🟦 12.01 - 14.00
🟪 14.01 - 16.00
🟪 16.01 - 18.00
🟪 18.01 - 20.00
🟪 20.01 - 22.00
🟪 22.01 - 24.00
〰 Levees (CLD, NLD)
▨ LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-LR1

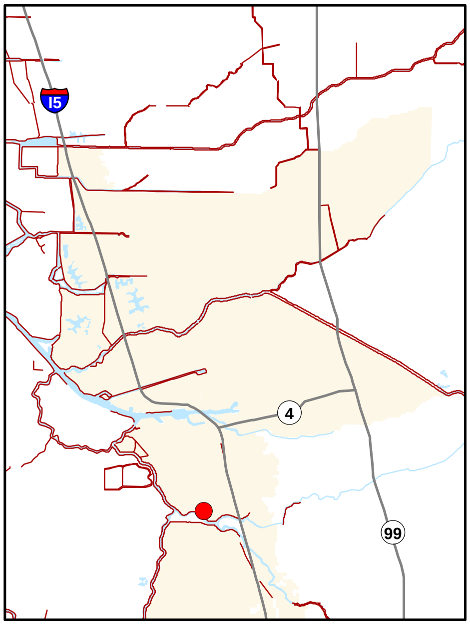
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

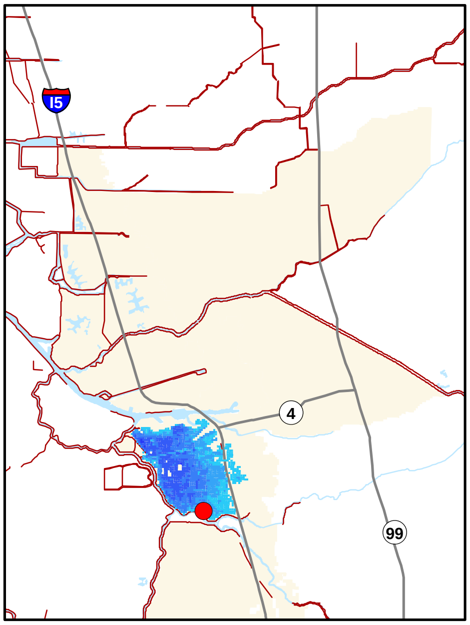
JUN 2014

PLATE 33H

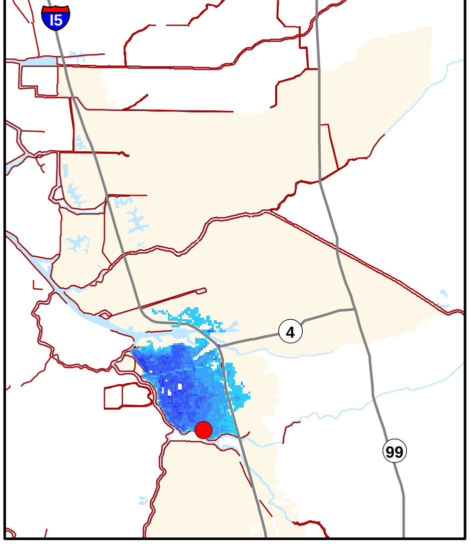
9/25/2014 DRAFT



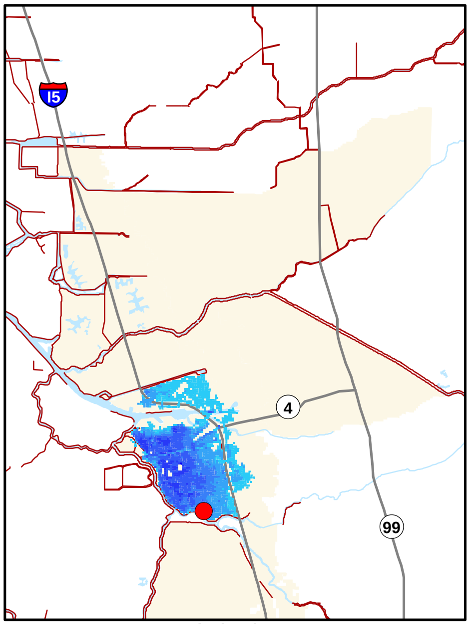
50% (1/2) ACE



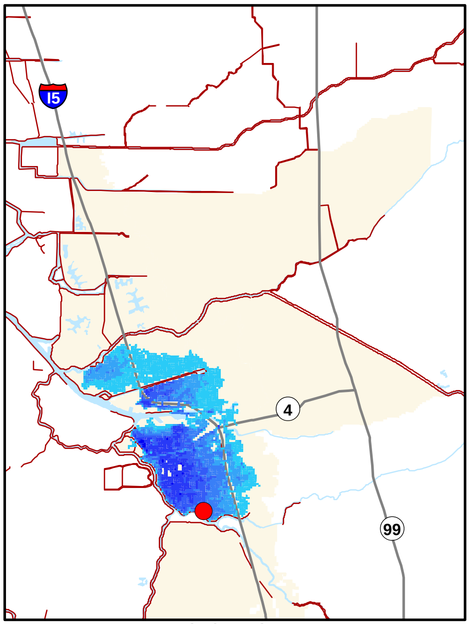
10% (1/10) ACE



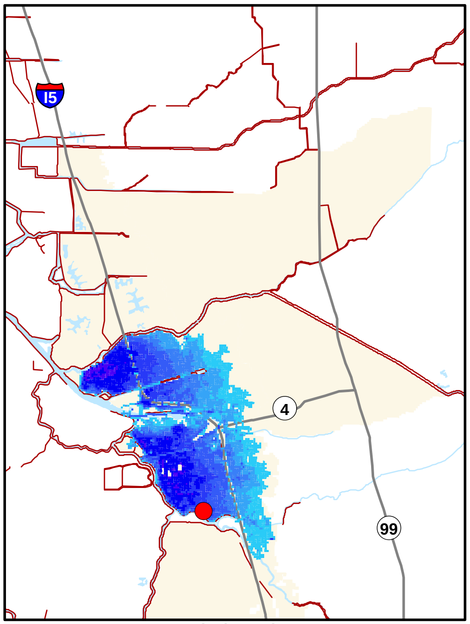
4% (1/25) ACE



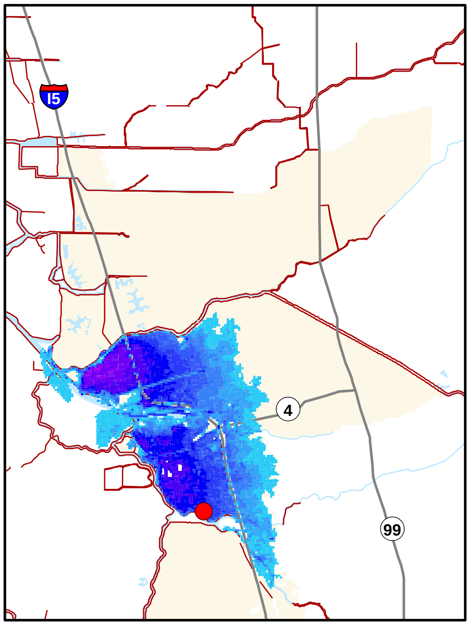
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-FR1

Depth (FT)

🟦 0.01 - 2.00
🟦 2.01 - 4.00
🟦 4.01 - 6.00
🟦 6.01 - 8.00
🟦 8.01 - 10.00
🟦 10.01 - 12.00
🟦 12.01 - 14.00
🟪 14.01 - 16.00
🟪 16.01 - 18.00
🟪 18.01 - 20.00
🟪 20.01 - 22.00
🟪 22.01 - 24.00
〰 Levees (CLD, NLD)
🟨 LSJ Damage Areas

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-FR1

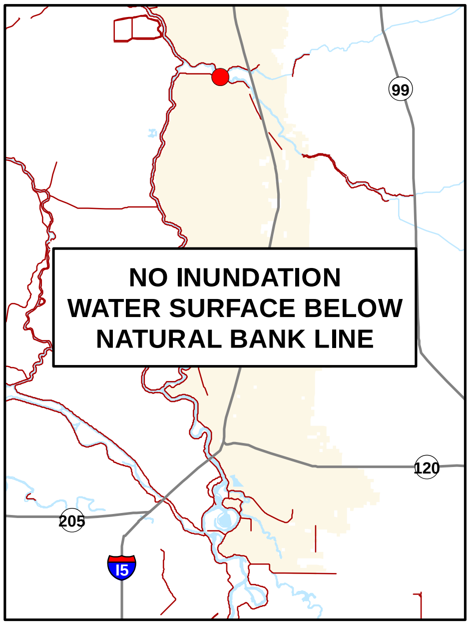
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

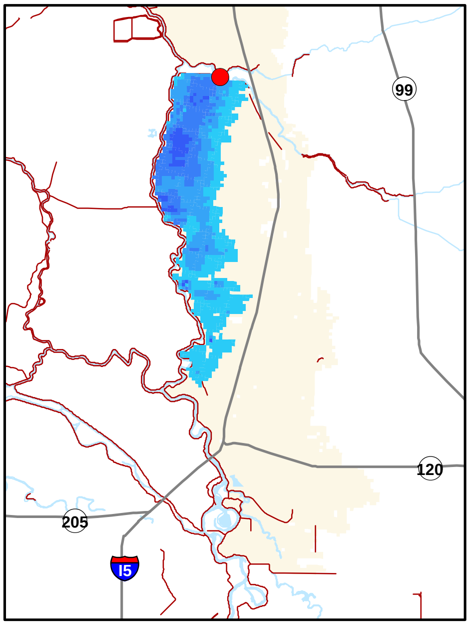
PLATE 33I

9/25/2014 DRAFT

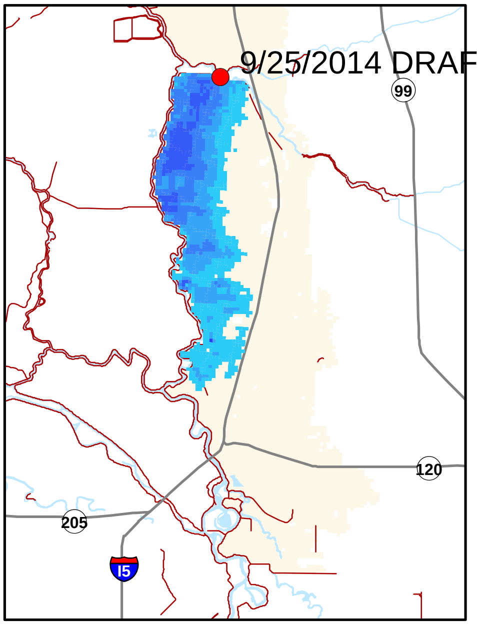


**NO INUNDATION
WATER SURFACE BELOW
NATURAL BANK LINE**

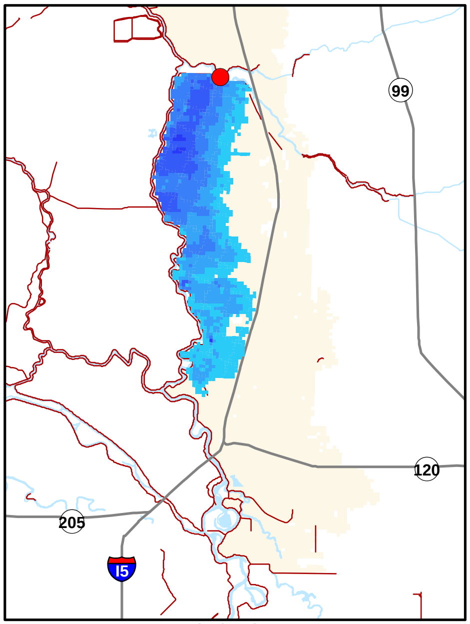
50% (1/2) ACE



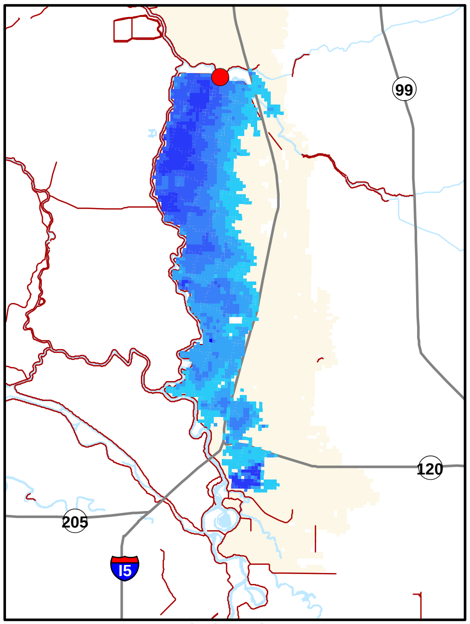
10% (1/10) ACE



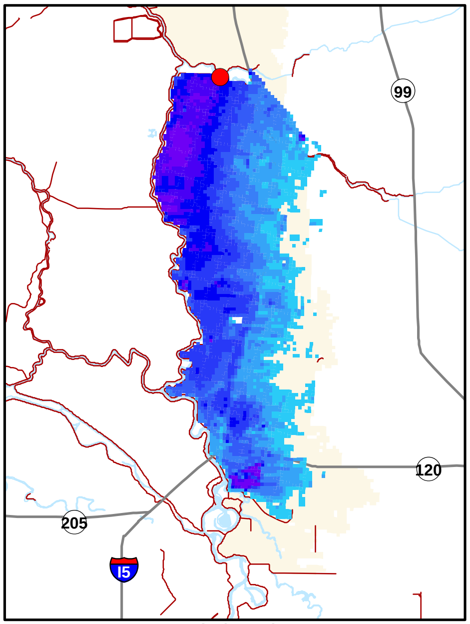
4% (1/25) ACE



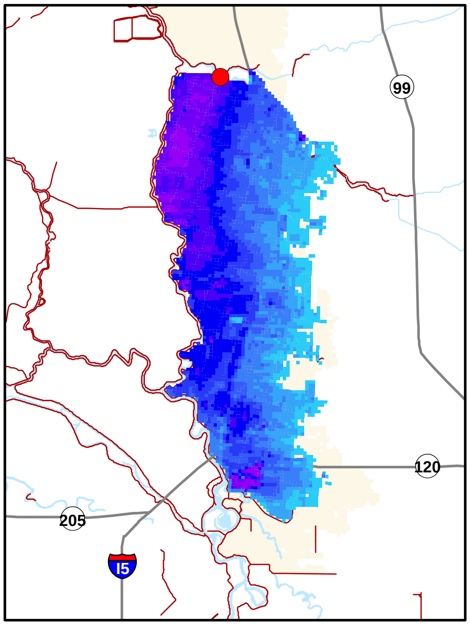
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-FL1

Depth (FT)

■ 0.01 - 2.00
■ 2.01 - 4.00
■ 4.01 - 6.00
■ 6.01 - 8.00
■ 8.01 - 10.00
■ 10.01 - 12.00
■ 12.01 - 14.00
■ 14.01 - 16.00
■ 16.01 - 18.00
■ 18.01 - 20.00
■ 20.01 - 22.00
■ 22.01 - 24.00
— Levees (CLD, NLD)
■ LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

**SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY**

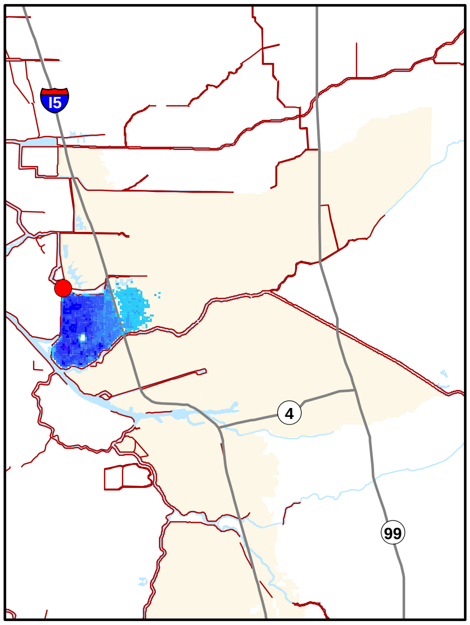
BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-FL1

**U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT**

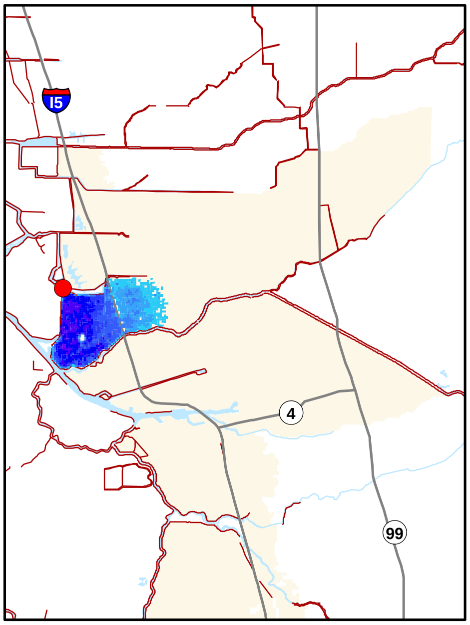
Prepared by C. Young

JUN 2014

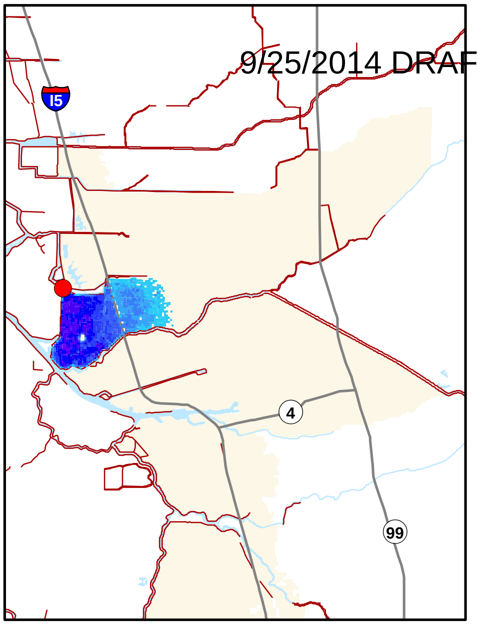
9/25/2014 DRAFT



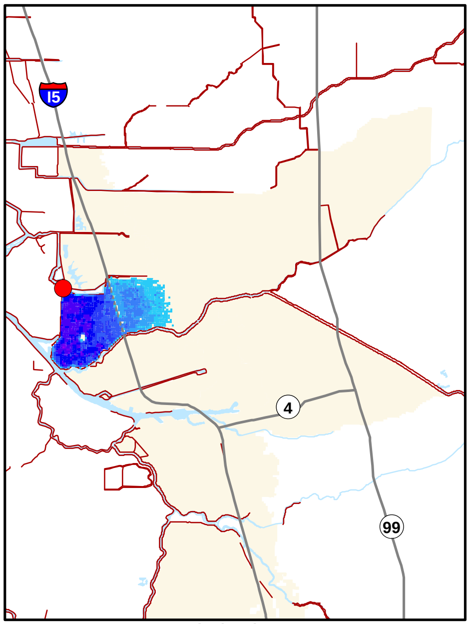
50% (1/2) ACE



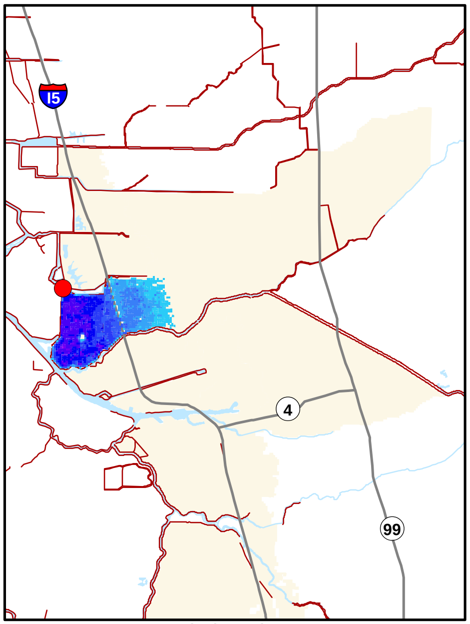
10% (1/10) ACE



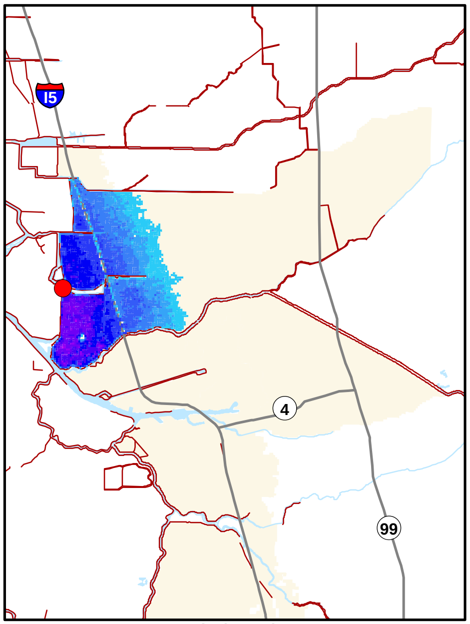
4% (1/25) ACE



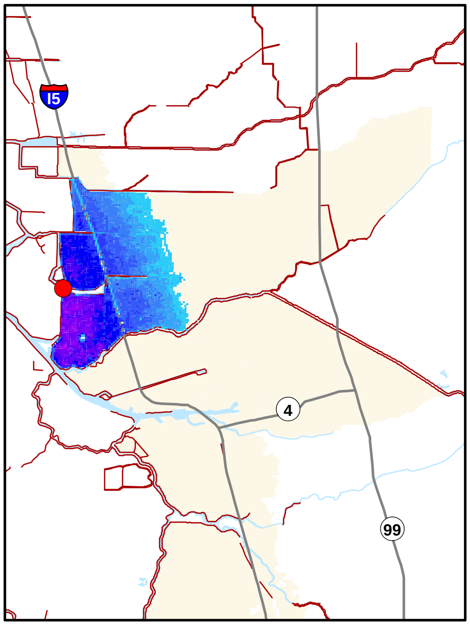
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-D-BS

Depth (FT)

■ 0.01 - 2.00
■ 2.01 - 4.00
■ 4.01 - 6.00
■ 6.01 - 8.00
■ 8.01 - 10.00
■ 10.01 - 12.00
■ 12.01 - 14.00
■ 14.01 - 16.00
■ 16.01 - 18.00
■ 18.01 - 20.00
■ 20.01 - 22.00
■ 22.01 - 24.00
～ Levees (CLD, NLD)
■ LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

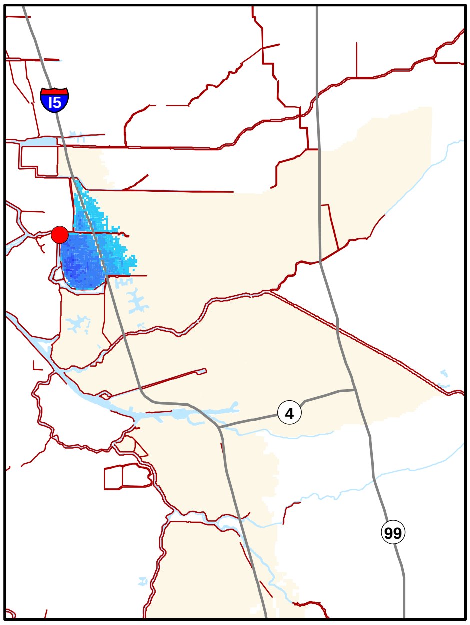
BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-D-BS

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

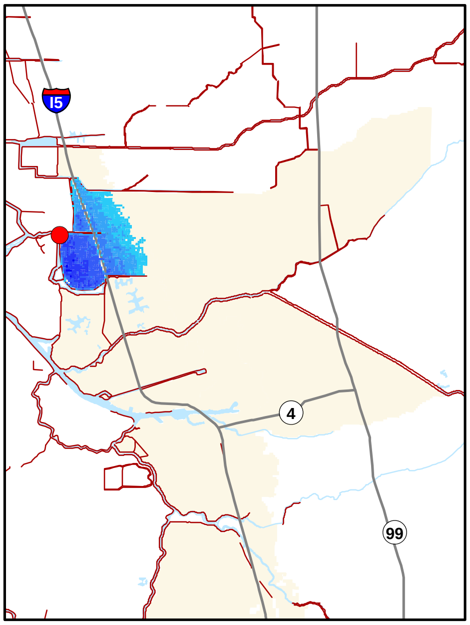
Prepared by C. Young

JUN 2014

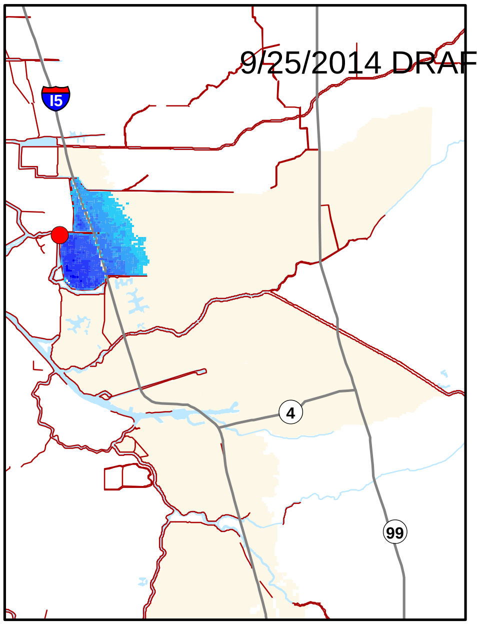
PLATE 34A



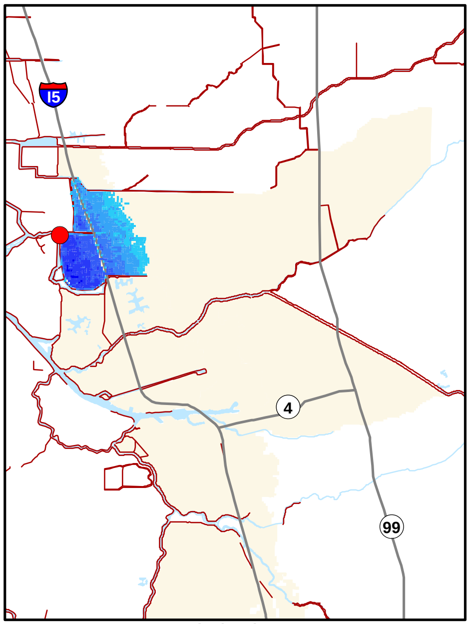
50% (1/2) ACE



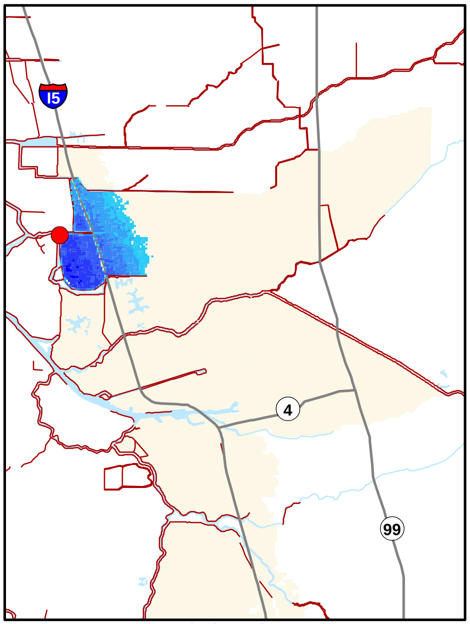
10% (1/10) ACE



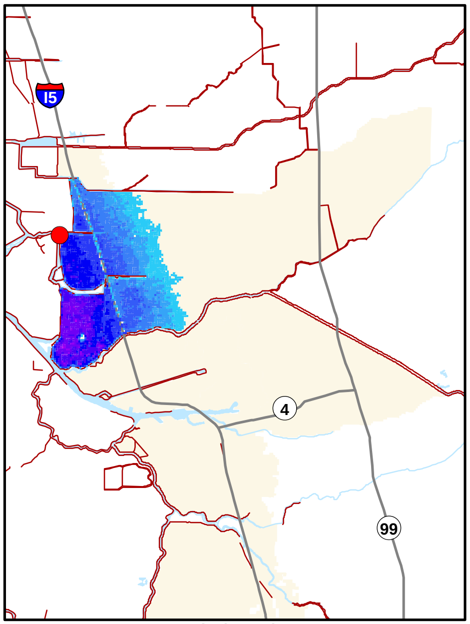
4% (1/25) ACE



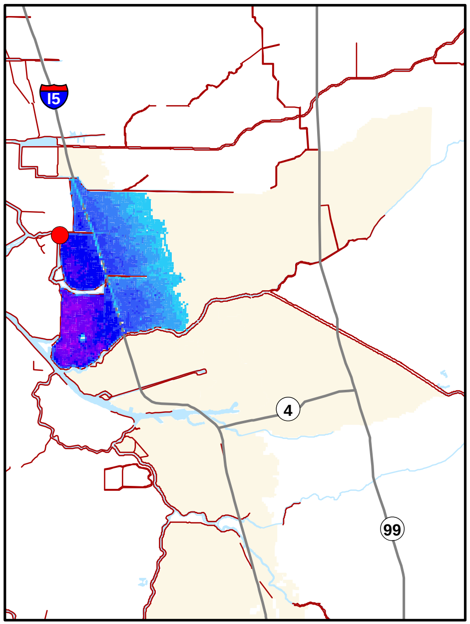
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-D3

Depth (FT)

🟦 0.01 - 2.00
🟦 2.01 - 4.00
🟦 4.01 - 6.00
🟦 6.01 - 8.00
🟦 8.01 - 10.00
🟦 10.01 - 12.00
🟪 12.01 - 14.00
🟪 14.01 - 16.00
🟪 16.01 - 18.00
🟪 18.01 - 20.00
🟪 20.01 - 22.00
🟪 22.01 - 24.00
Levees (CLD, NLD)
▨ LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

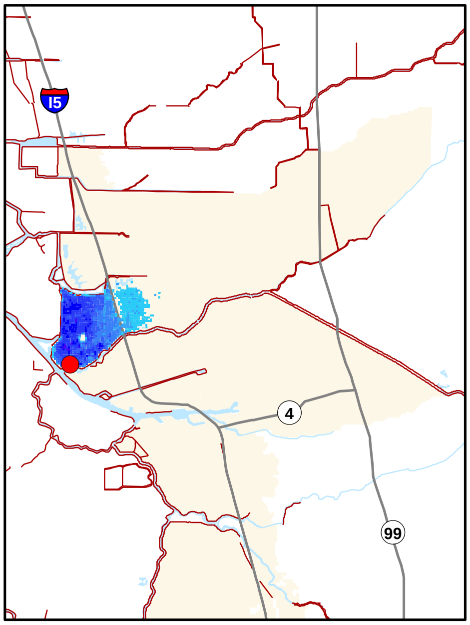
BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-D3

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

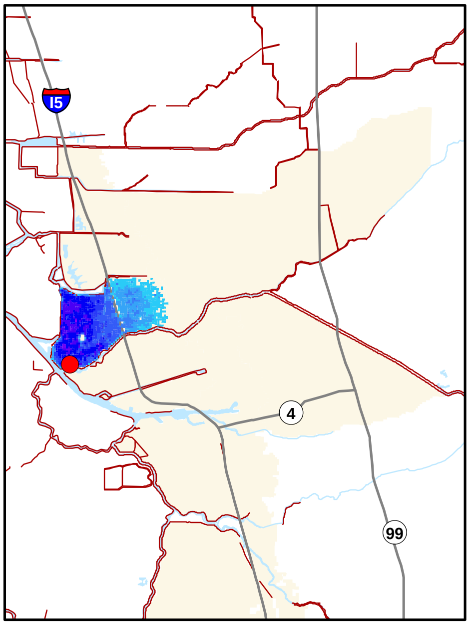
Prepared by C. Young

JUN 2014

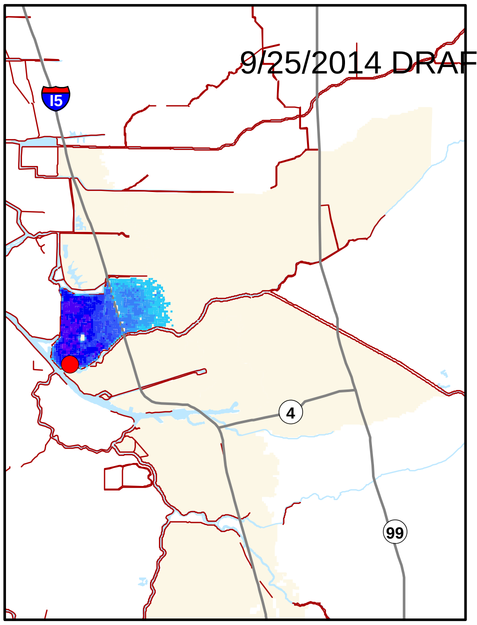
9/25/2014 DRAFT



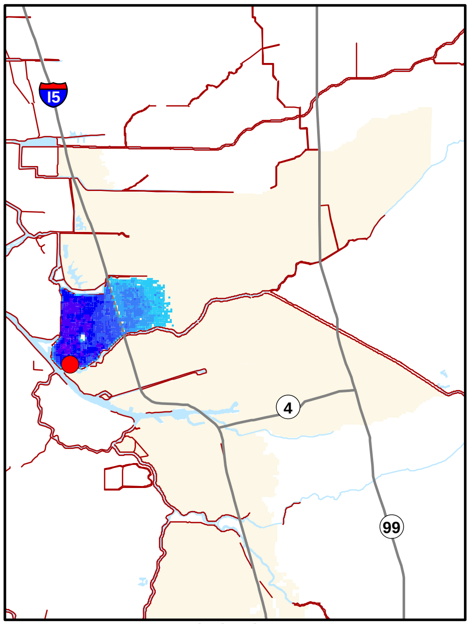
50% (1/2) ACE



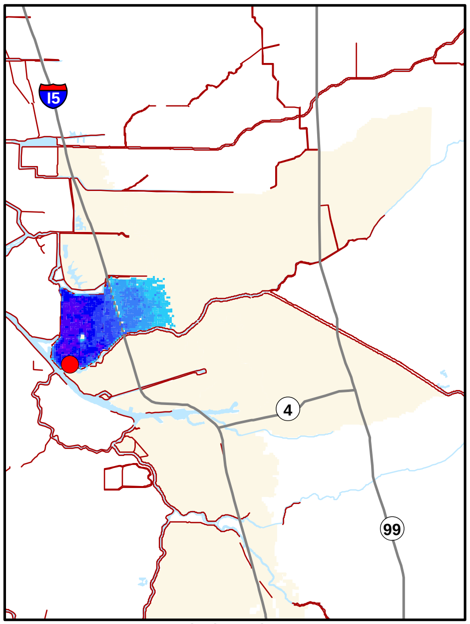
10% (1/10) ACE



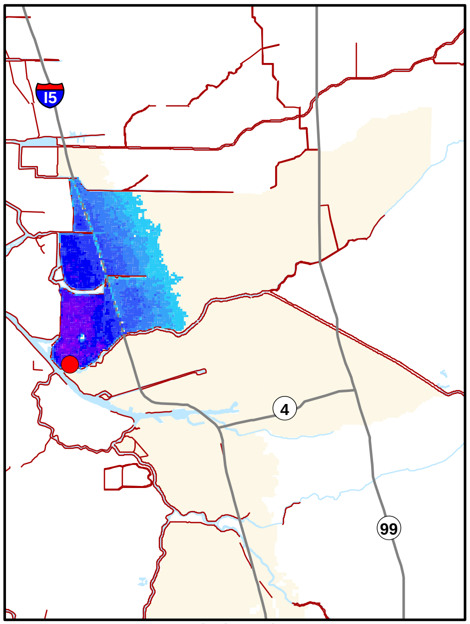
4% (1/25) ACE



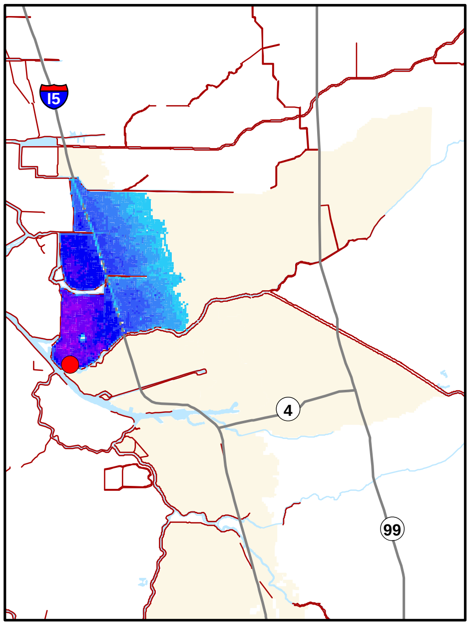
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-D5

Depth (FT)

🟦 0.01 - 2.00
🟦 2.01 - 4.00
🟦 4.01 - 6.00
🟦 6.01 - 8.00
🟦 8.01 - 10.00
🟦 10.01 - 12.00
🟪 12.01 - 14.00
🟪 14.01 - 16.00
🟪 16.01 - 18.00
🟪 18.01 - 20.00
🟪 20.01 - 22.00
🟪 22.01 - 24.00
〰 Levees (CLD, NLD)
▭ LSJ Damage Areas

0 5 Miles

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-D4

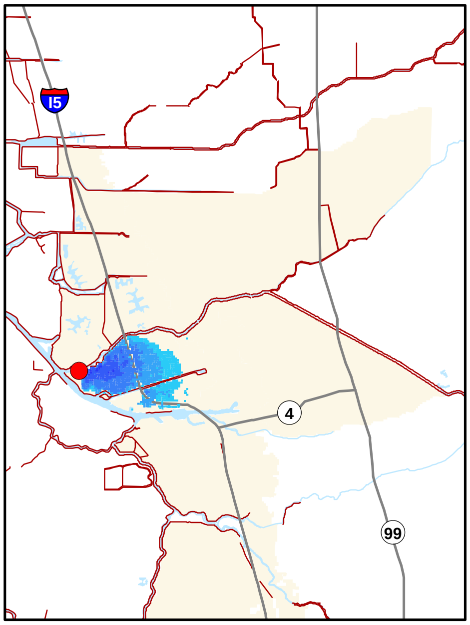
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

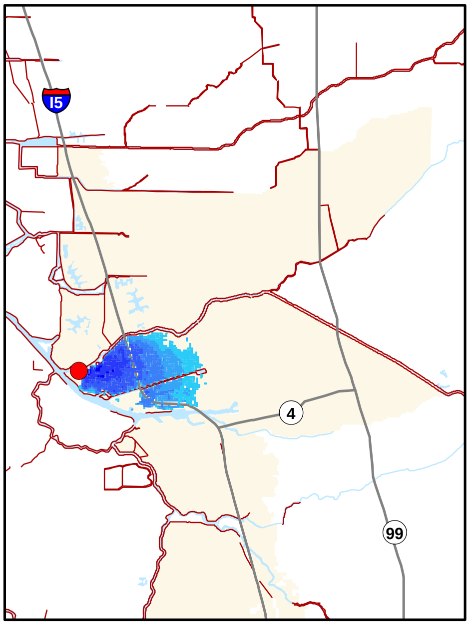
JUN 2014

PLATE 34C

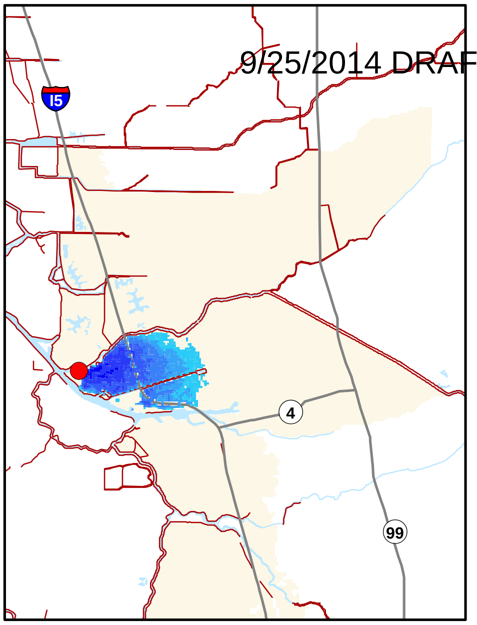
9/25/2014 DRAFT



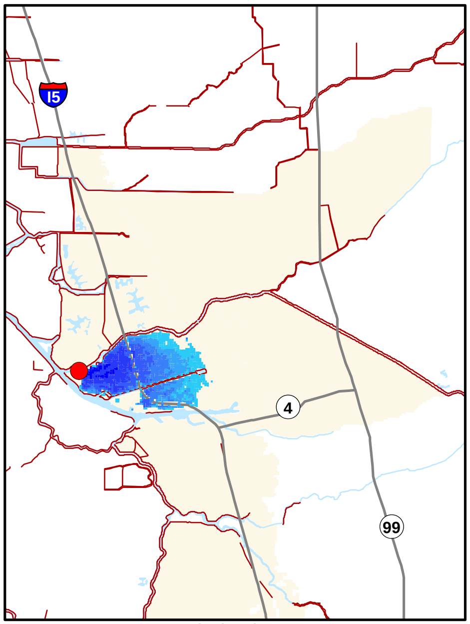
50% (1/2) ACE



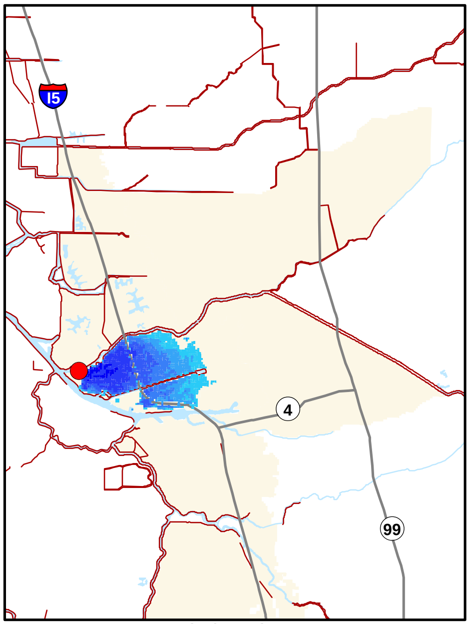
10% (1/10) ACE



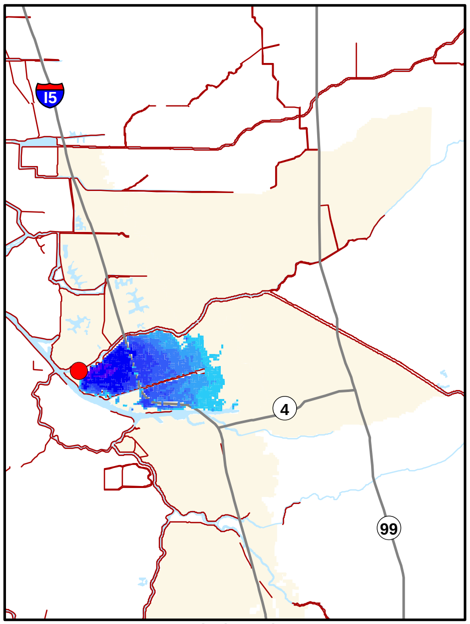
4% (1/25) ACE



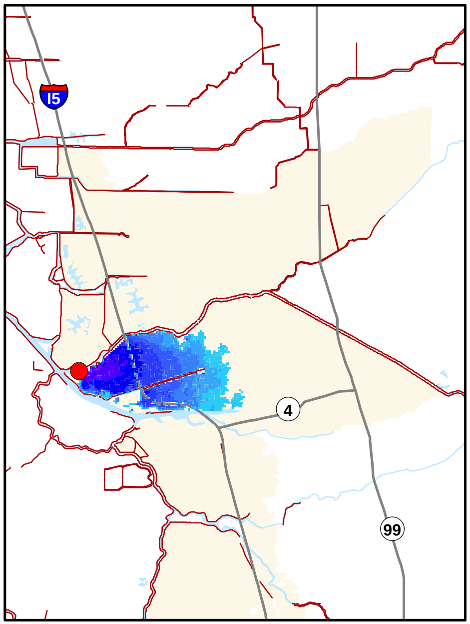
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Breach Location B-D5

Depth (FT)

■ 0.01 - 2.00
■ 2.01 - 4.00
■ 4.01 - 6.00
■ 6.01 - 8.00
■ 8.01 - 10.00
■ 10.01 - 12.00
■ 12.01 - 14.00
■ 14.01 - 16.00
■ 16.01 - 18.00
■ 18.01 - 20.00
■ 20.01 - 22.00
■ 22.01 - 24.00
Levees (CLD, NLD)
■ LSJ Damage Areas

Miles
0 5

N

NOTE: MAP DEPICTS OVERTOPPING
WITHOUT FAILURE IN REACHES
WITHOUT A BREACH

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

BREACH SIMULATION
ALTERNATIVE 1 - NO ACTION
LOCATION B-D5

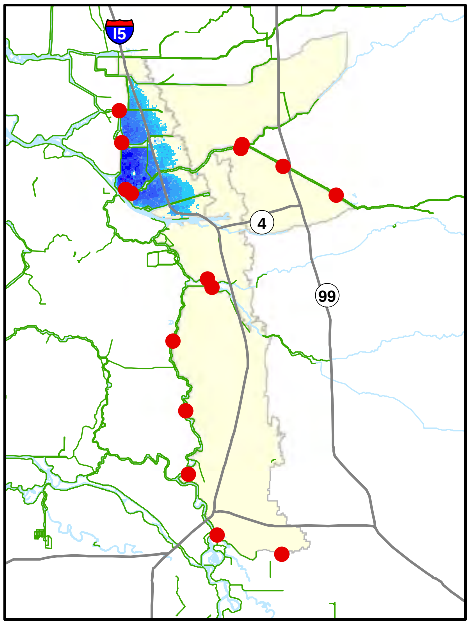
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

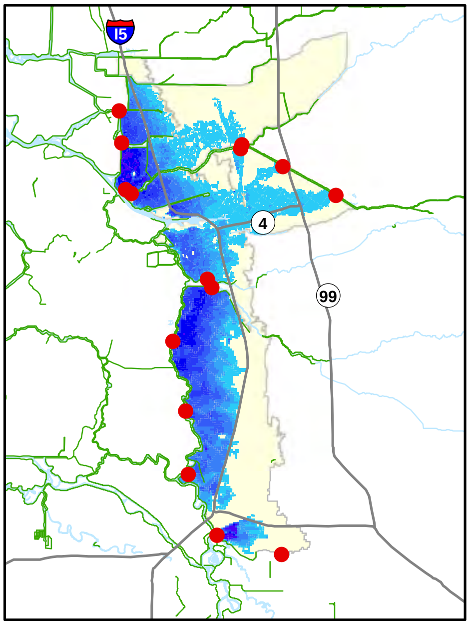
JUN 2014

PLATE 34D

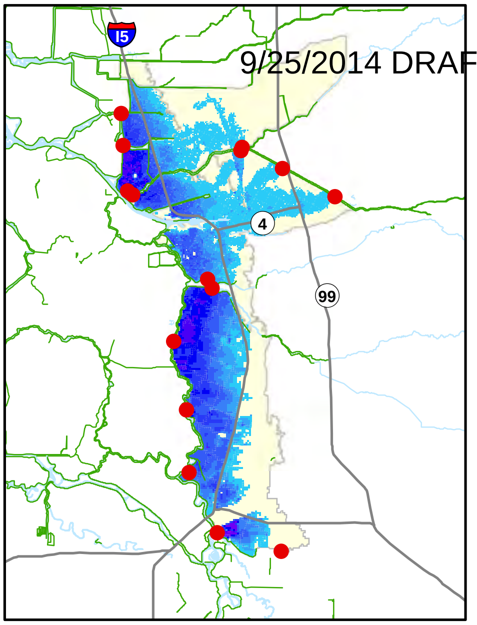
9/25/2014 DRAFT



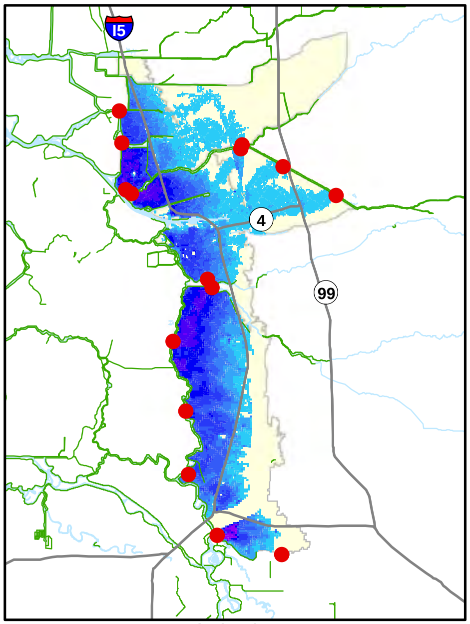
50% (1/2) ACE



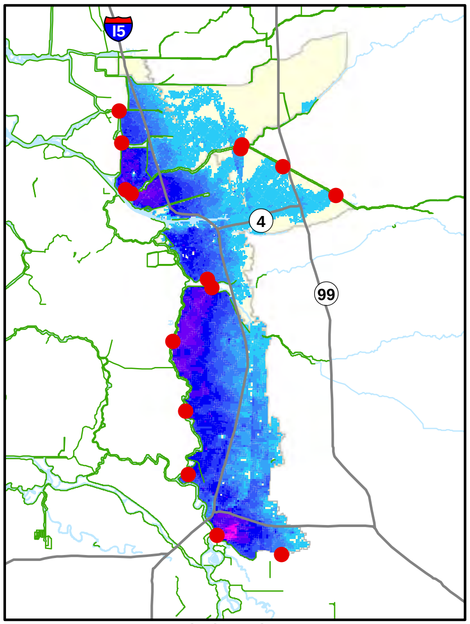
10% (1/10) ACE



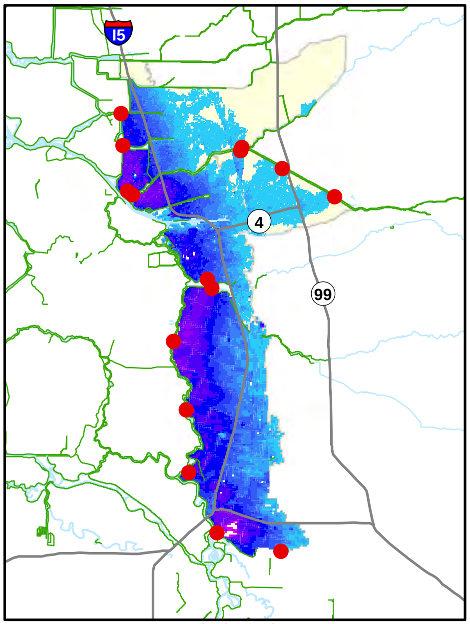
4% (1/25) ACE



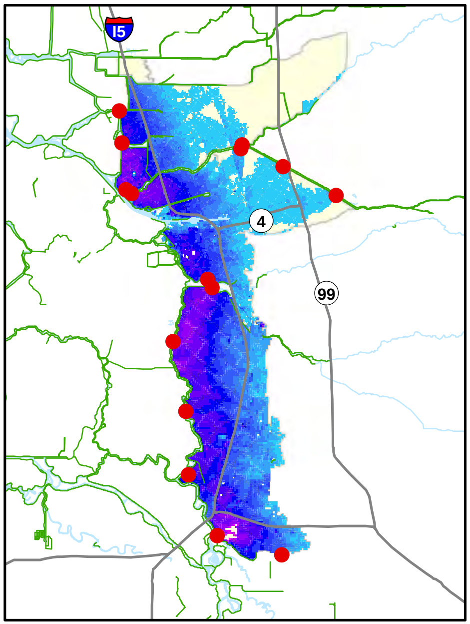
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Fails R&U Criteria
○ Meets R&U Criteria

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00
- Levees (CLD, NLD)
- LSJ Study Extent

Miles
0 5

N

NOTE: All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within Study Extent

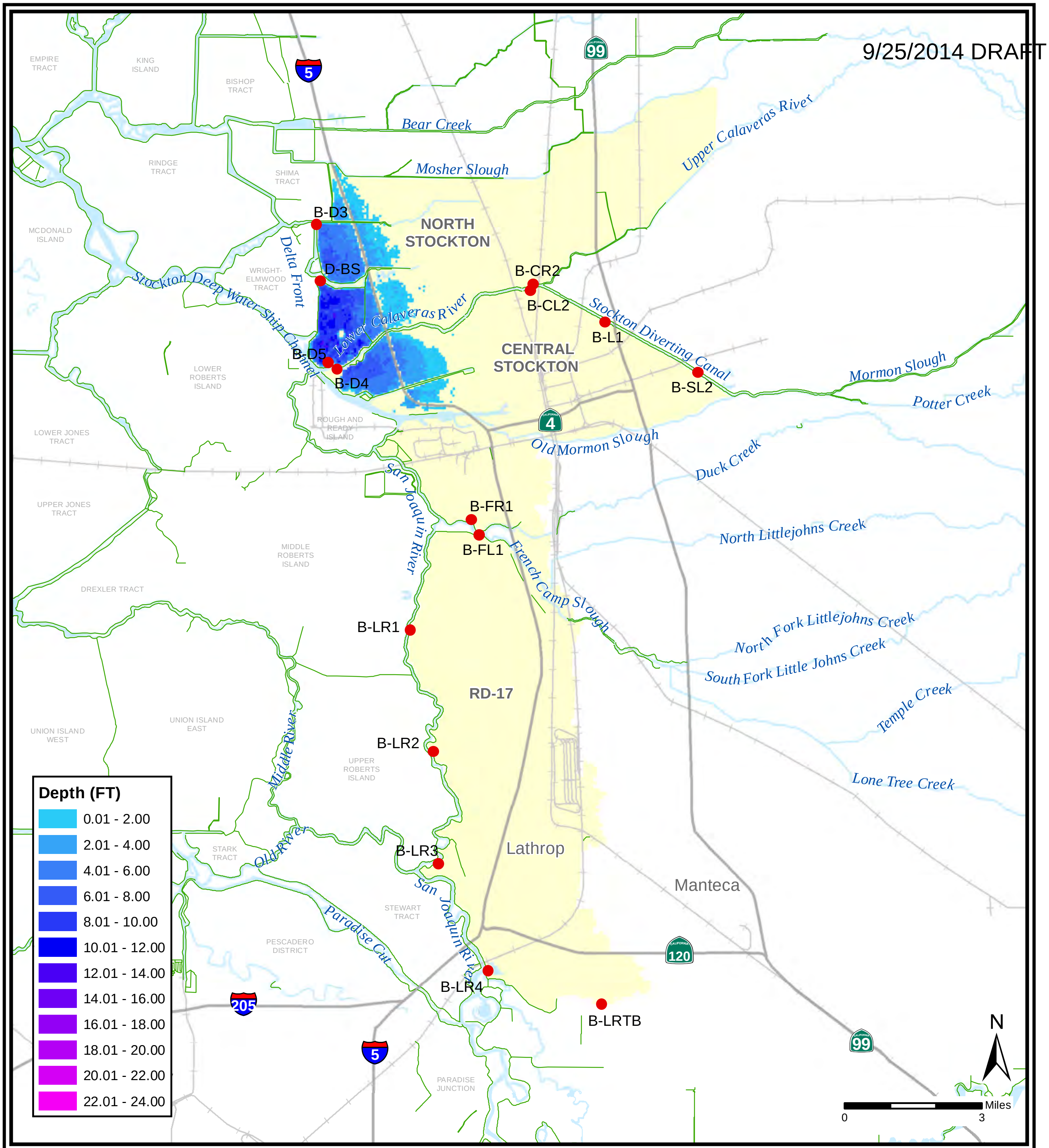
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NATURAL COMPOSITE FLOODPLAIN
ALTERNATIVE - 1
NO ACTION

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014



NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NATURAL COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
50% (1/2) ACE

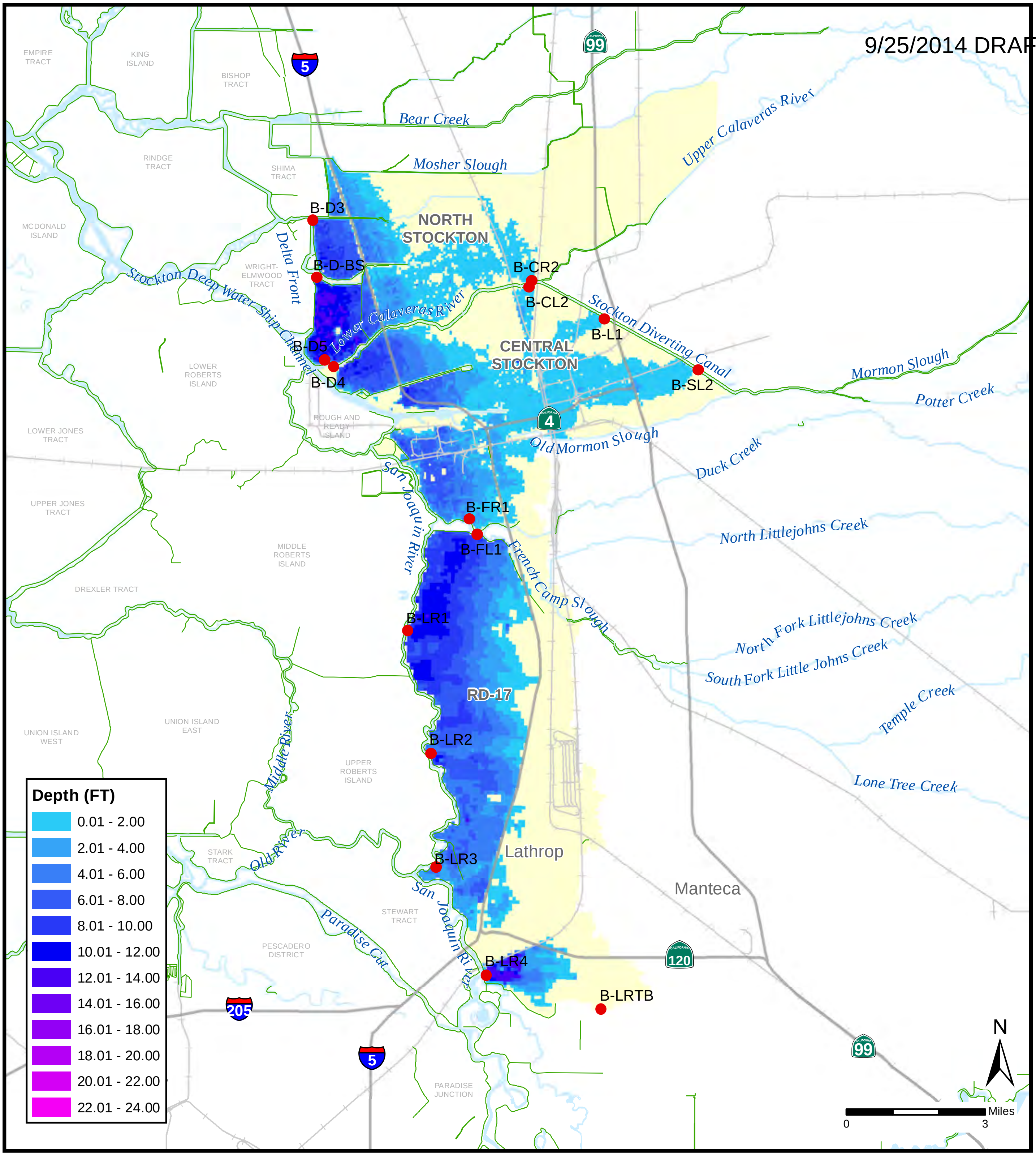
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 36

9/25/2014 DRAFT



NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

Imagery Source: 2012 NAIP, 1m

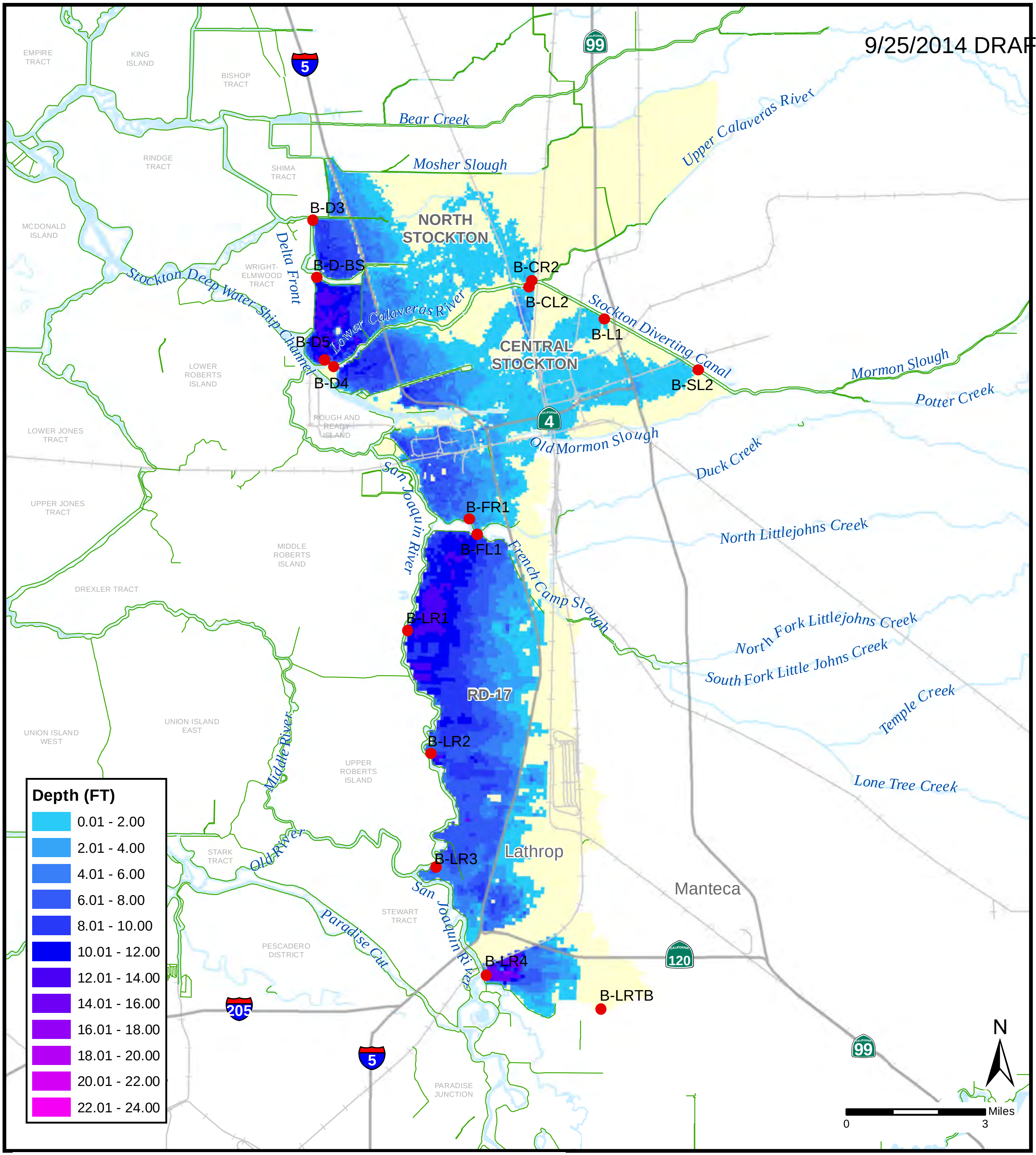
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NATURAL COMPOSITE FLOODPLAIN ALTERNATIVE 1 - NO ACTION 10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NATURAL COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
4% (1/25) ACE

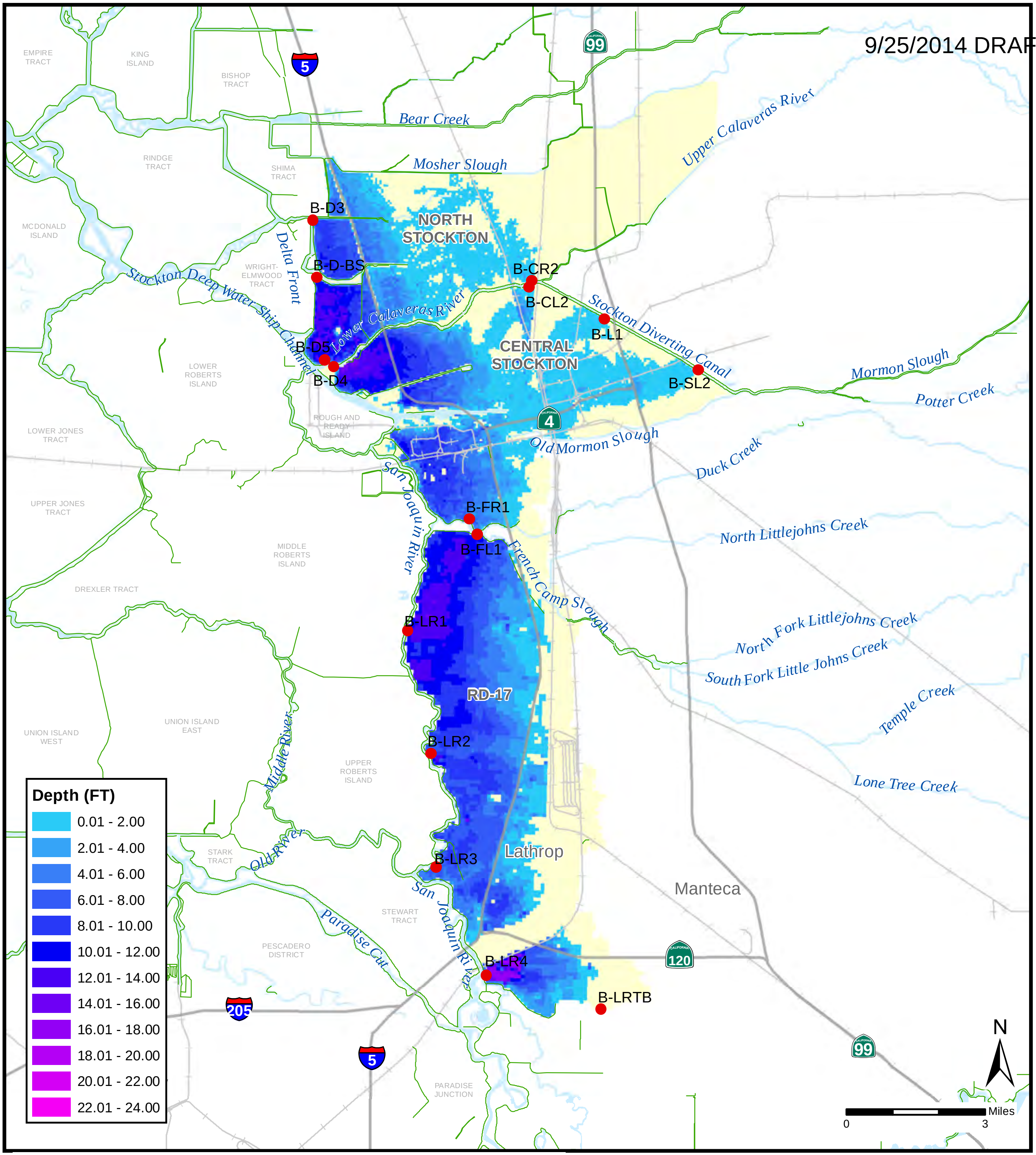
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 38

9/25/2014 DRAFT



Depth (FT)

🟦	0.01 - 2.00
🟦	2.01 - 4.00
🟦	4.01 - 6.00
🟦	6.01 - 8.00
🟦	8.01 - 10.00
🟦	10.01 - 12.00
🟪	12.01 - 14.00
🟪	14.01 - 16.00
🟪	16.01 - 18.00
🟪	18.01 - 20.00
🟪	20.01 - 22.00
🟪	22.01 - 24.00

NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

Imagery Source: 2012 NAIP, 1m

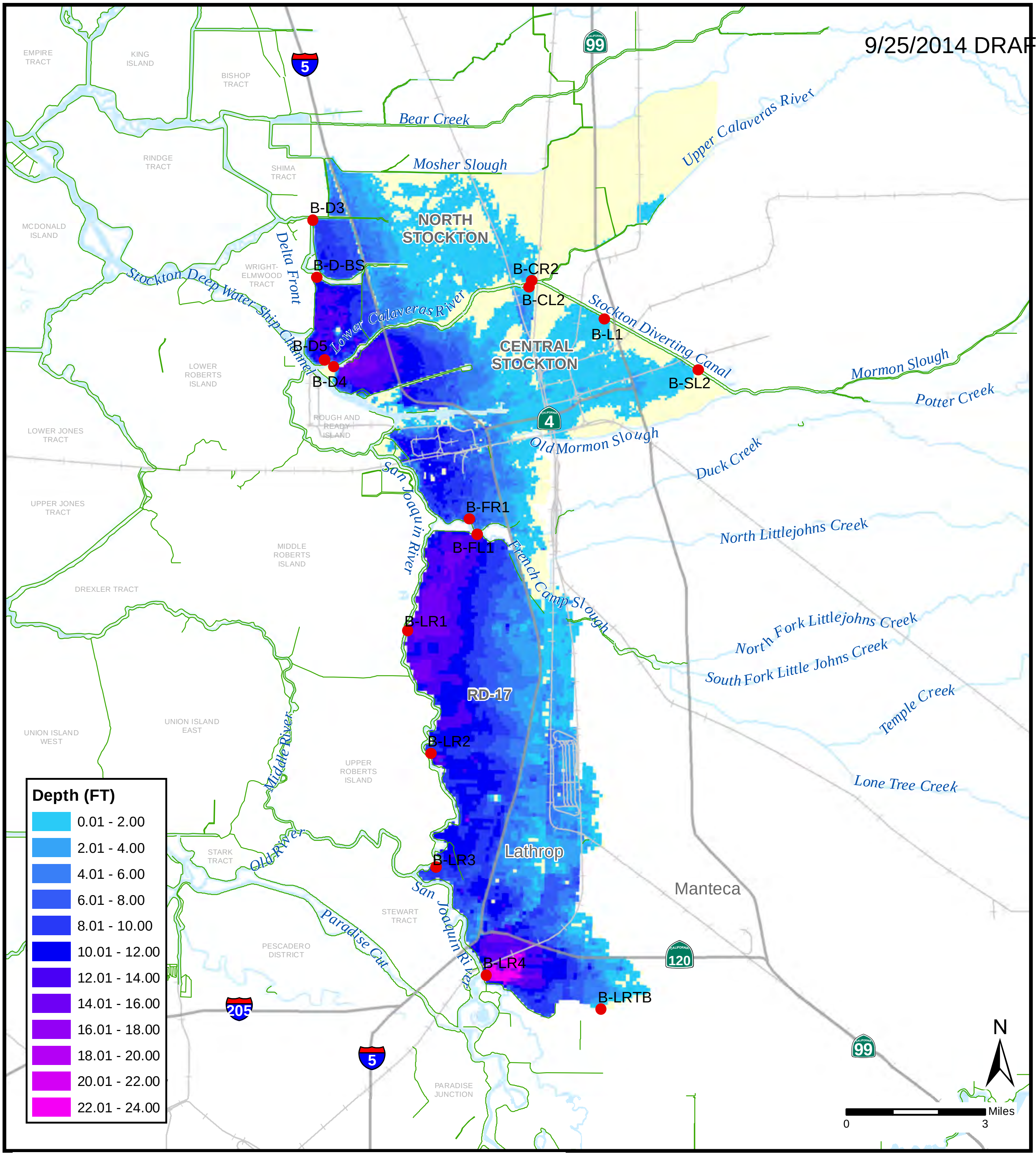
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

NATURAL COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
2% (1/50) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

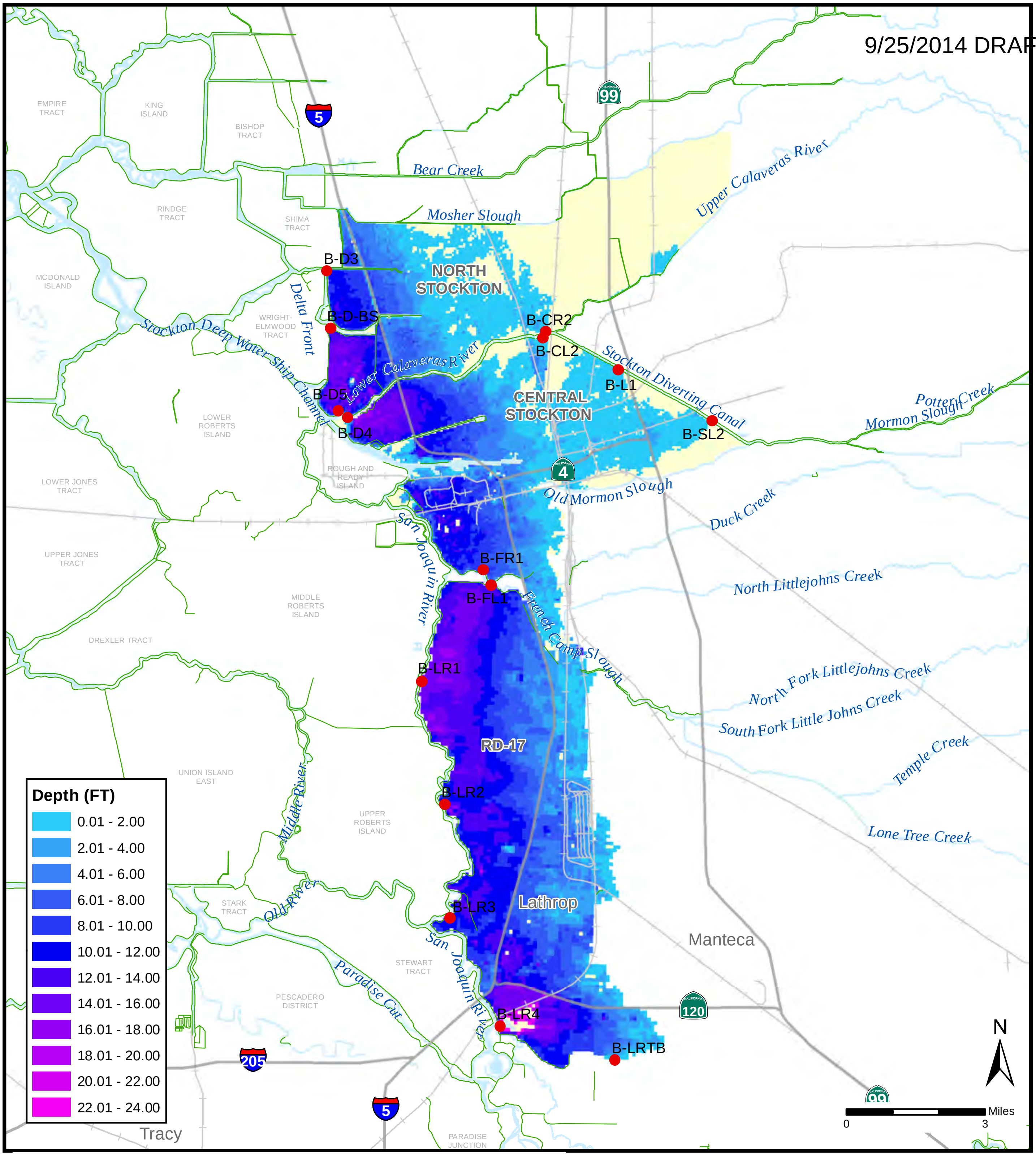
NATURAL COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
1% (1/100) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

9/25/2014 DRAFT



NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

**SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY**

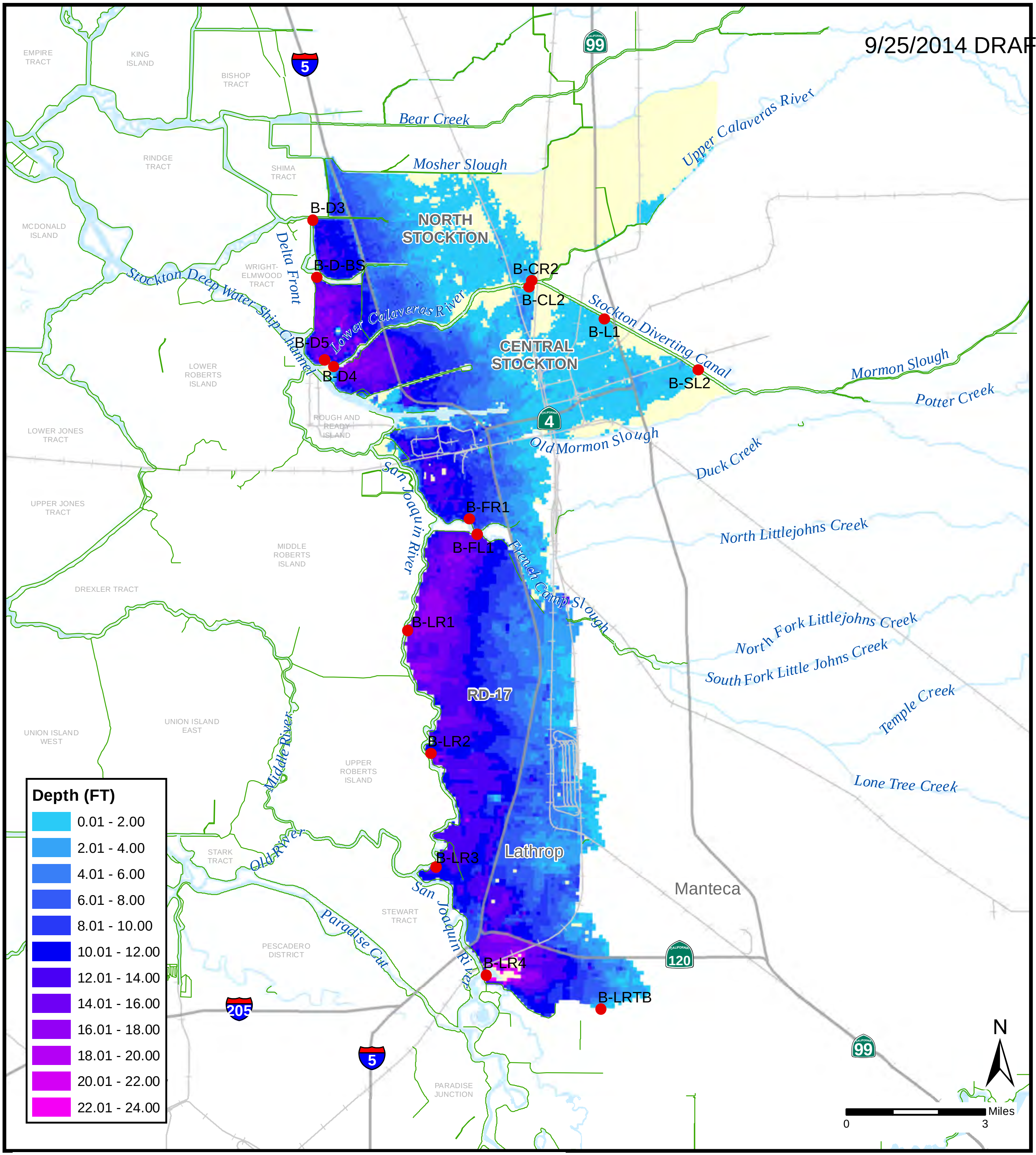
NATURAL COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
0.5% (1/200) ACE

**U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT**

Prepared by V. Nino-Tapia

JUN 2014

PLATE 41



NOTE: Map intended to show the potential floodwater depth from a breach along any of the principle flood sources identified in this study. All breach simulations shown regarless of levee performance.

Composite Floodplains only shown within study extent (yellow area).

● Levee Breach Included

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

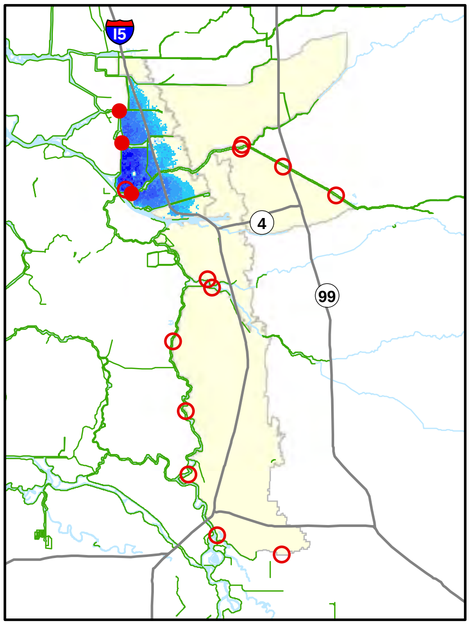
NATURAL COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
0.2% (1/500) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

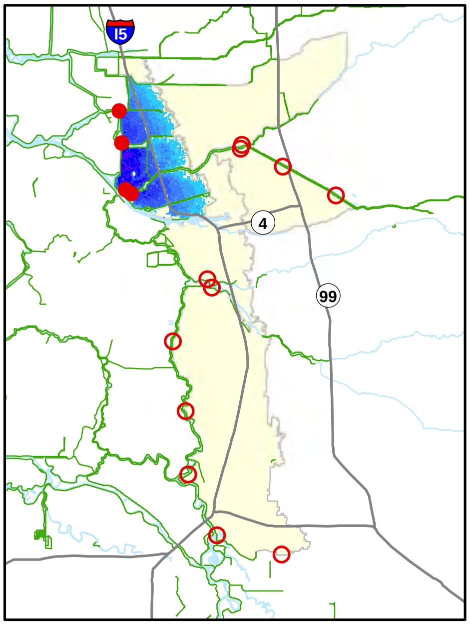
Prepared by V. Nino-Tapia

JUN 2014

PLATE 42

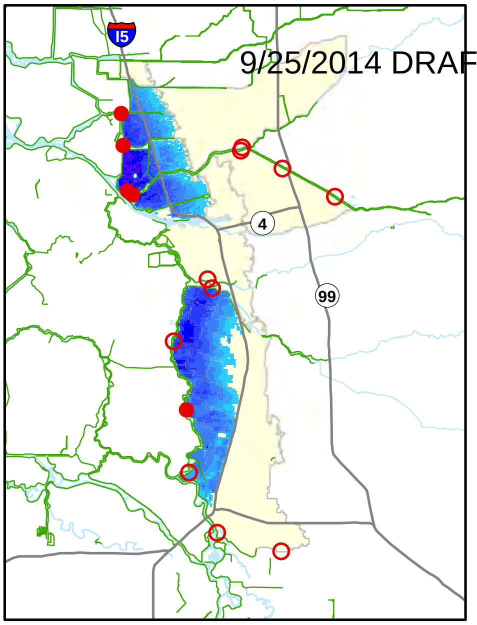


50% (1/2) ACE

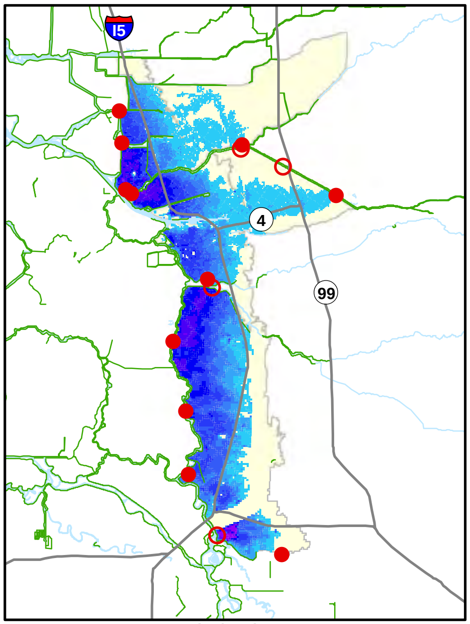


10% (1/10) ACE

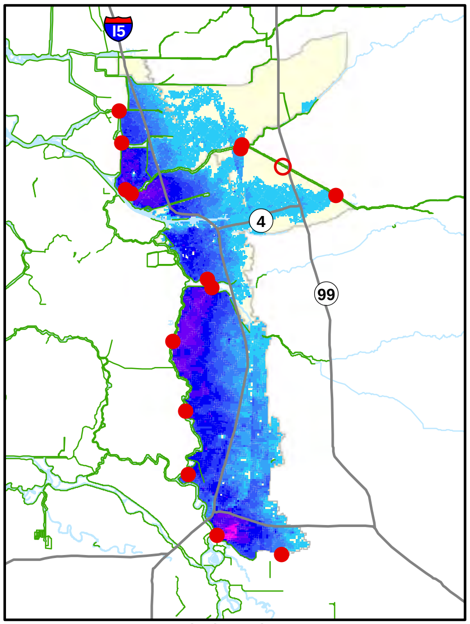
9/25/2014 DRAFT



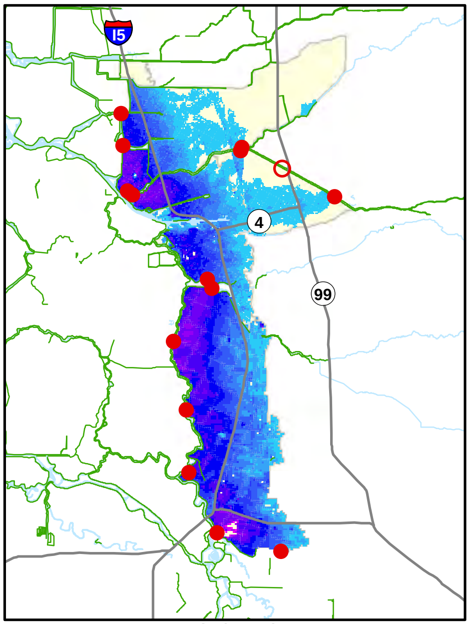
4% (1/25) ACE



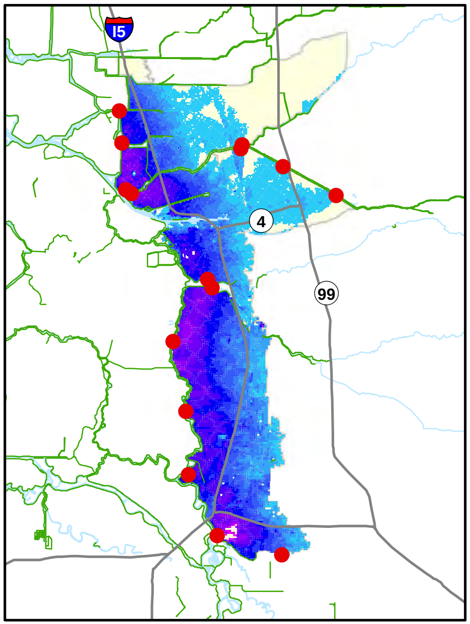
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Fails R&U Criteria
○ Meets R&U Criteria

Depth (FT)

🟦 0.01 - 2.00
🟦 2.01 - 4.00
🟦 4.01 - 6.00
🟦 6.01 - 8.00
🟦 8.01 - 10.00
🟦 10.01 - 12.00
🟪 12.01 - 14.00
🟪 14.01 - 16.00
🟪 16.01 - 18.00
🟪 18.01 - 20.00
🟪 20.01 - 22.00
🟪 22.01 - 24.00
— Levees (CLD, NLD)
▭ LSJ Study Extent

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

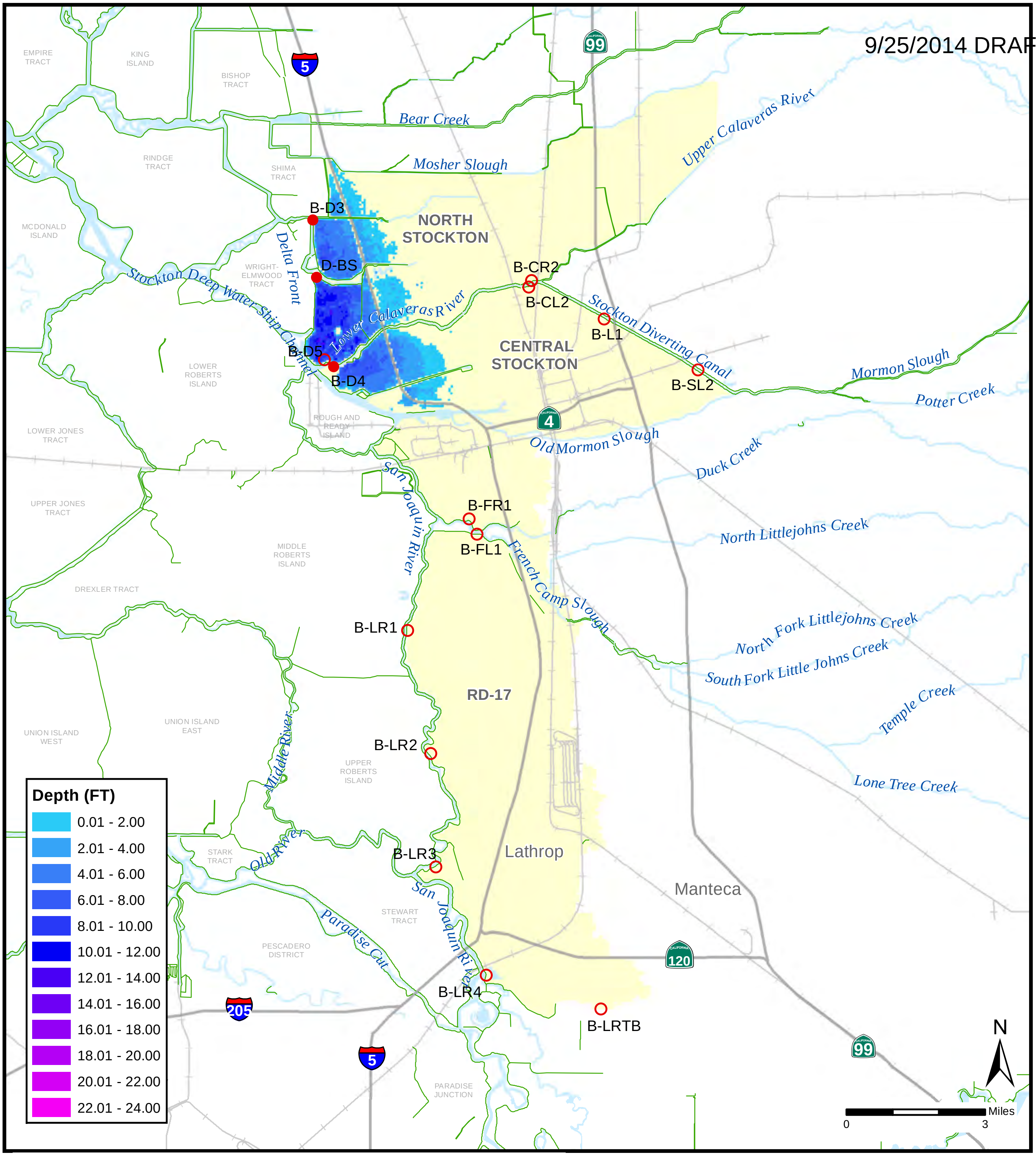
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 1
NO ACTION

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014



Legend

— Levees (CLD, NLD)
╱ Highway
┼ Railroads

● Fails R&U Criteria
○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

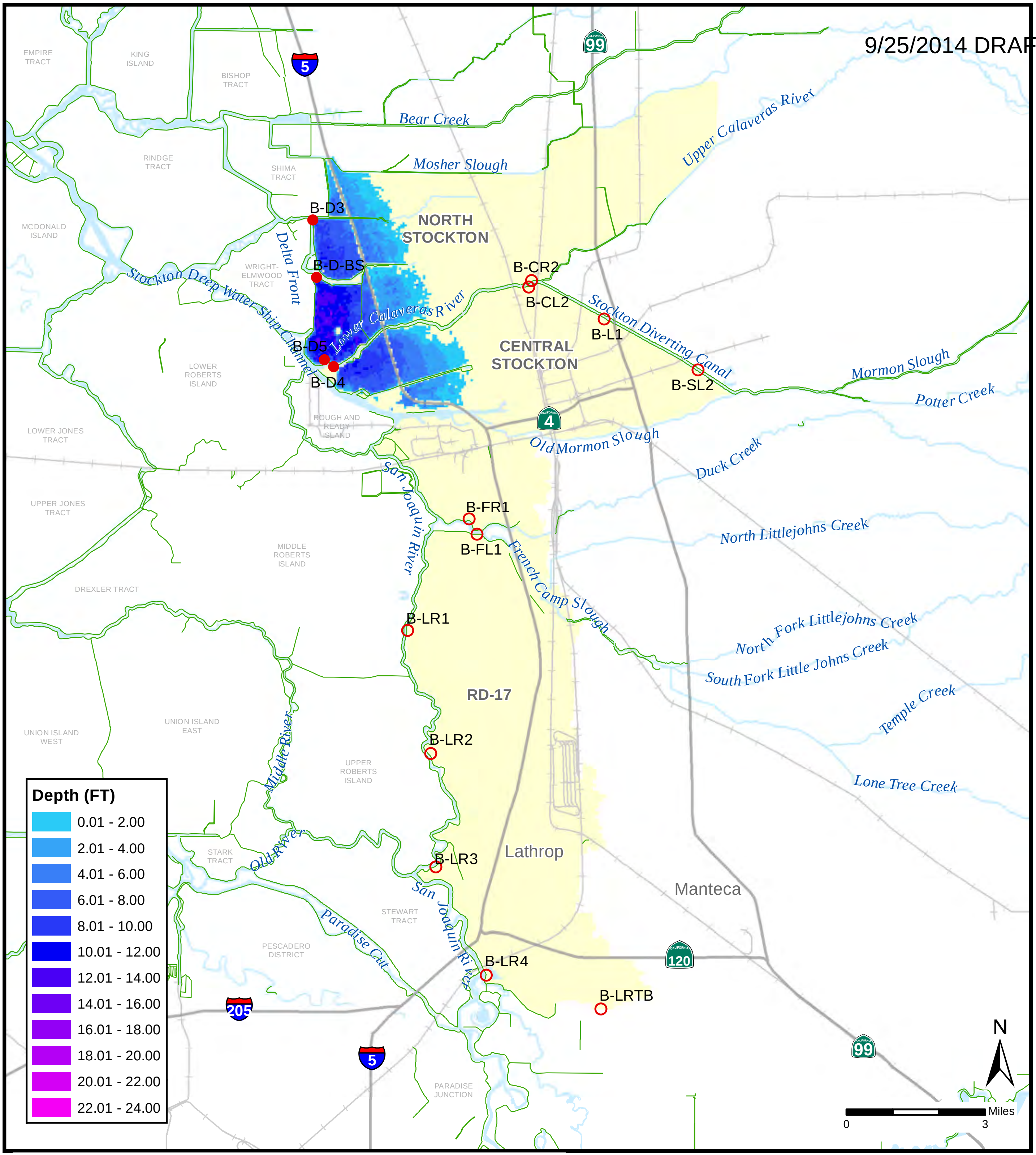
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
50% (1/2) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 44



Legend

Levees (CLD, NLD)
Highway
Railroads

● Fails R&U Criteria
○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

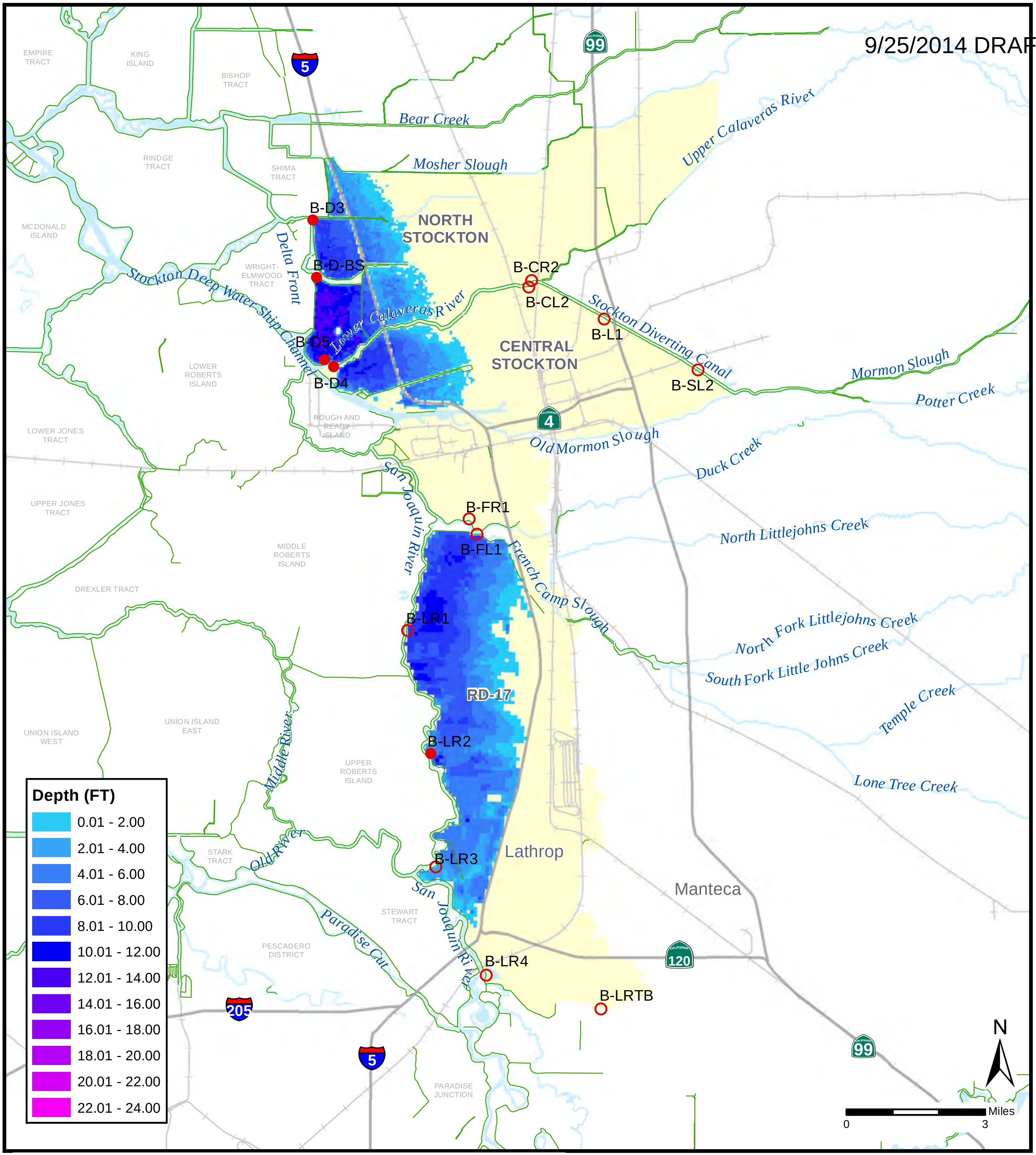
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 45



Legend

- Levees (CLD, NLD)
- Highway
- Railroads

● Fails R&U Criteria

○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
4% (1/25) ACE

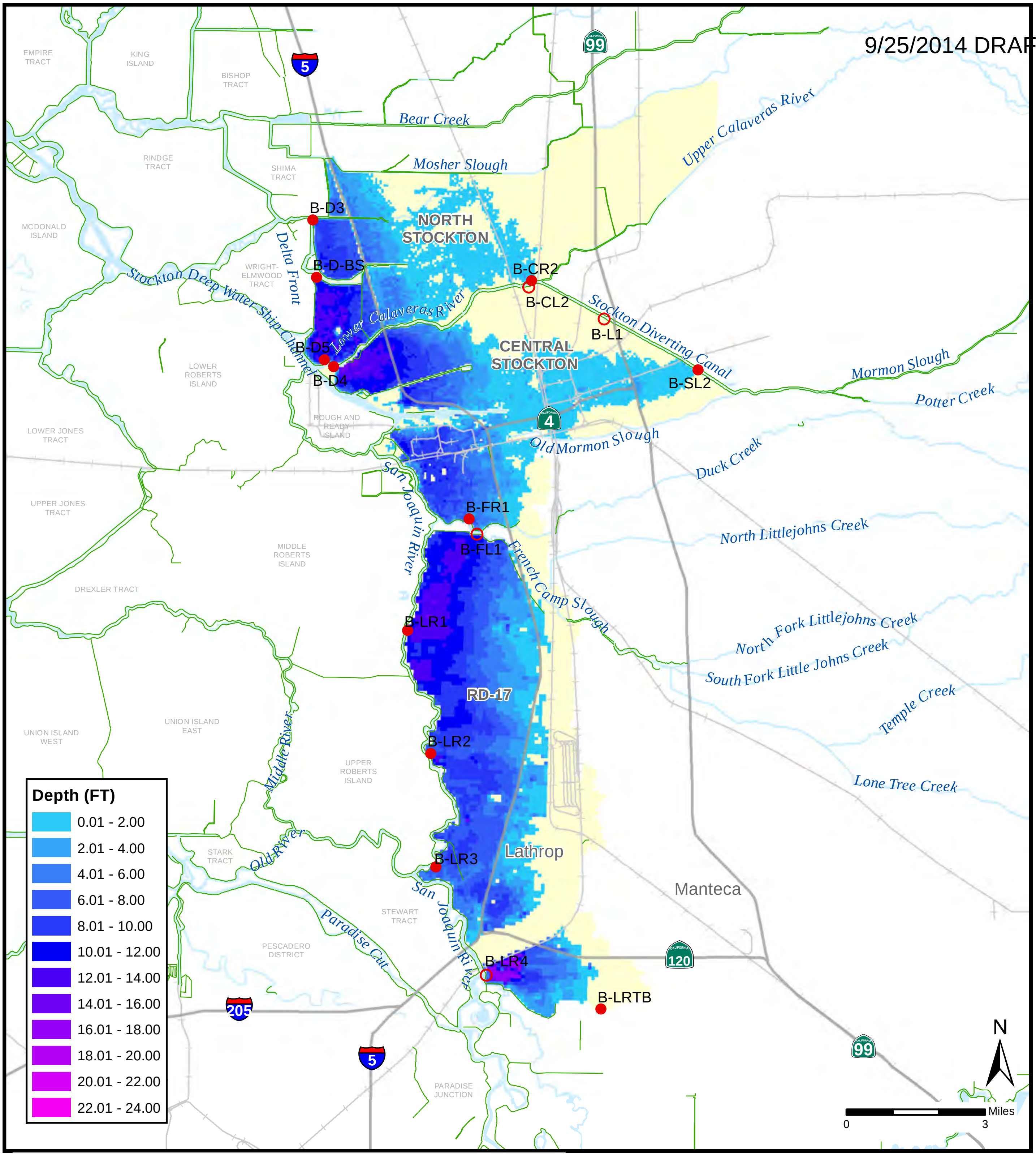
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 46

9/25/2014 DRAFT



Legend

Levees (CLD, NLD)

Highway

Railroads

● Fails R&U Criteria

○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
2% (1/50) ACE

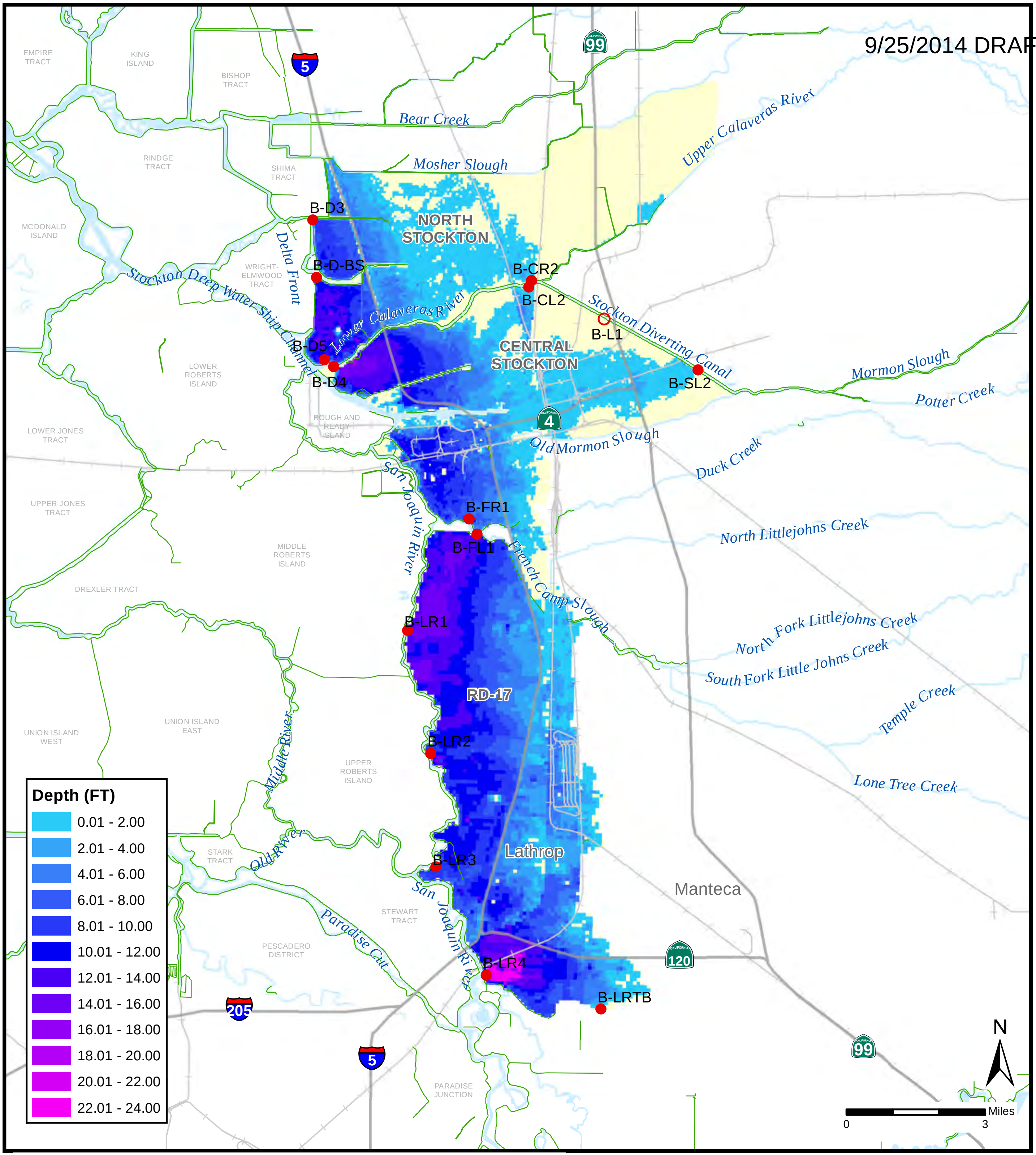
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 47

9/25/2014 DRAFT



Legend

- Levees (CLD, NLD)
- Highway
- Railroads
- ● Fails R&U Criteria
- ○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

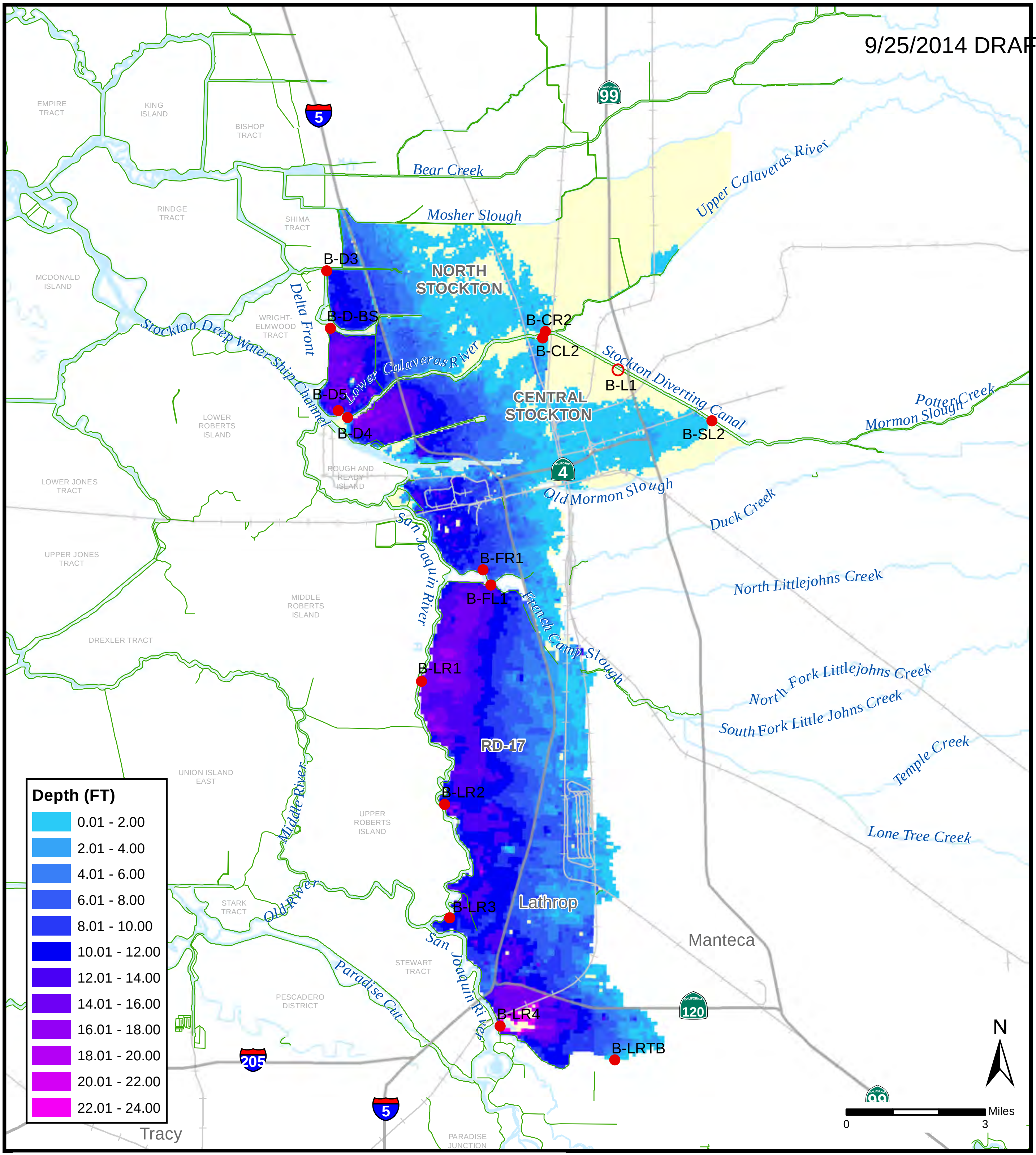
R&U COMPOSITE FLOODPLAIN ALTERNATIVE 1 - NO ACTION 1% (1/100) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

9/25/2014 DRAFT



Legend

Levees (CLD, NLD)
Highway
Railroads

● Fails R&U Criteria
○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

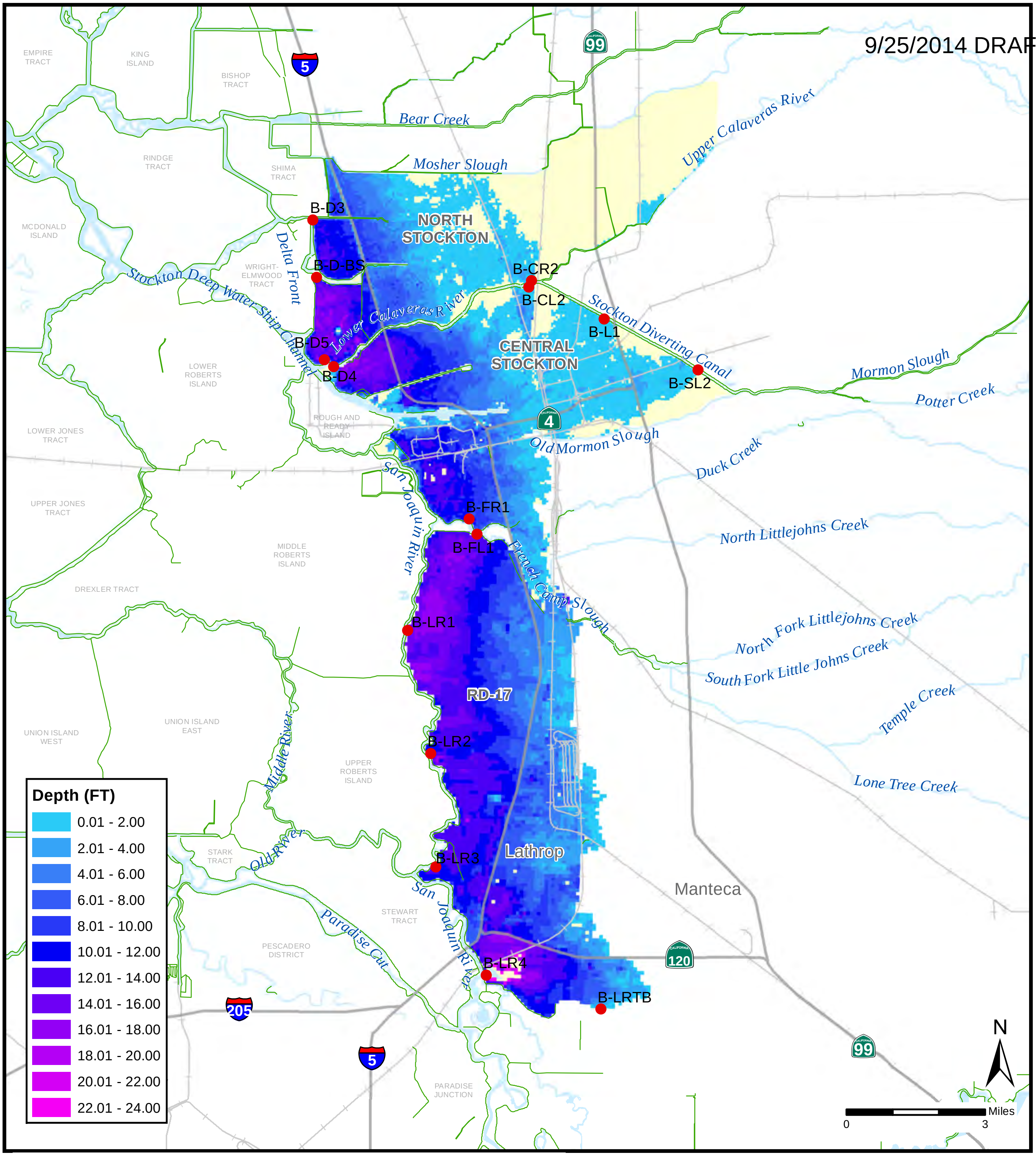
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE 1 - NO ACTION
0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



Legend

Levees (CLD, NLD)

Highway

Railroads

● Fails R&U Criteria

○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

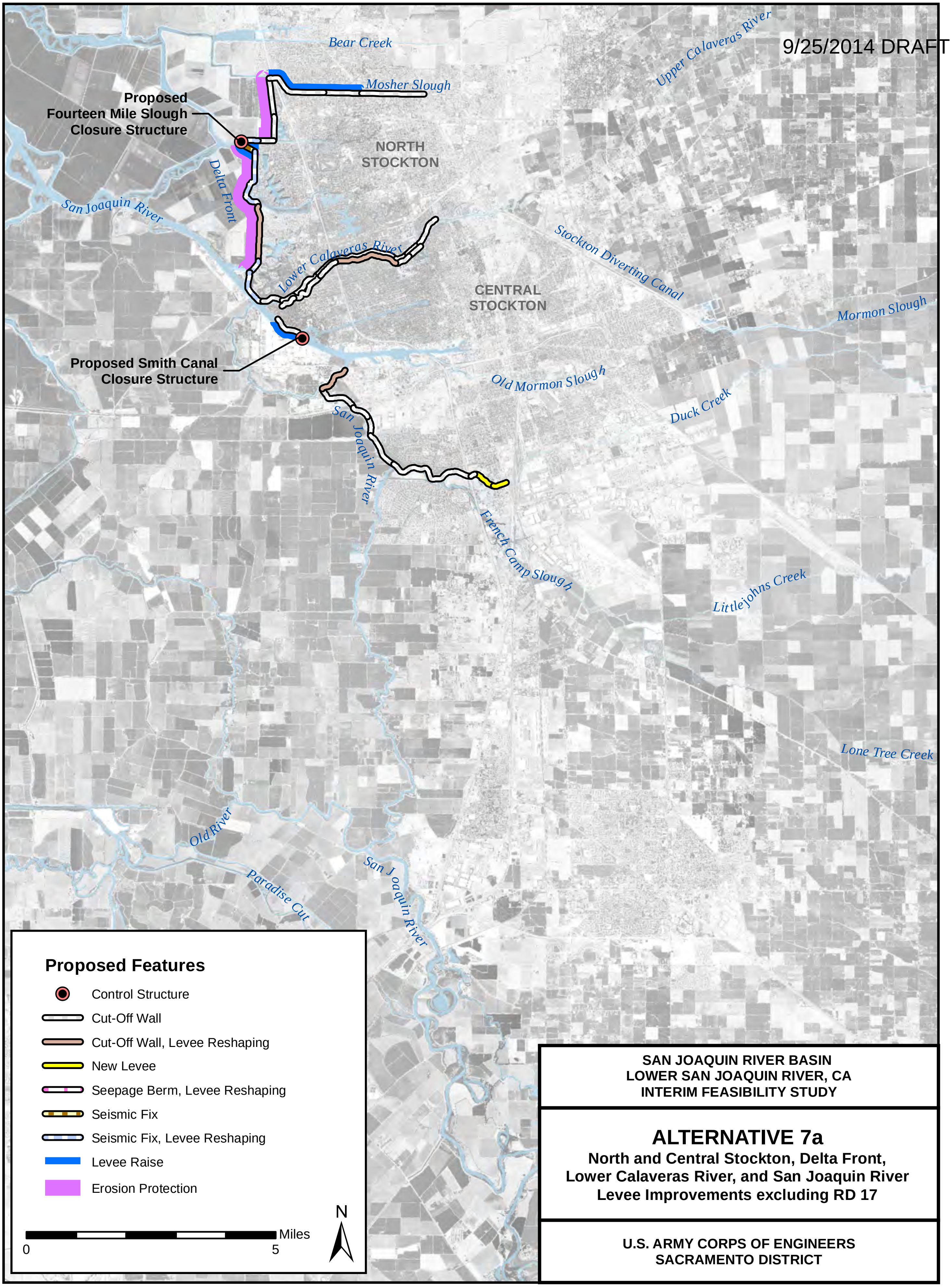
R&U COMPOSITE FLOODPLAIN ALTERNATIVE 1 - NO ACTION 0.2% (1/500) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

9/25/2014 DRAFT



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

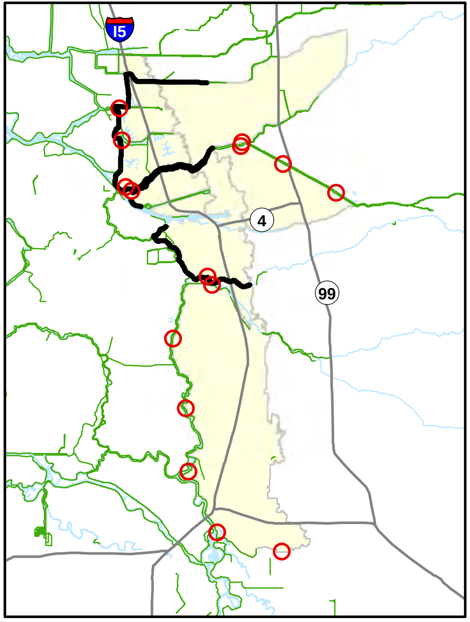
ALTERNATIVE 7a

**North and Central Stockton, Delta Front,
Lower Calaveras River, and San Joaquin River
Levee Improvements excluding RD 17**

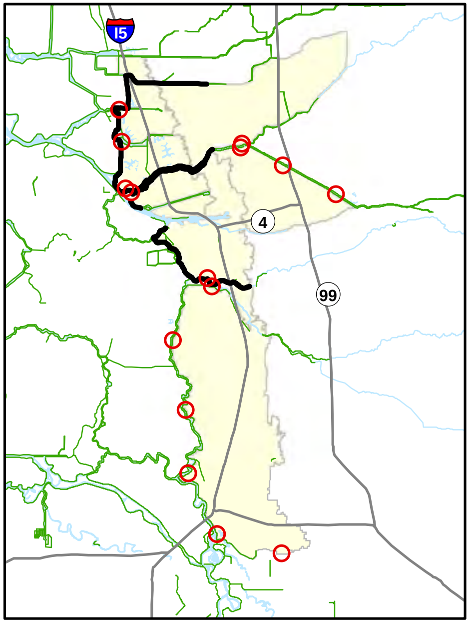
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014

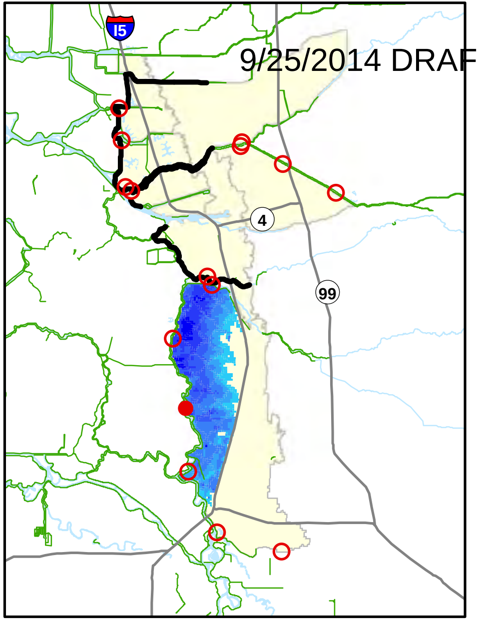
PLATE 51



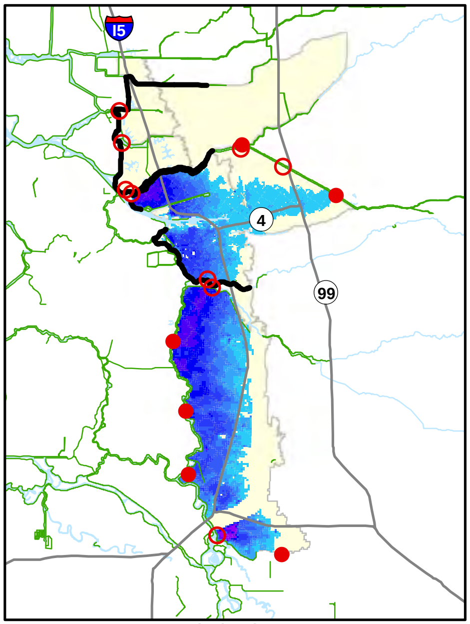
50% (1/2) ACE



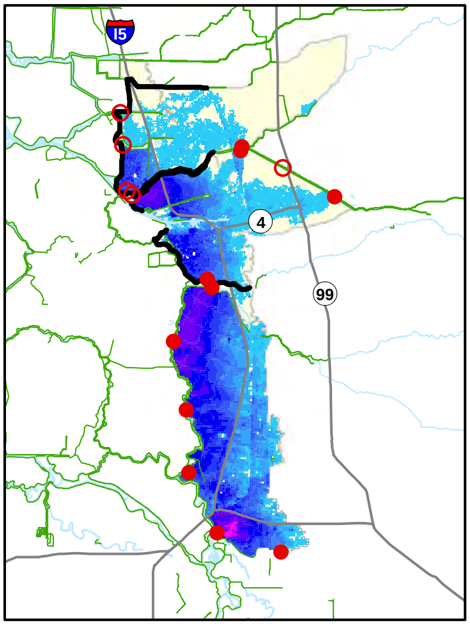
10% (1/10) ACE



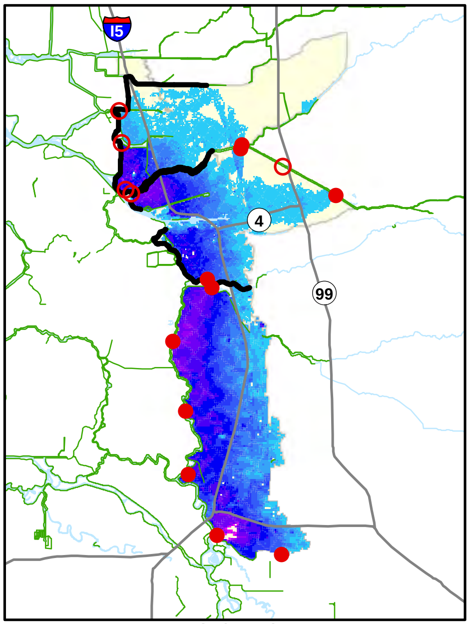
4% (1/25) ACE



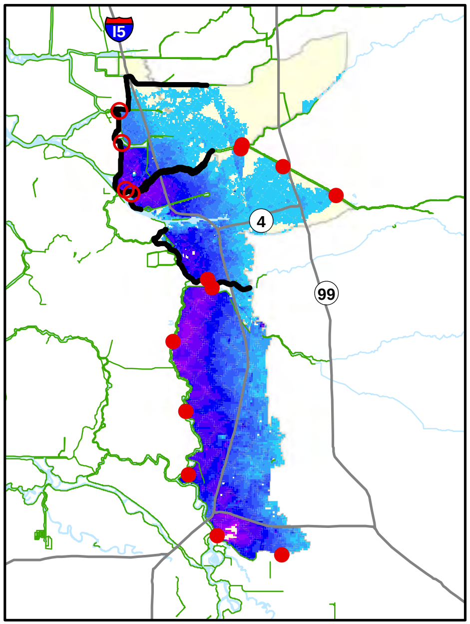
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Fails R&U Criteria
○ Meets R&U Criteria

Depth (FT)

0.01 - 2.00
2.01 - 4.00
4.01 - 6.00
6.01 - 8.00
8.01 - 10.00
10.01 - 12.00
12.01 - 14.00
14.01 - 16.00
16.01 - 18.00
18.01 - 20.00
20.01 - 22.00
22.01 - 24.00
Levees (CLD, NLD)
LSJ Study Extent
Project Features

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7A

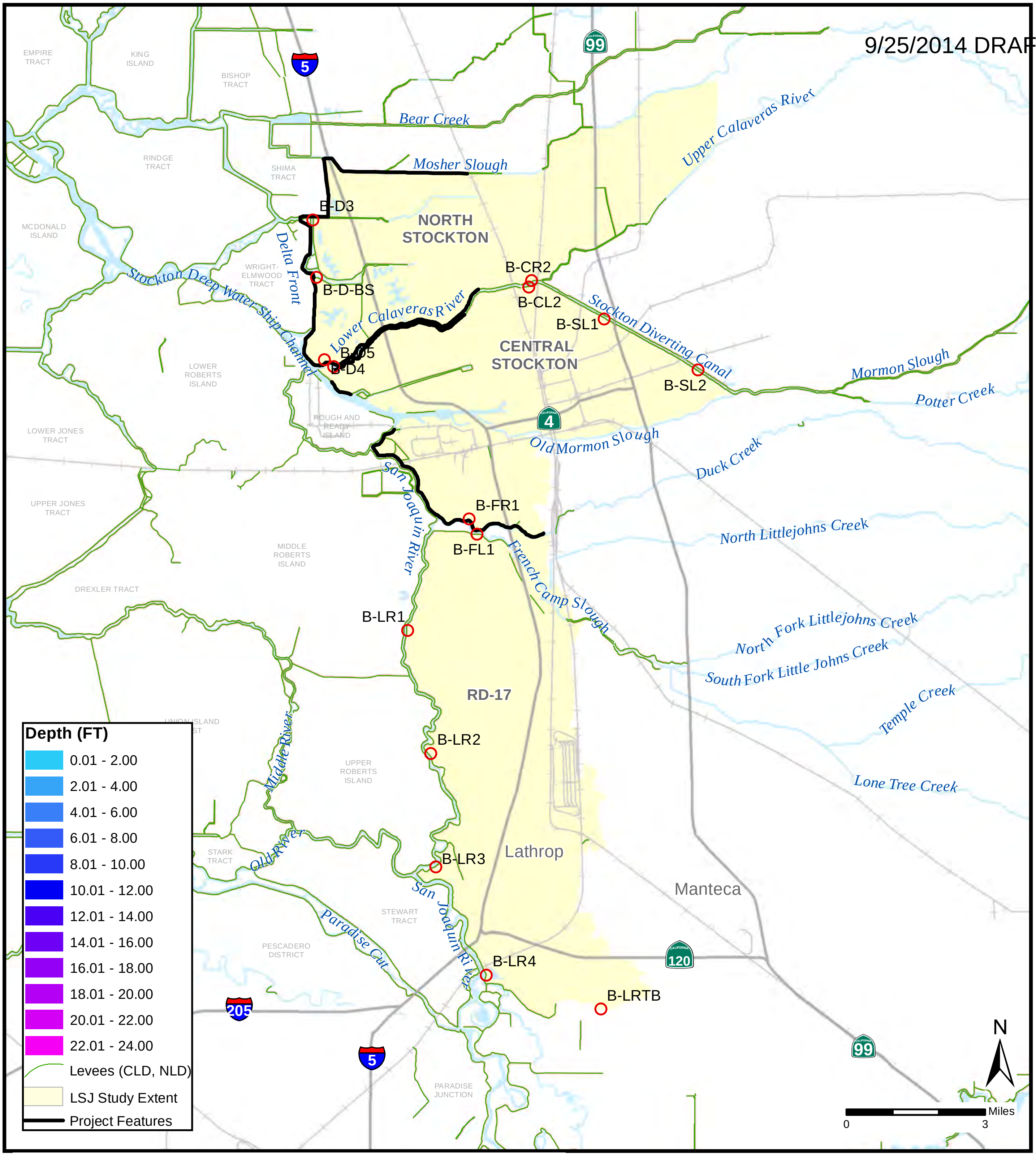
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 52

9/25/2014 DRAFT



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

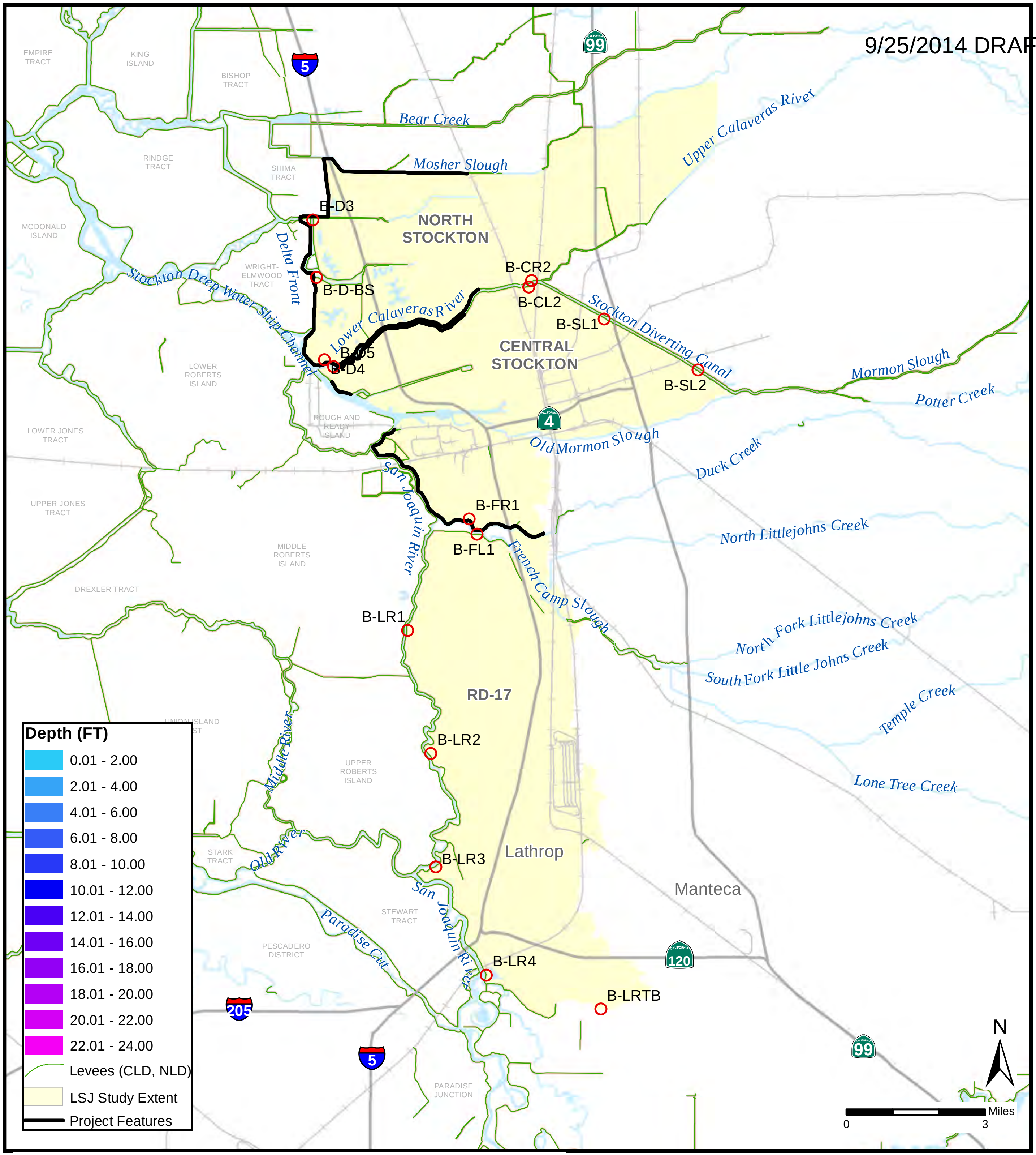
R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7A 50% (1/2) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 53



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

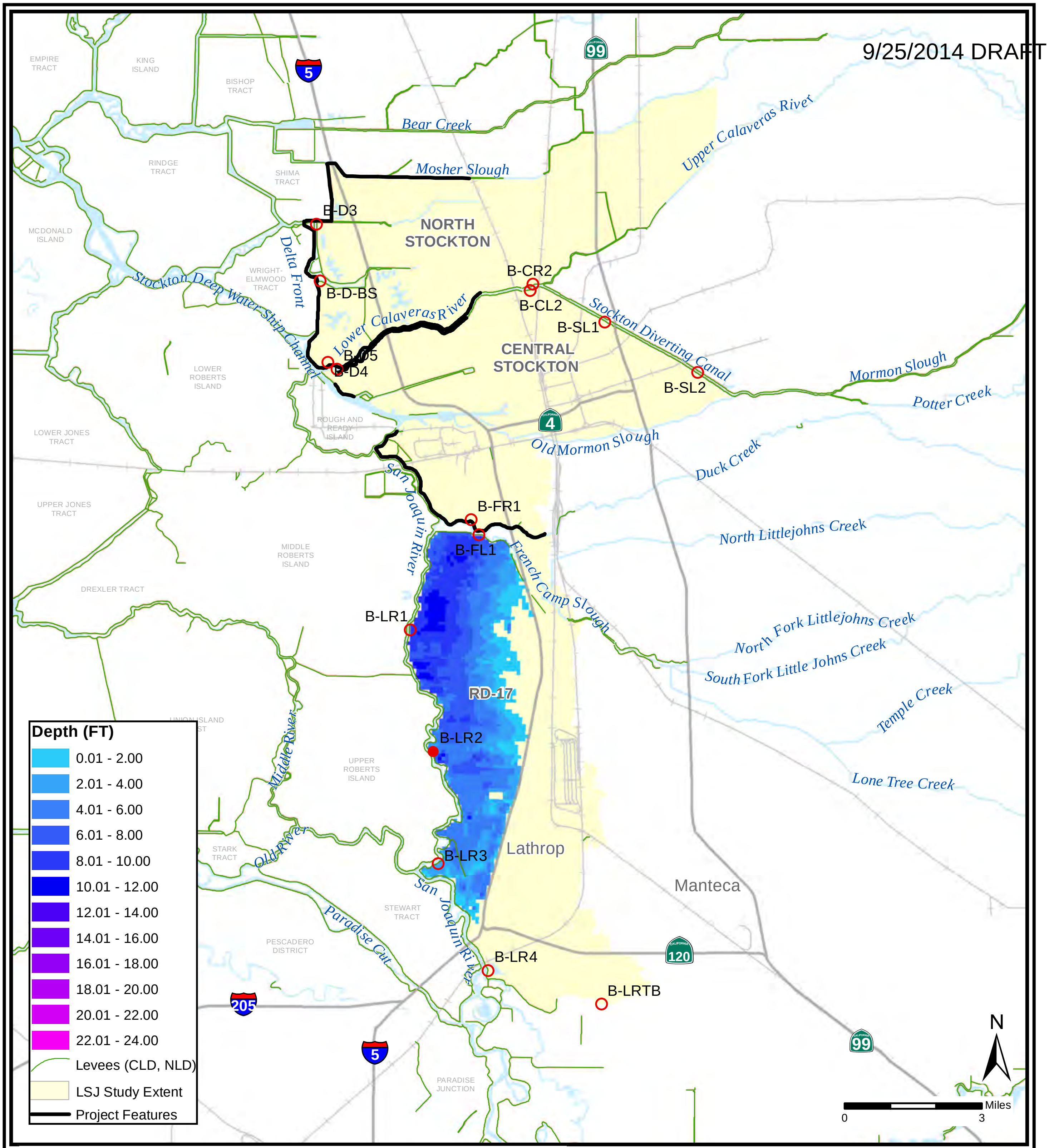
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 7A
10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 54



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

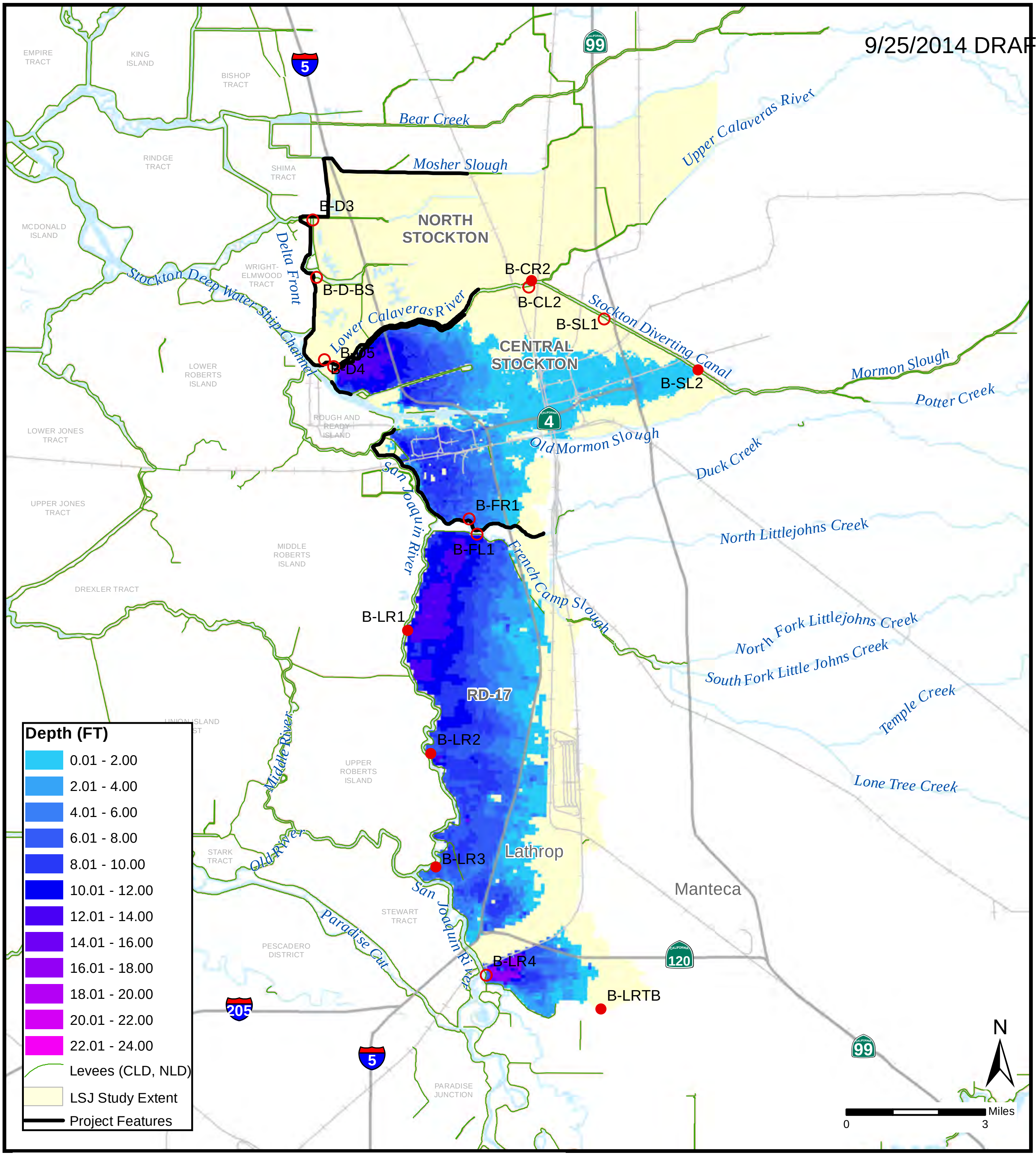
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 7A
4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

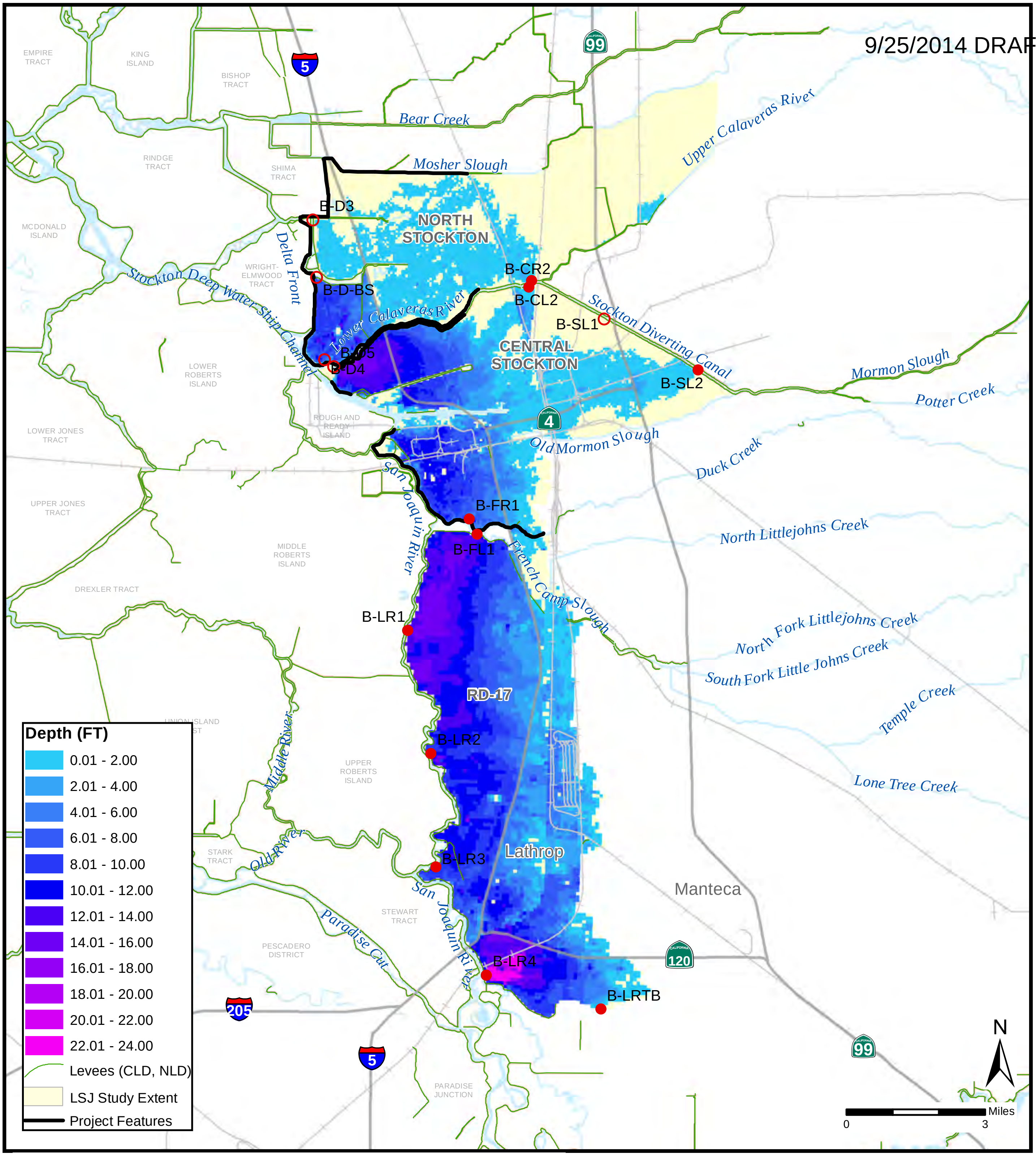
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 7A
2% (1/50) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

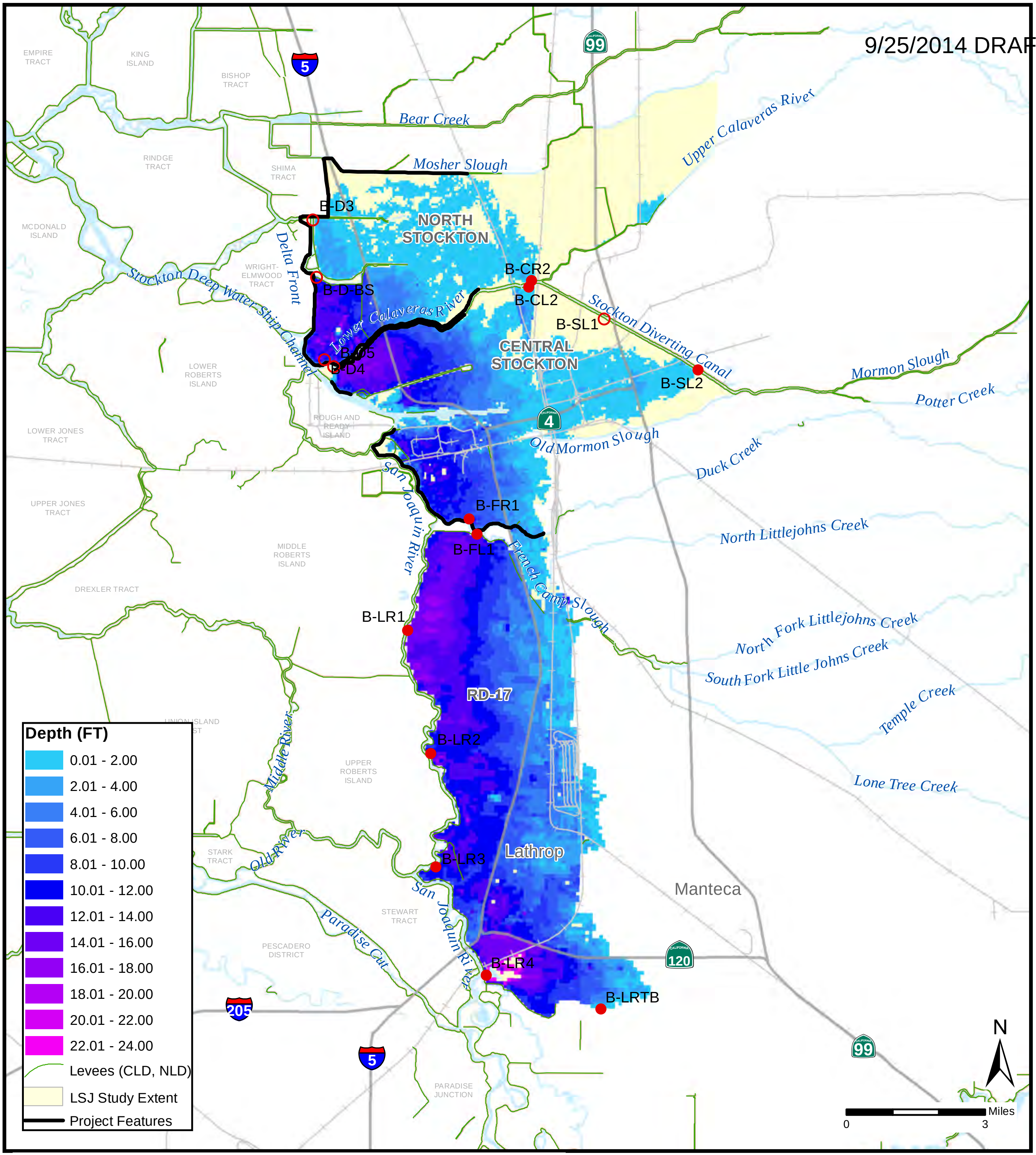
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 7A
1% (1/100) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

- ● Fails R&U Criteria
- ○ Meets R&U Criteria

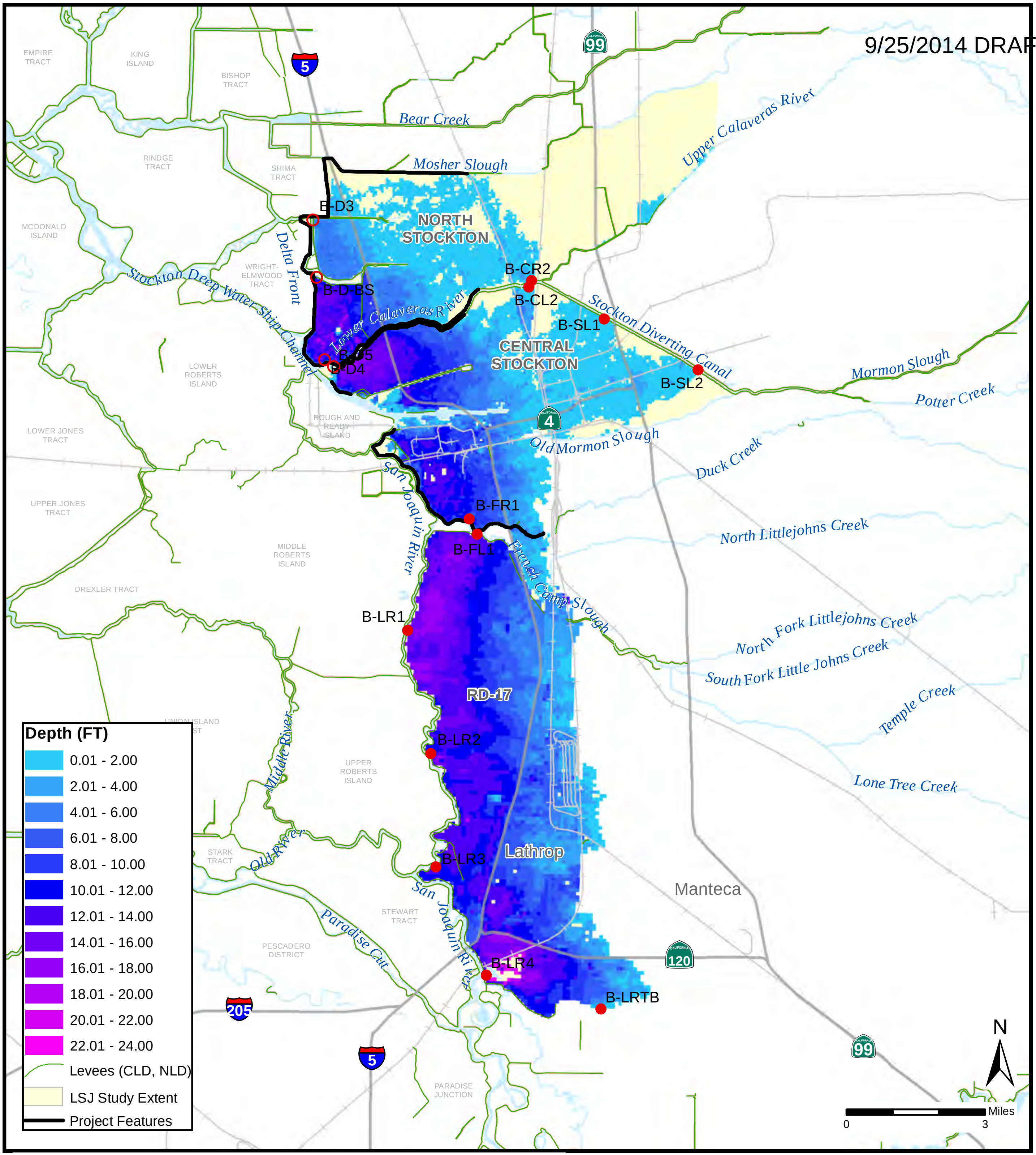
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 7A
0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 7A
0.2% (1/500) ACE

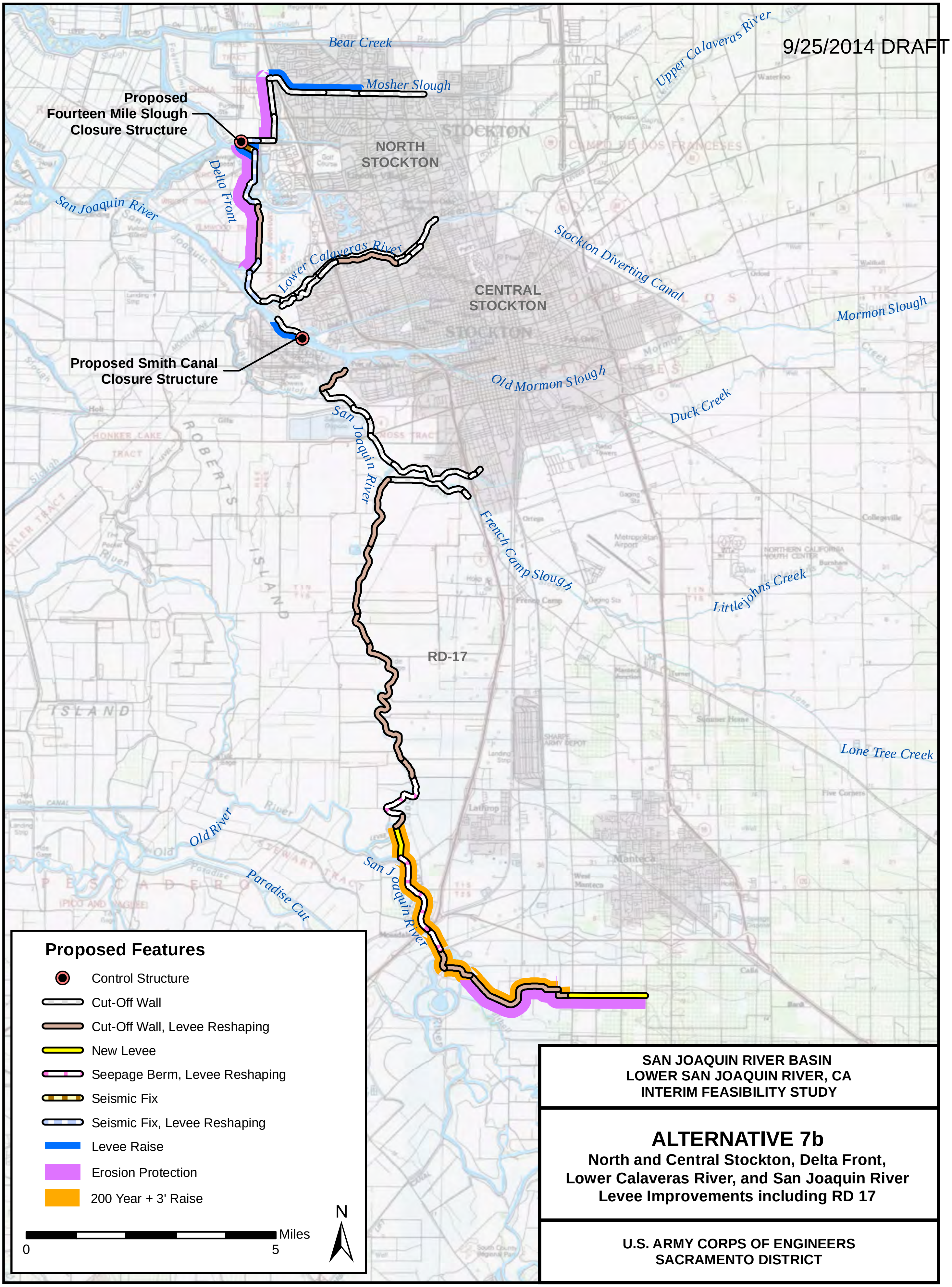
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 59

9/25/2014 DRAFT



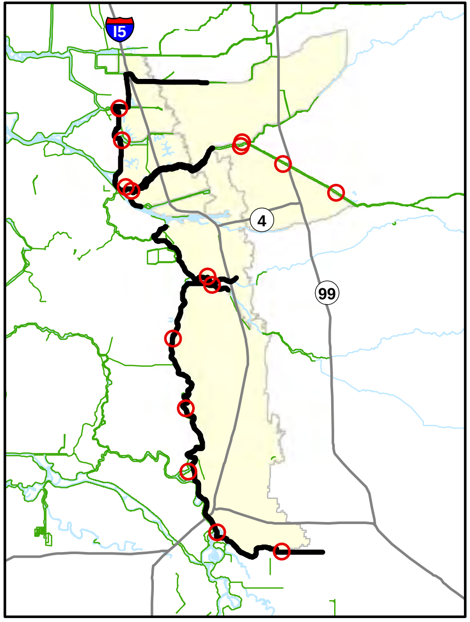
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

ALTERNATIVE 7b

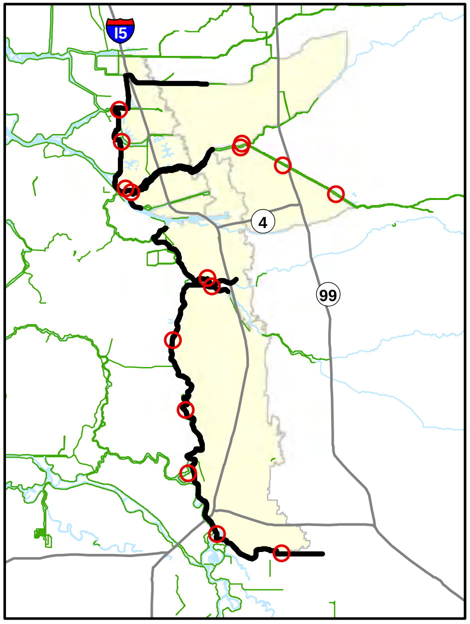
**North and Central Stockton, Delta Front,
Lower Calaveras River, and San Joaquin River
Levee Improvements including RD 17**

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

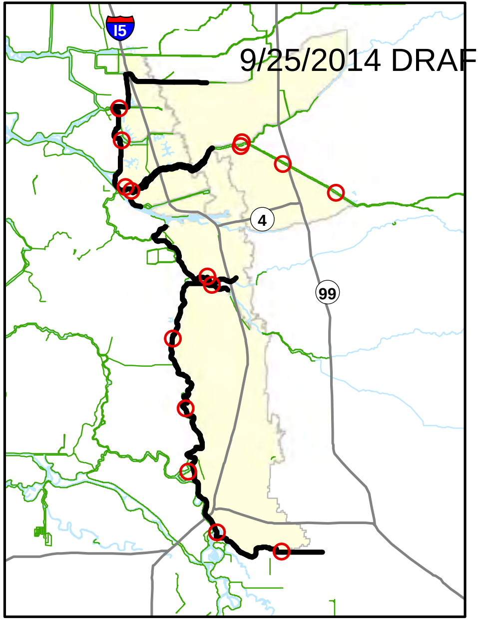
JUN 2014



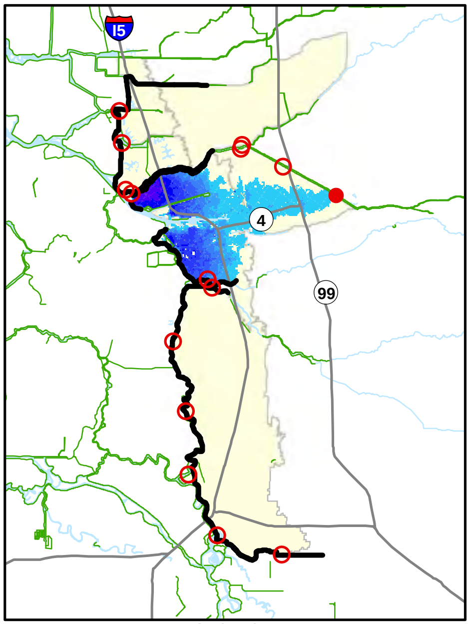
50% (1/2) ACE



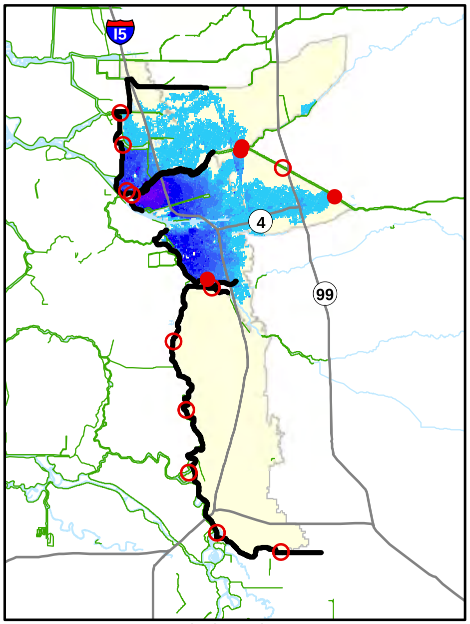
10% (1/10) ACE



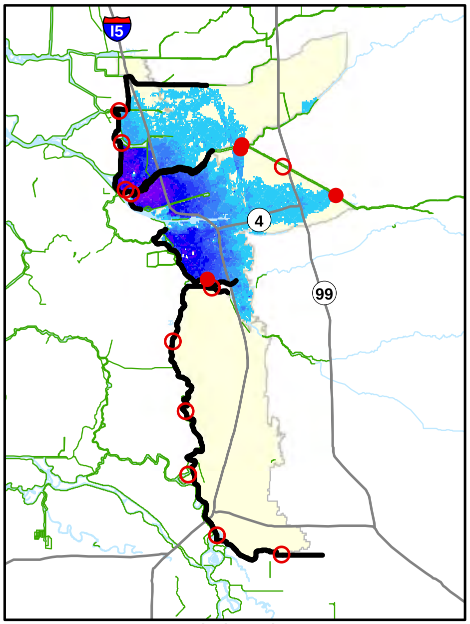
4% (1/25) ACE



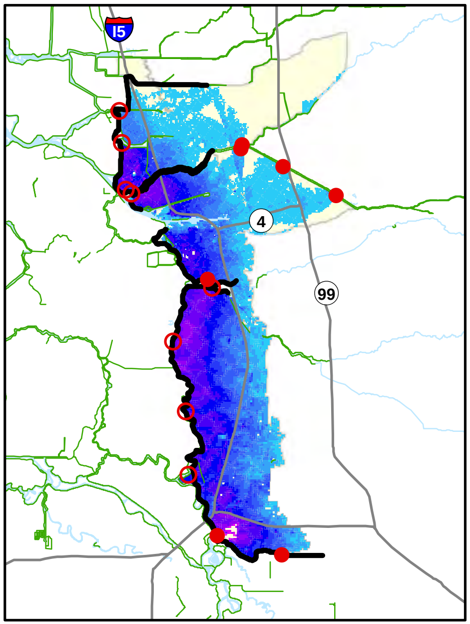
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Fails R&U Criteria
○ Meets R&U Criteria

Depth (FT)

▨ 0.01 - 2.00
▨ 2.01 - 4.00
▨ 4.01 - 6.00
▨ 6.01 - 8.00
▨ 8.01 - 10.00
▨ 10.01 - 12.00
▨ 12.01 - 14.00
▨ 14.01 - 16.00
▨ 16.01 - 18.00
▨ 18.01 - 20.00
▨ 20.01 - 22.00
▨ 22.01 - 24.00
Levees (CLD, NLD)
LSJ Study Extent
Project Features

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

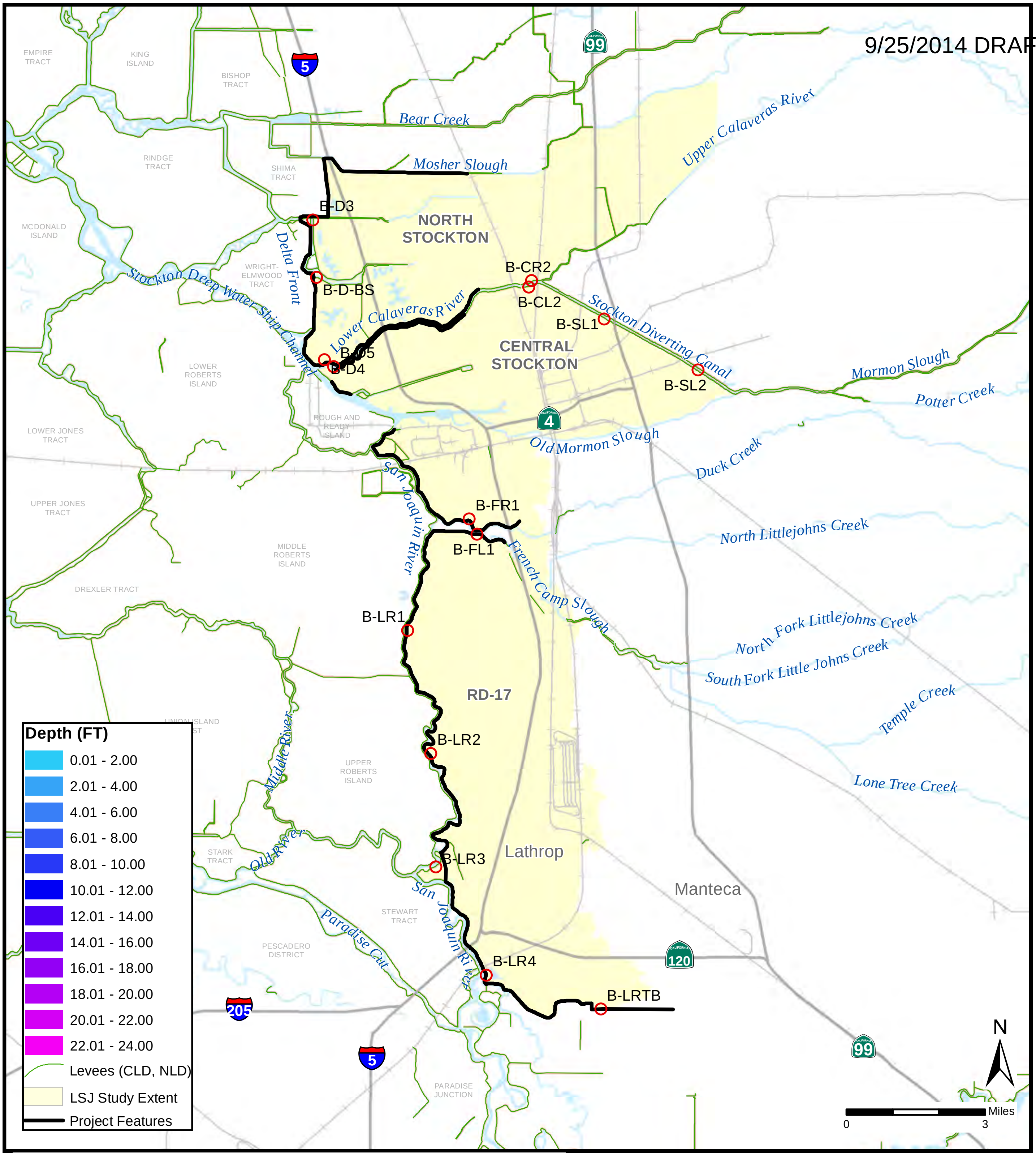
R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7B

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 61



9/25/2014 DRAFT

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7B 50% (1/2) ACE

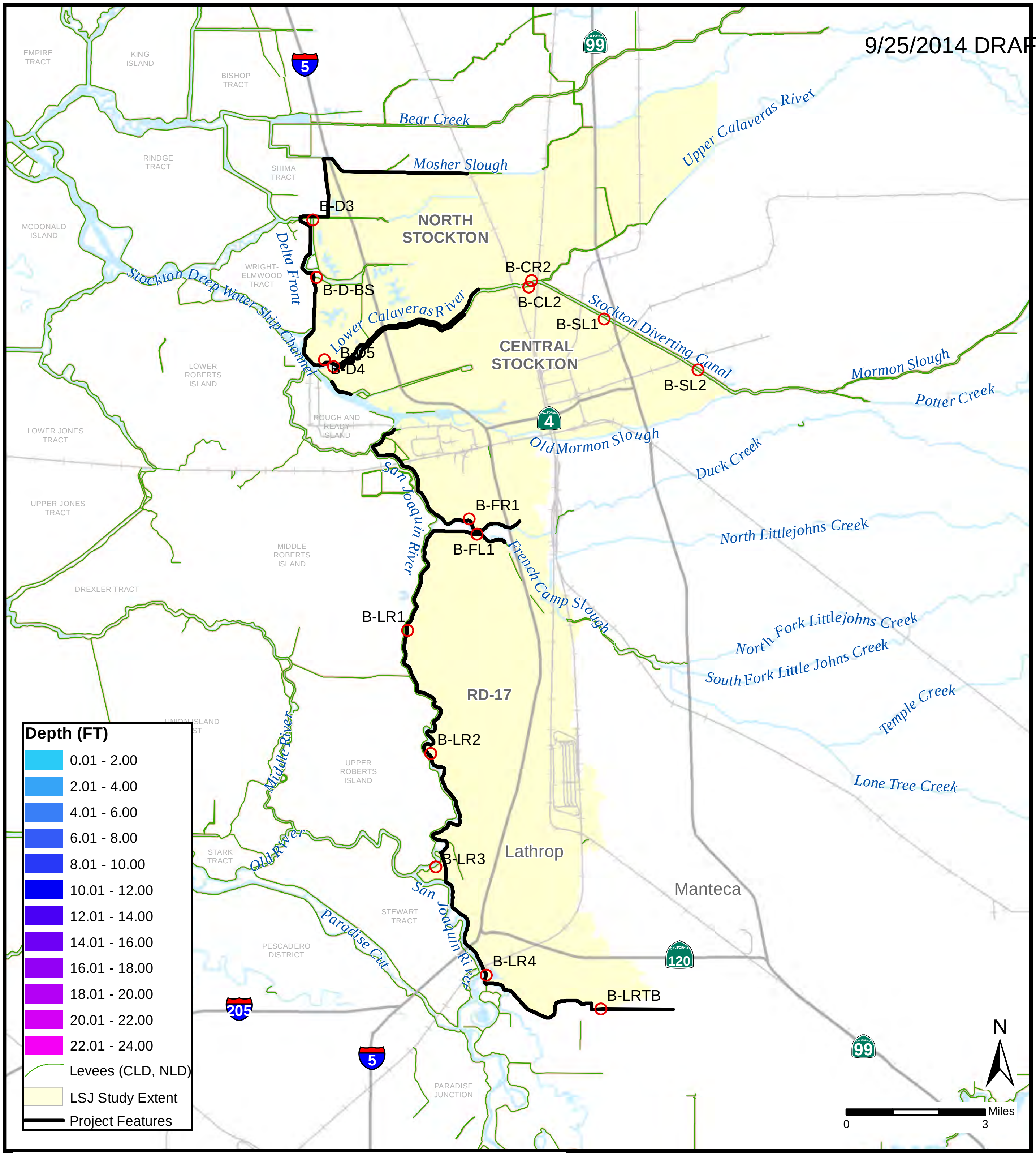
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 62

9/25/2014 DRAFT



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 7B
10% (1/10) ACE

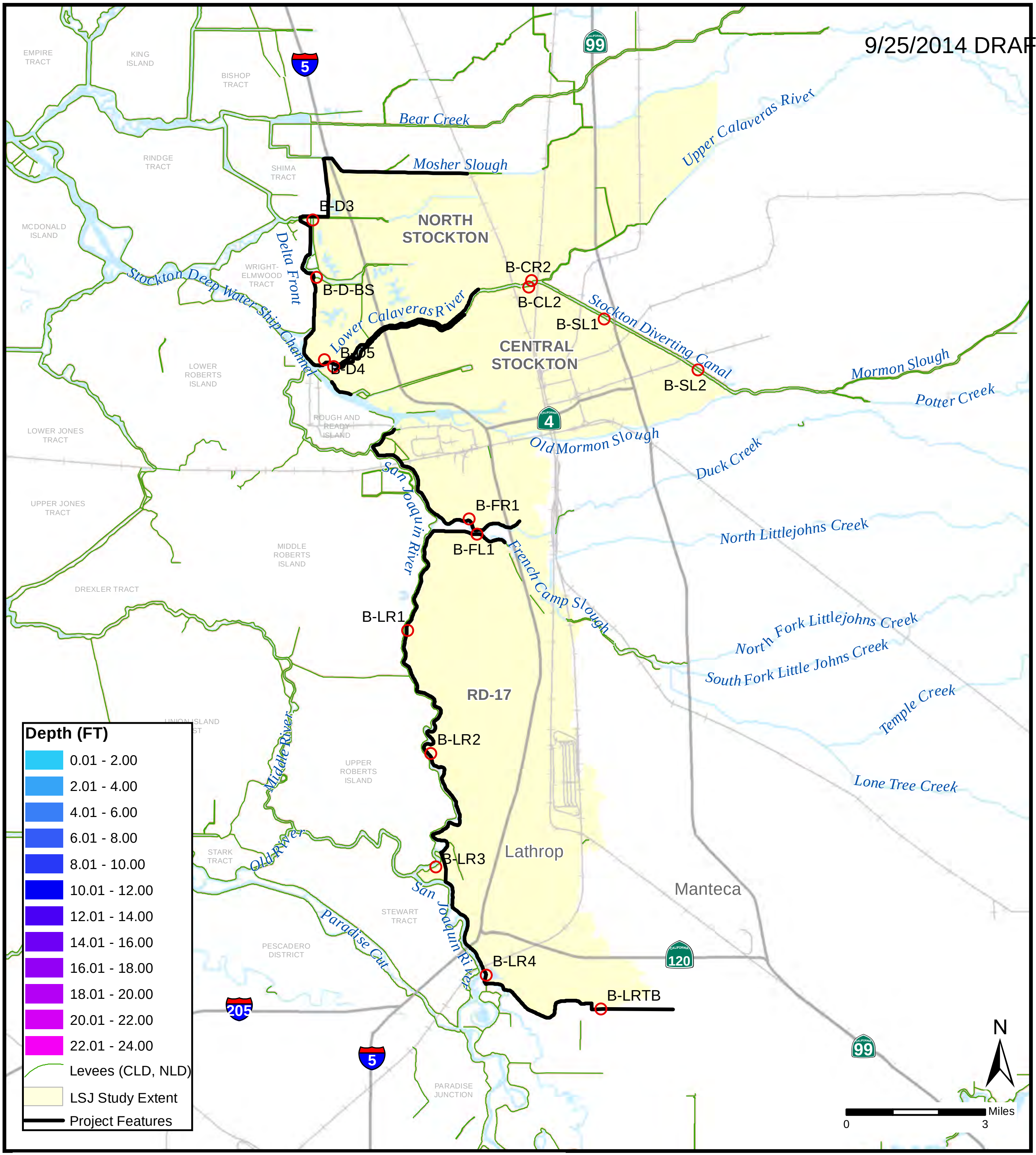
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Imagery Source: 2012 NAIP, 1m

Prepared by V. Nino-Tapia

JUN 2014

PLATE 63



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

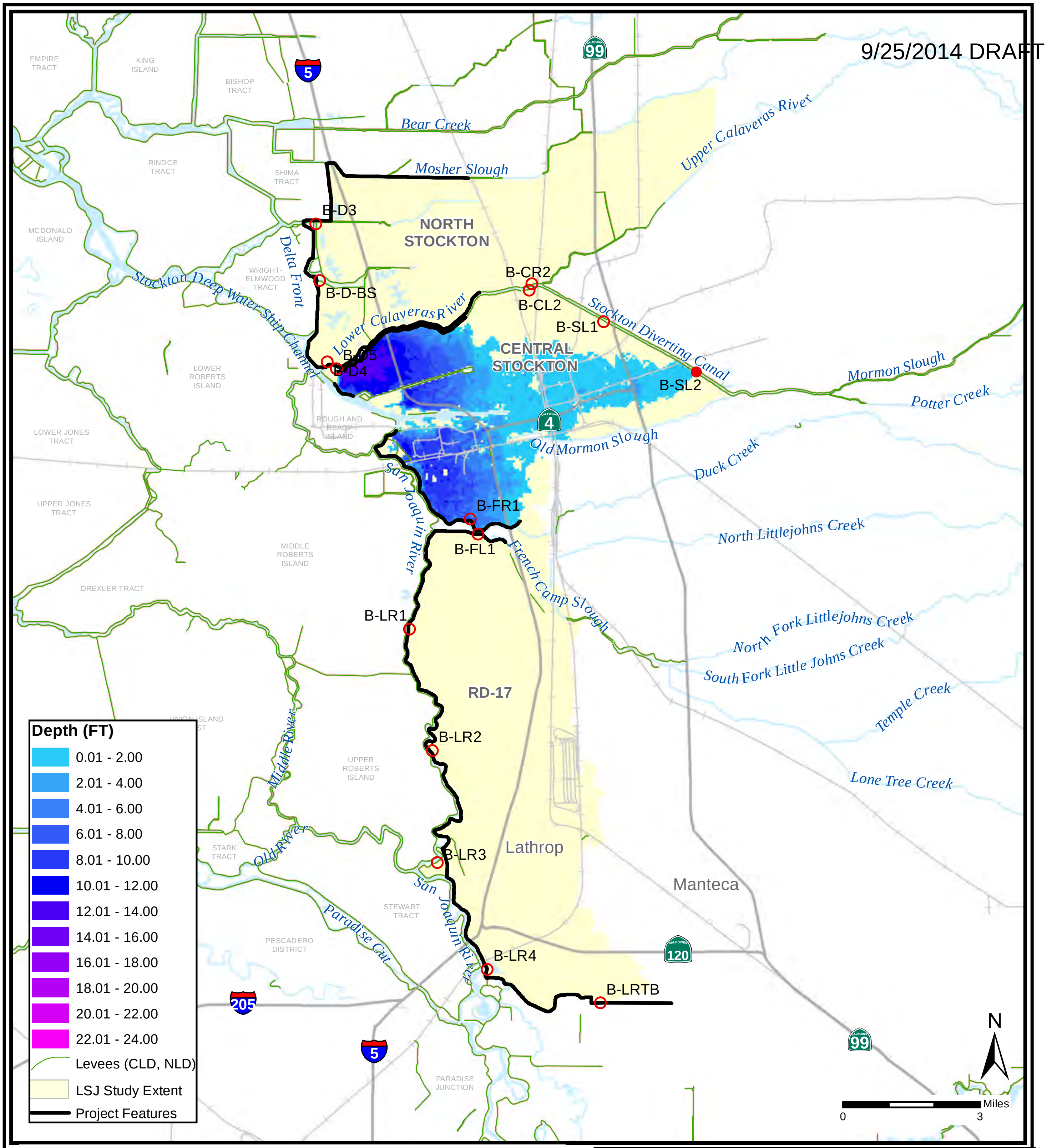
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 7B
4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

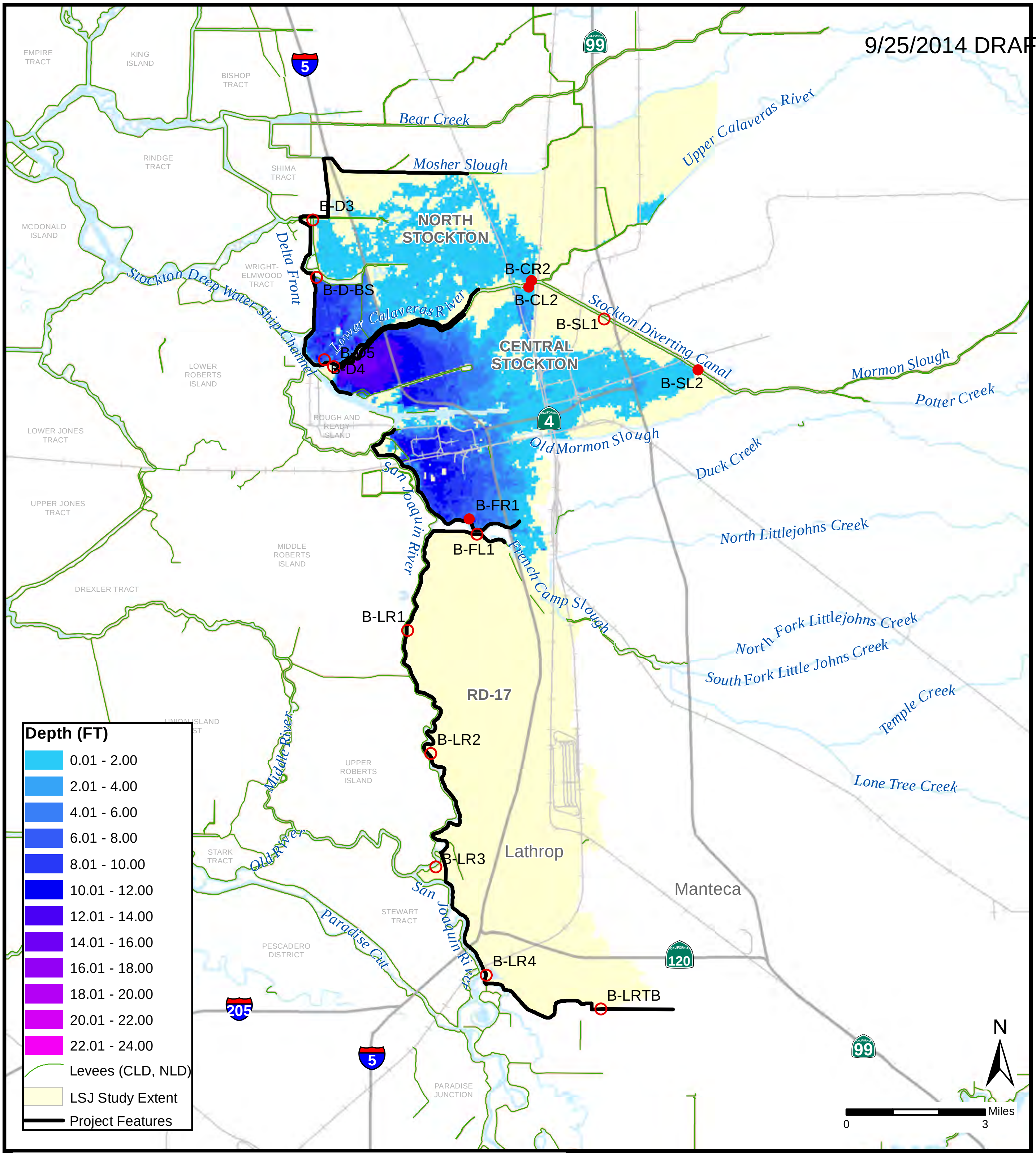
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 7B
2% (1/50) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

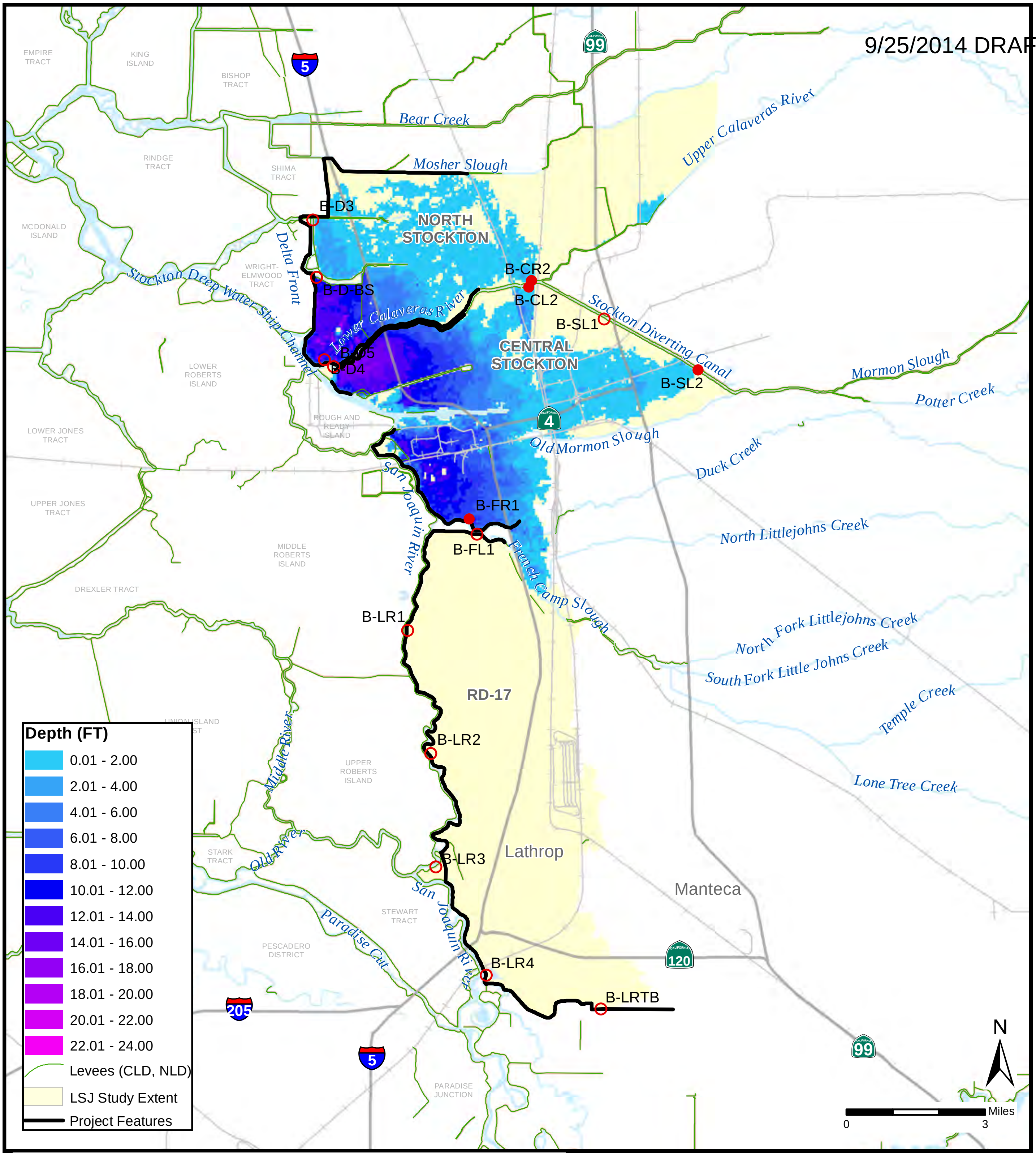
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 7B
1% (1/100)ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

**SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY**

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 7B 0.5% (1/200) ACE

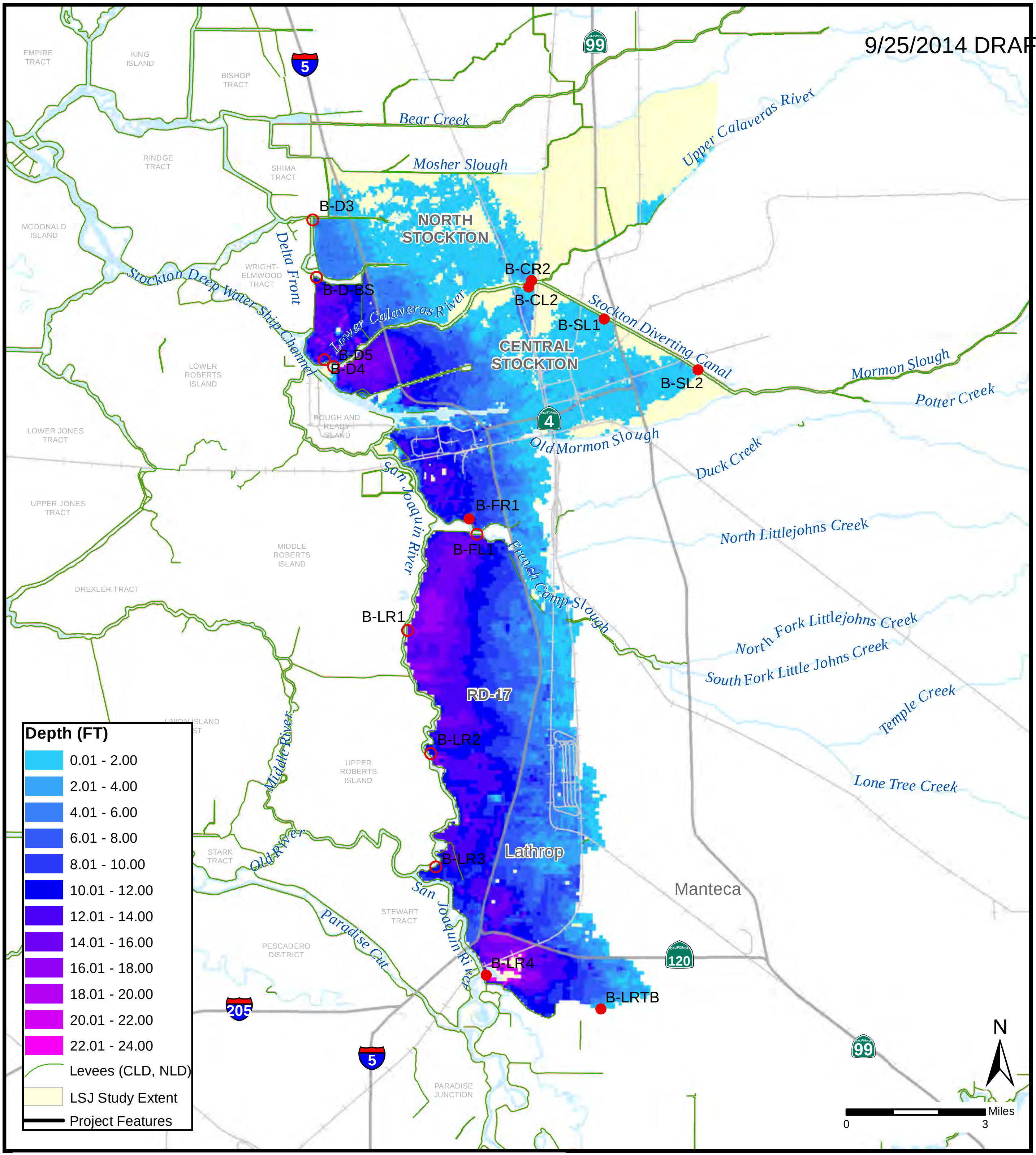
**U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT**

Prepared by V. Nino-Tapia

JUN 2014

PLATE 67

9/25/2014 DRAFT



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

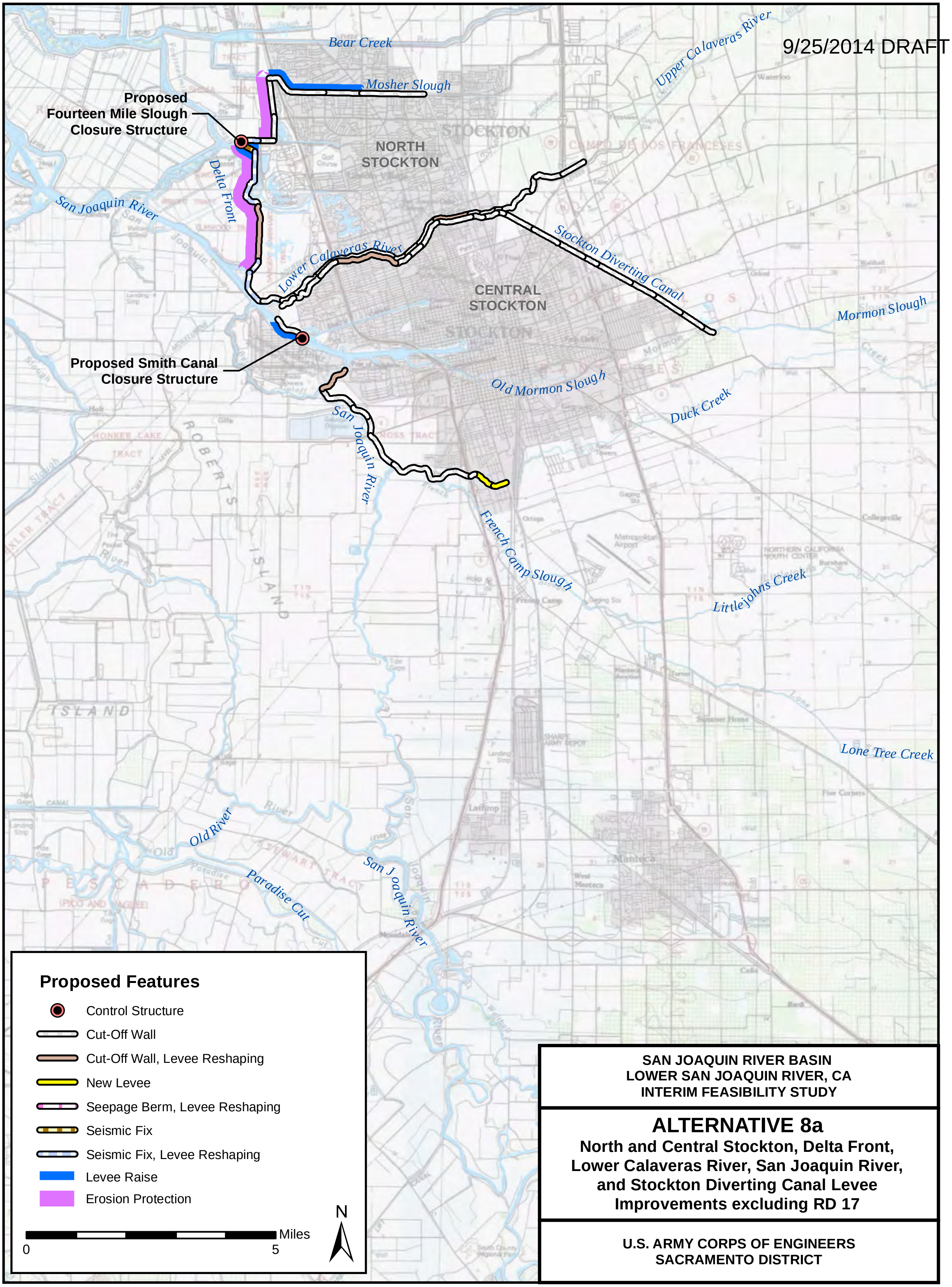
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 7B
0.2% (1/500) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

9/25/2014 DRAFT



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

ALTERNATIVE 8a

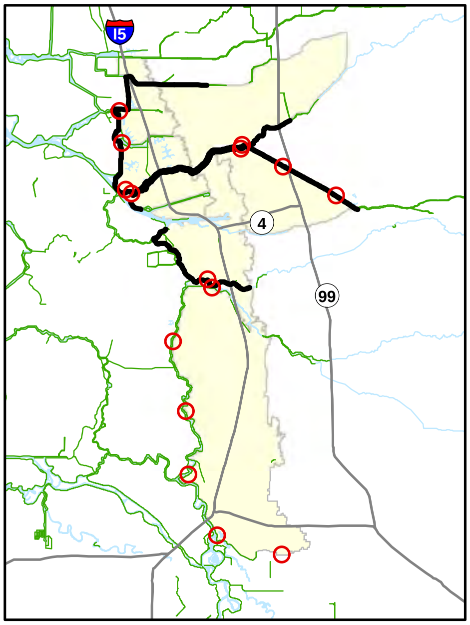
North and Central Stockton, Delta Front, Lower Calaveras River, San Joaquin River, and Stockton Diverting Canal Levee Improvements excluding RD 17

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

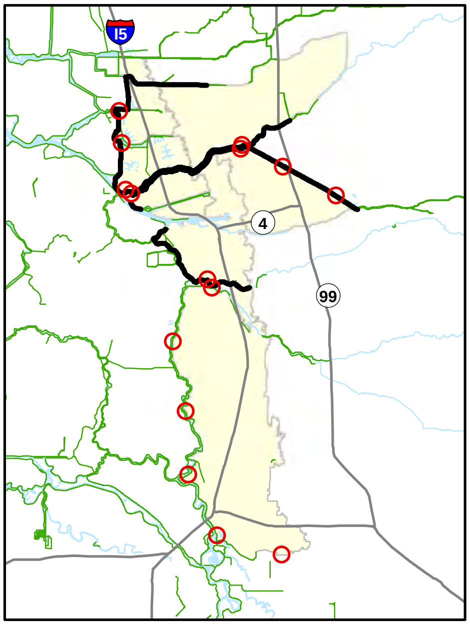
JUN 2014

PLATE 69

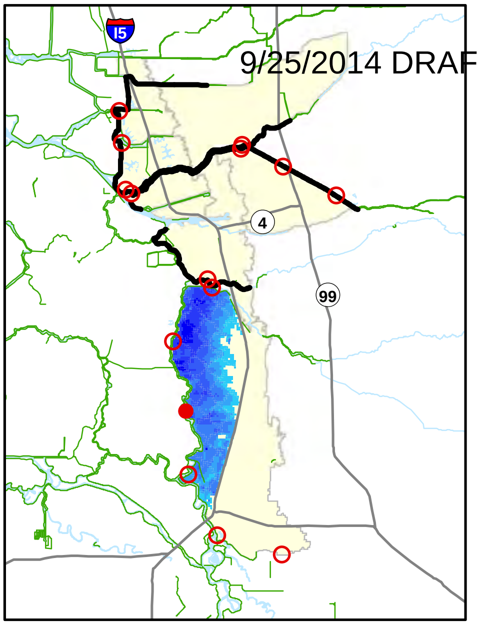
9/25/2014 DRAFT



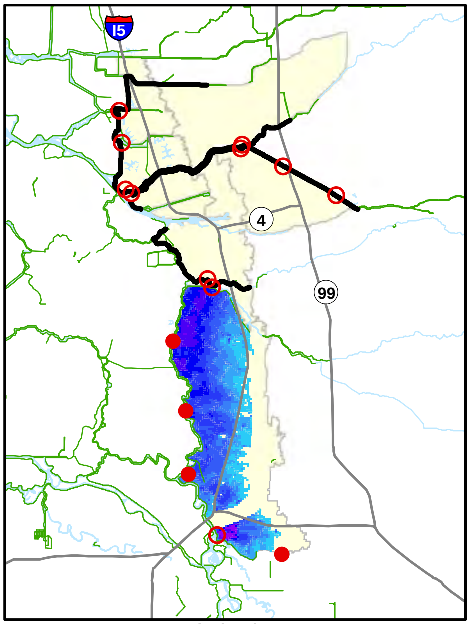
50% (1/2) ACE



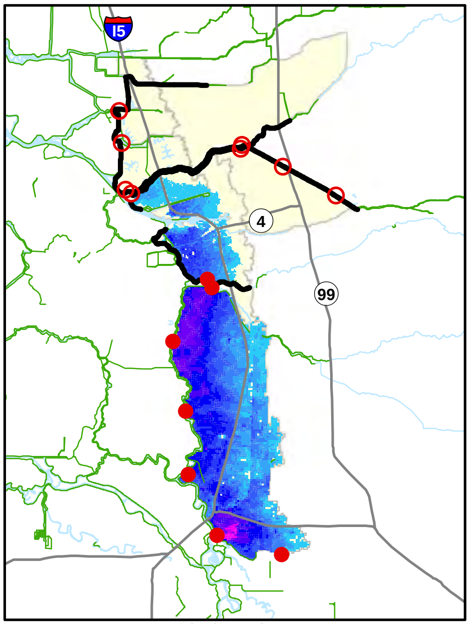
10% (1/10) ACE



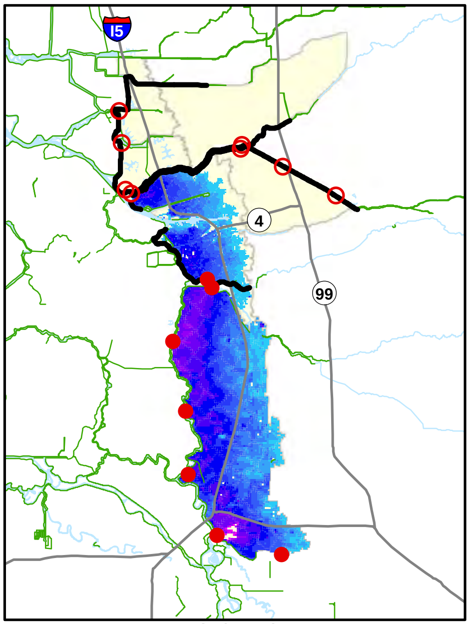
4% (1/25) ACE



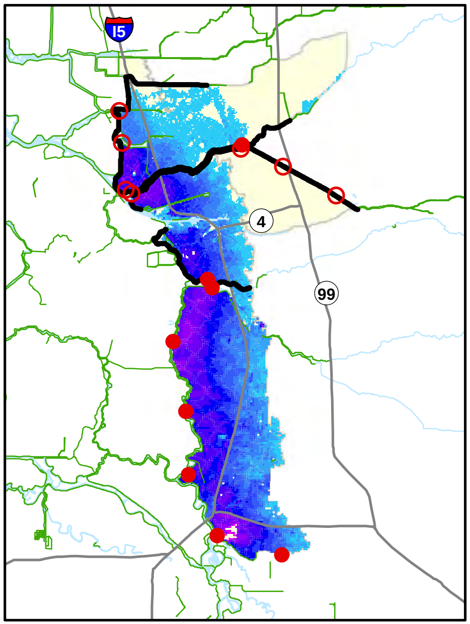
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Fails R&U Criteria
○ Meets R&U Criteria

Depth (FT)

- 0.01 - 2.00
- 2.01 - 4.00
- 4.01 - 6.00
- 6.01 - 8.00
- 8.01 - 10.00
- 10.01 - 12.00
- 12.01 - 14.00
- 14.01 - 16.00
- 16.01 - 18.00
- 18.01 - 20.00
- 20.01 - 22.00
- 22.01 - 24.00
- Levees (CLD, NLD)
- LSJ Study Extent
- Project Features

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

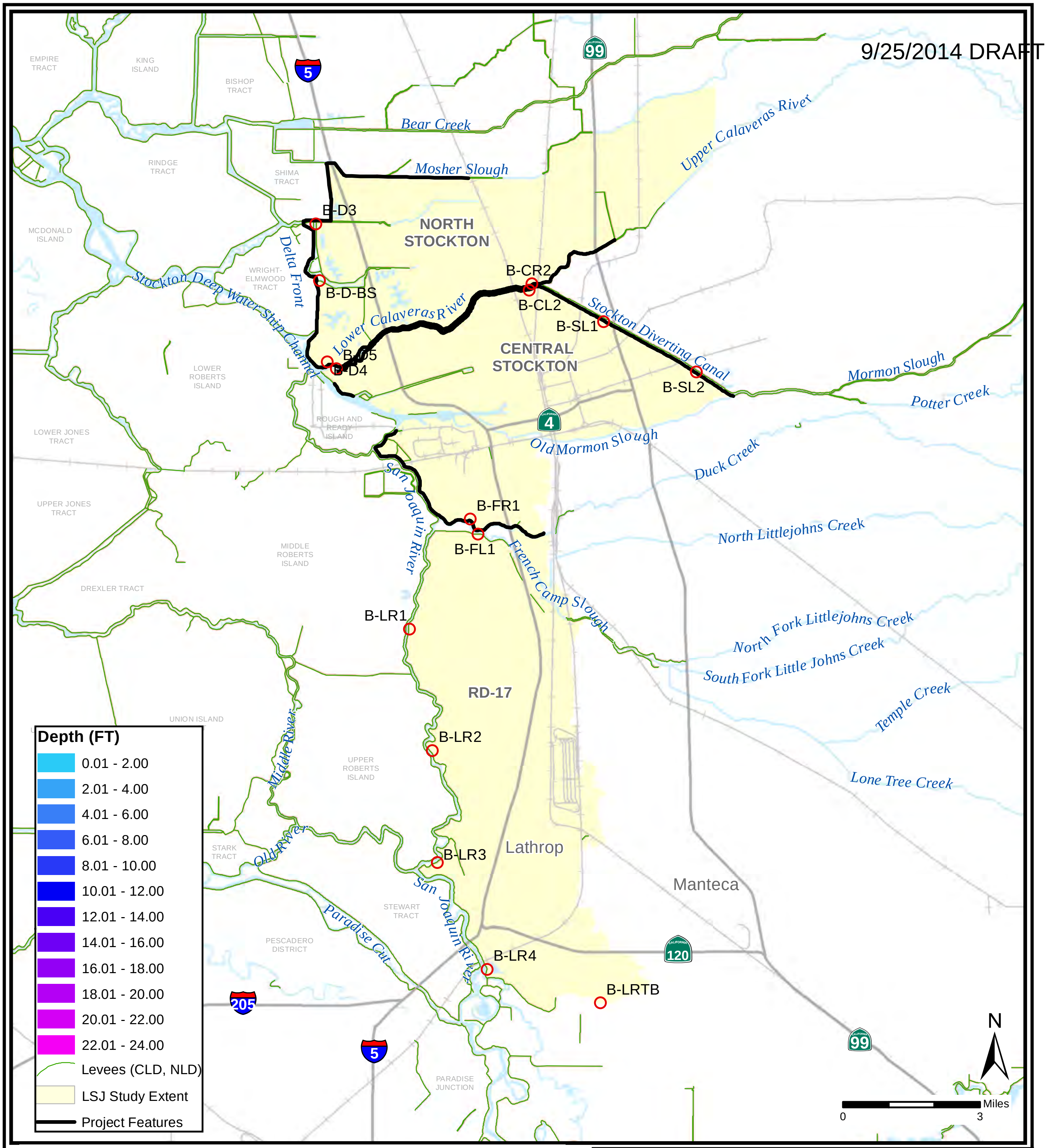
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8A

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 8A
50% (1/2) ACE

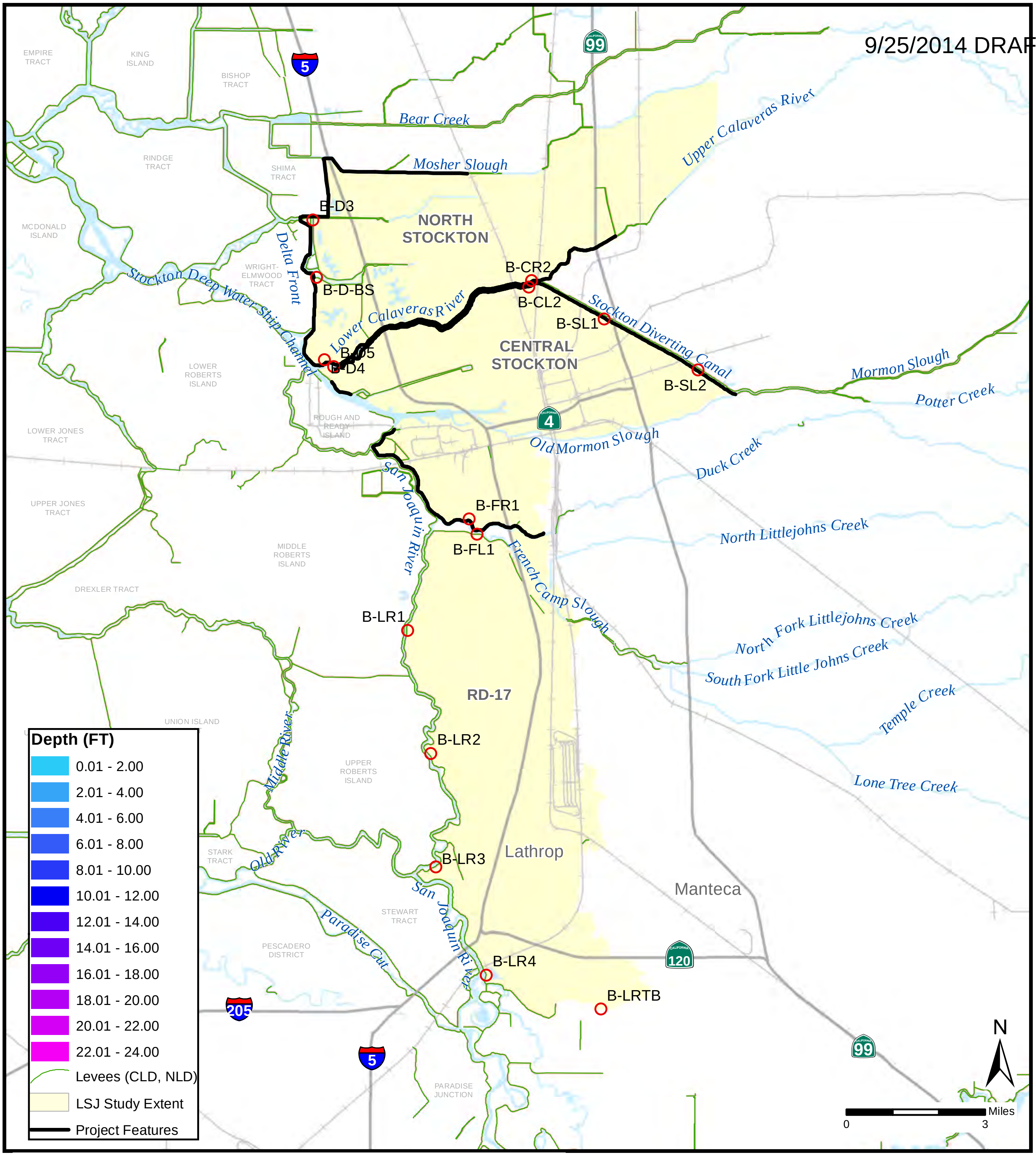
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 71

9/25/2014 DRAFT



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

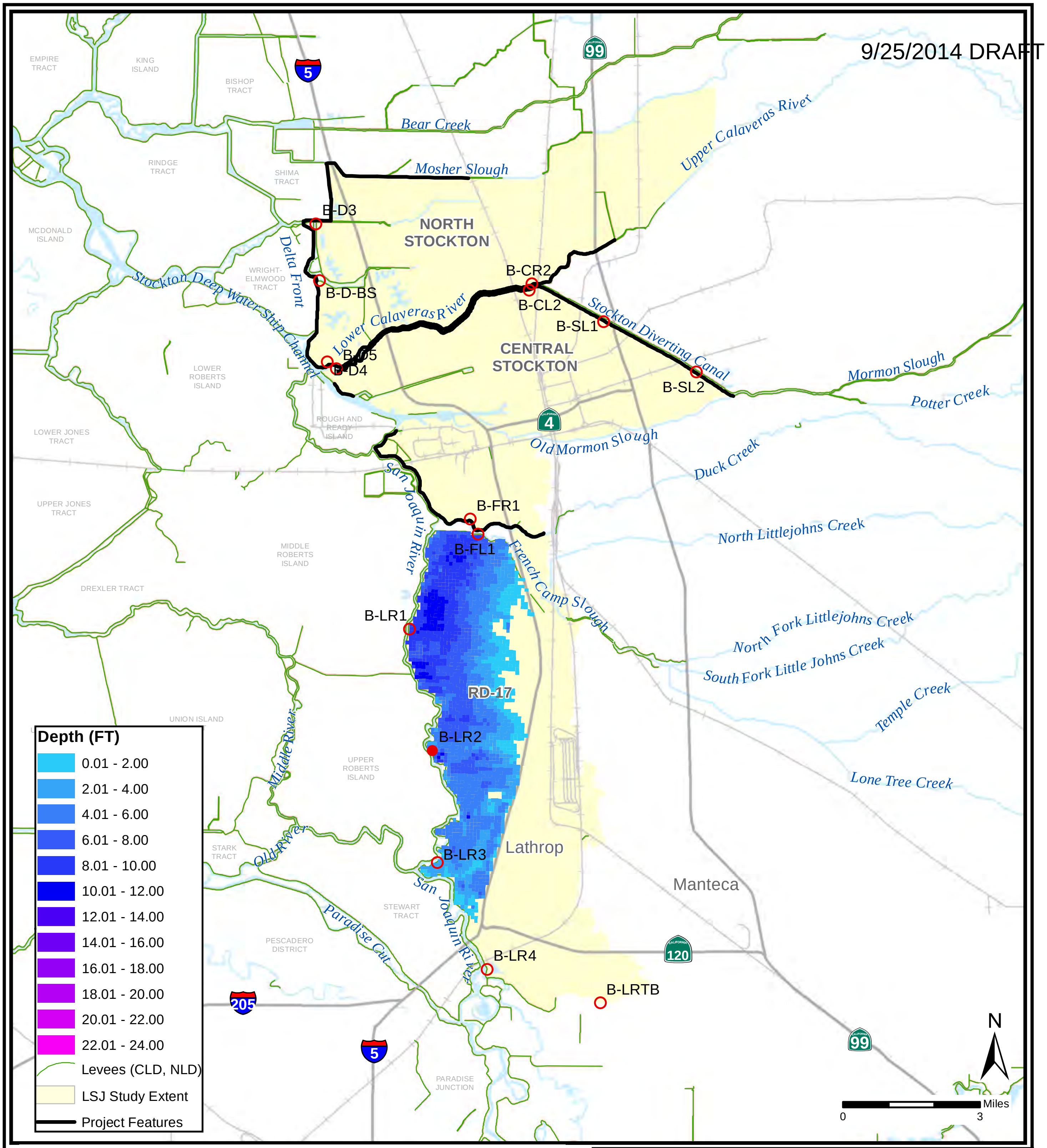
R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8B 10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 72



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

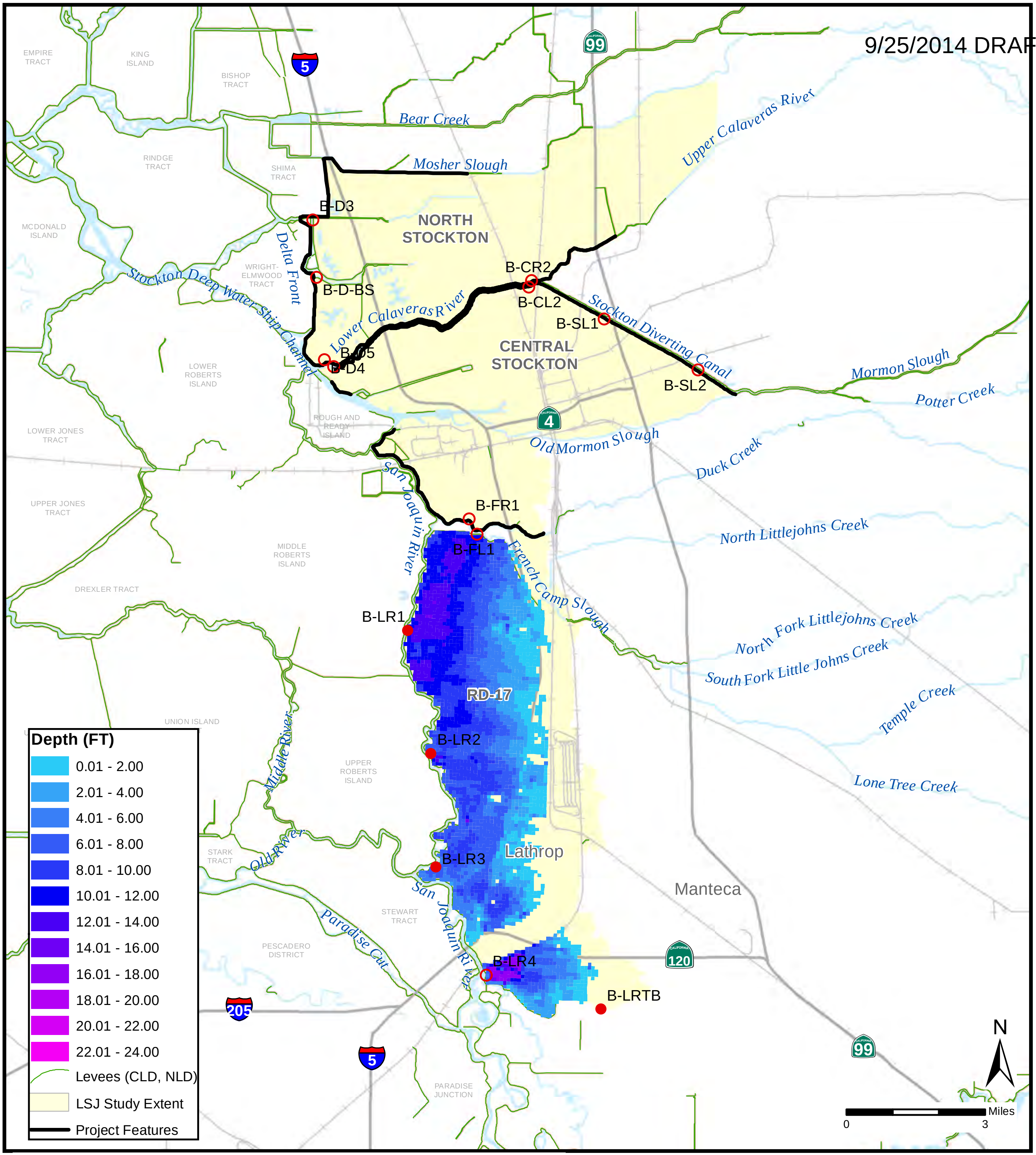
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 8A
4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

9/25/2014 DRAFT



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 8B
2% (1/50) ACE

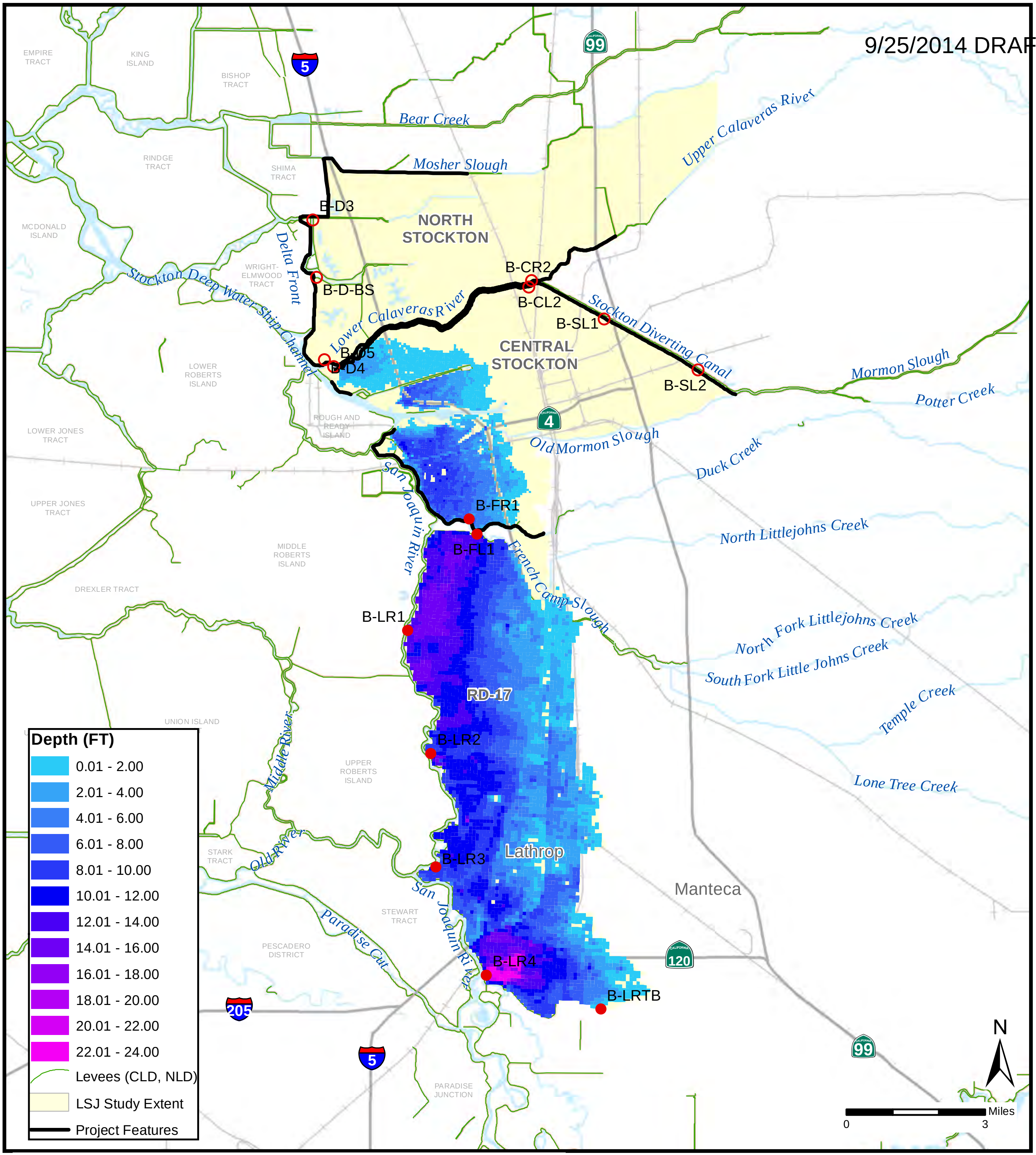
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 74

9/25/2014 DRAFT



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

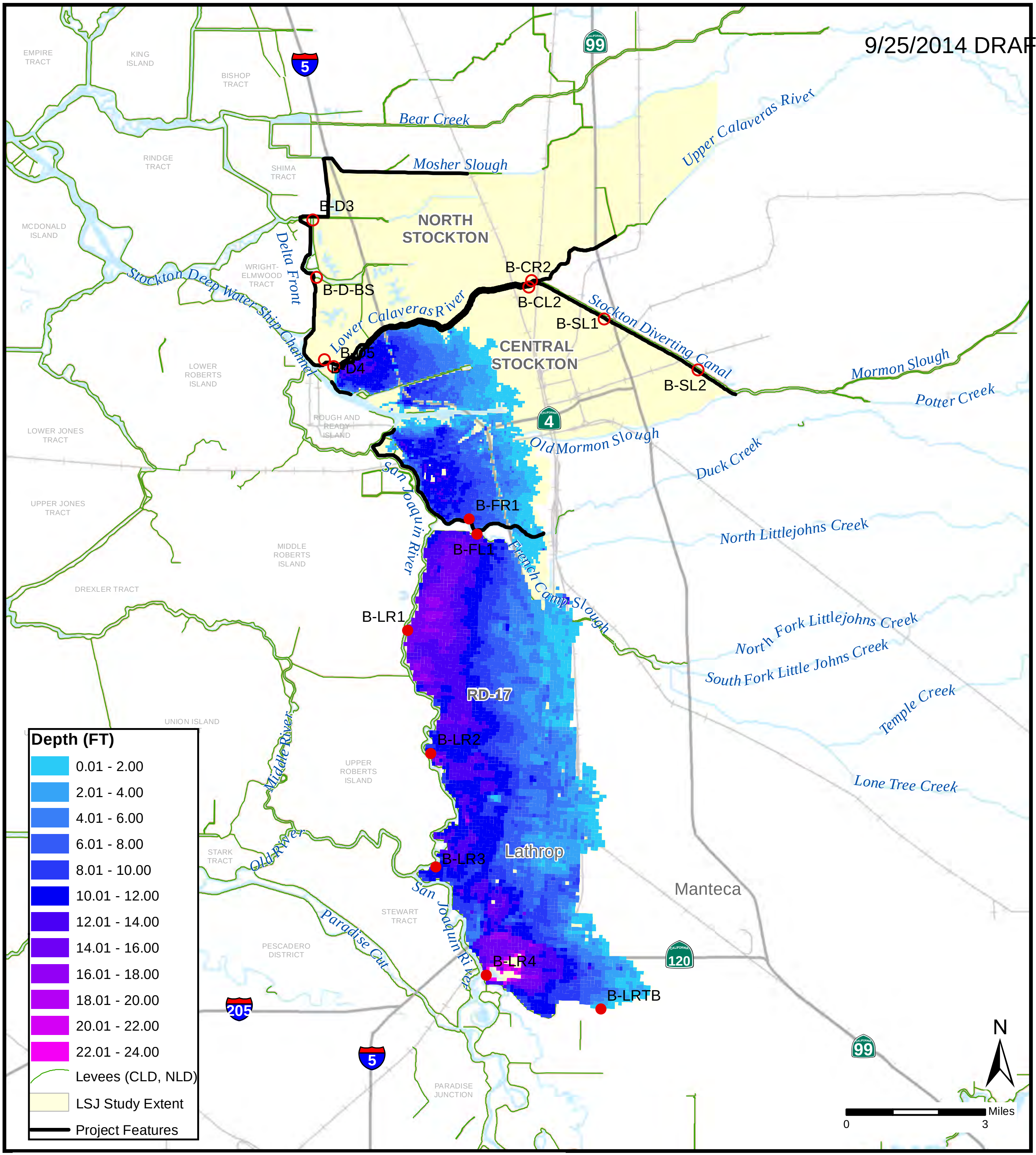
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 8A
1% (1/100) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

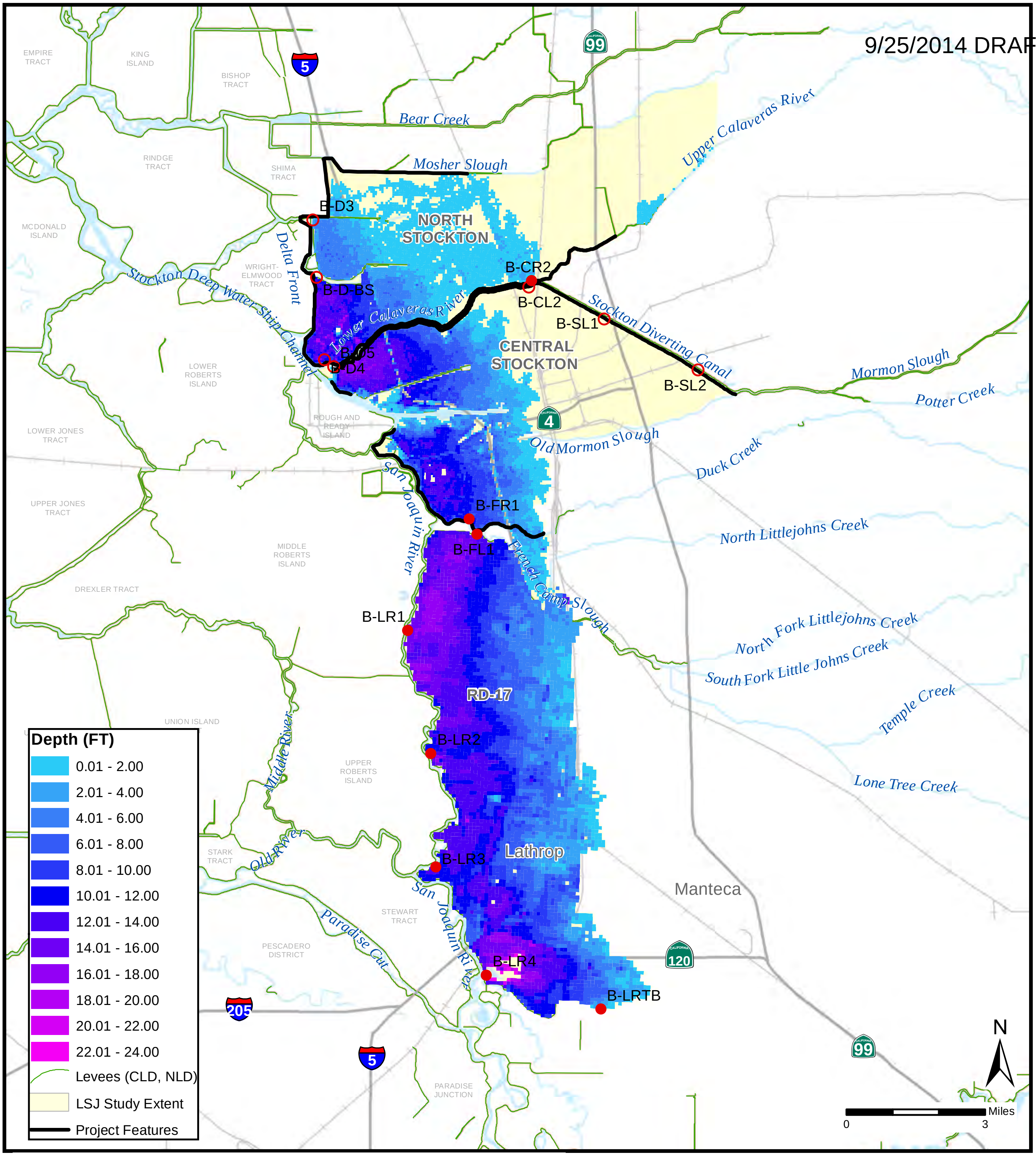
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 8A
0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

9/25/2014 DRAFT



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 8A
0.2% (1/500) ACE

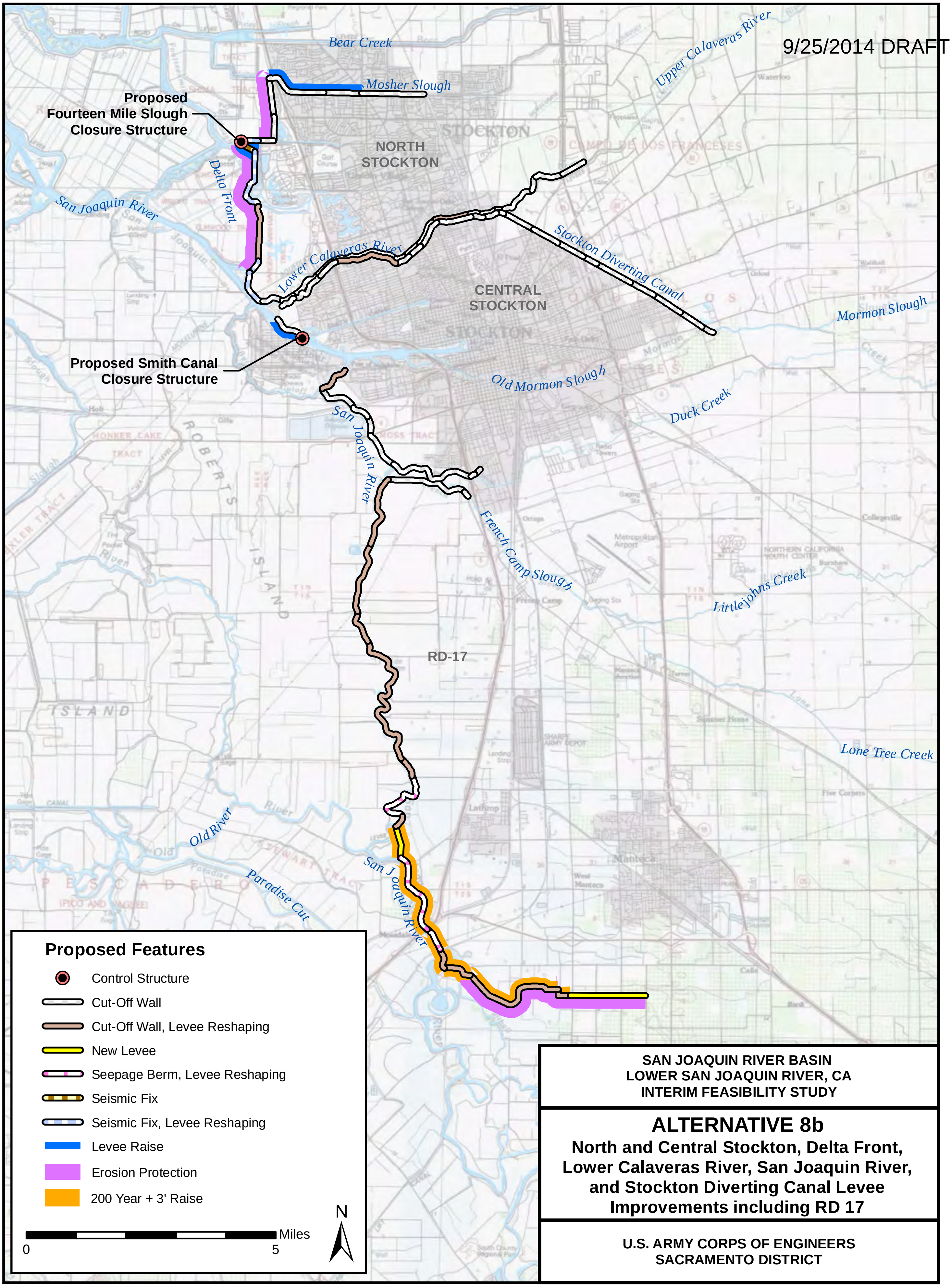
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 77

9/25/2014 DRAFT



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

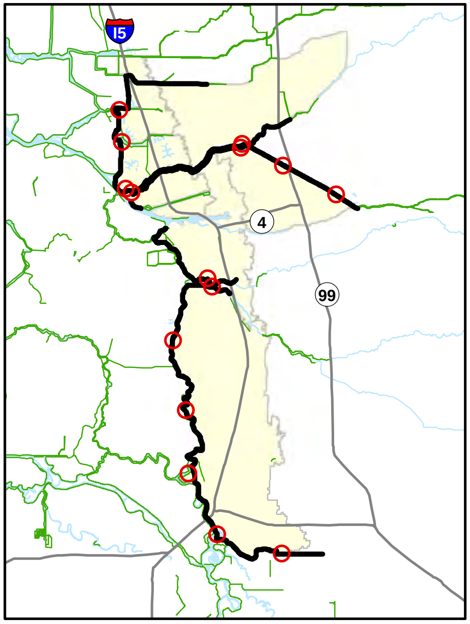
ALTERNATIVE 8b

North and Central Stockton, Delta Front, Lower Calaveras River, San Joaquin River, and Stockton Diverting Canal Levee Improvements including RD 17

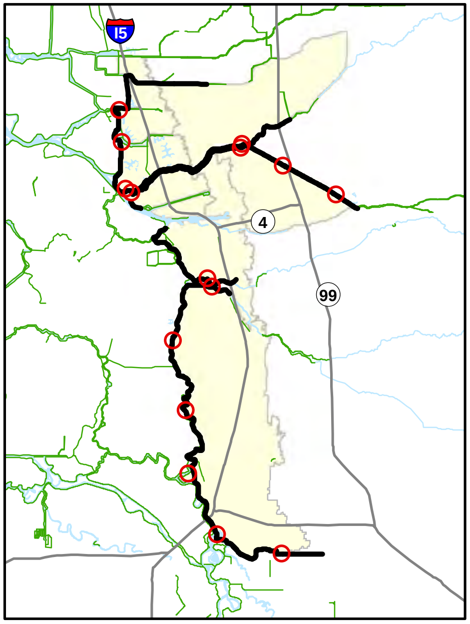
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014

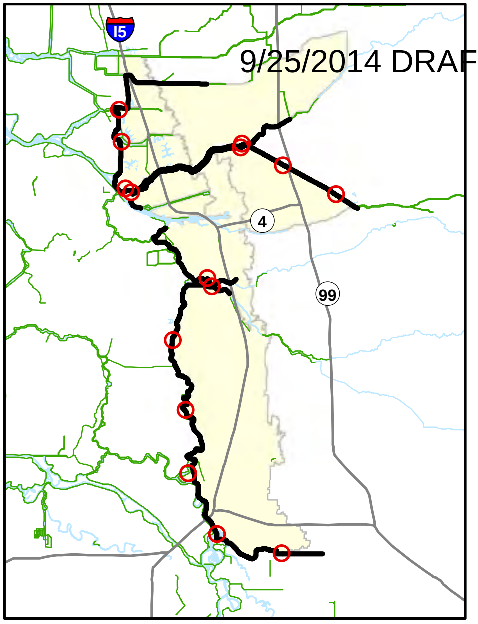
PLATE 78



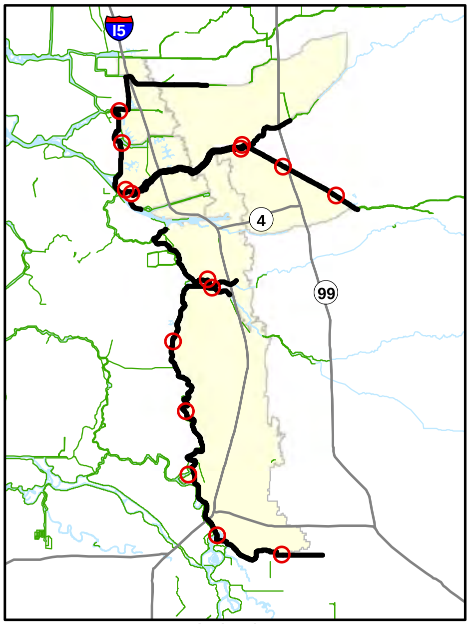
50% (1/2) ACE



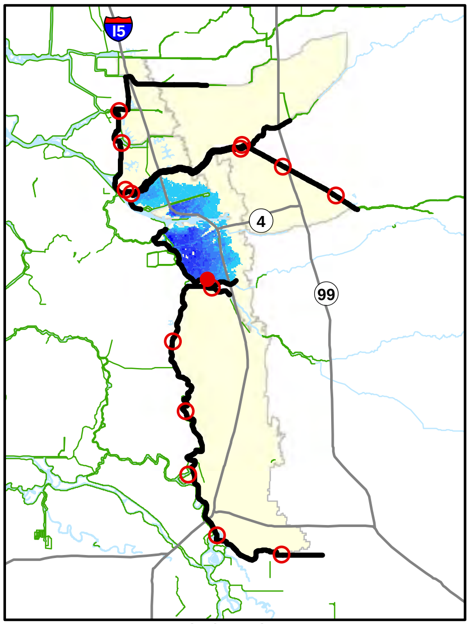
10% (1/10) ACE



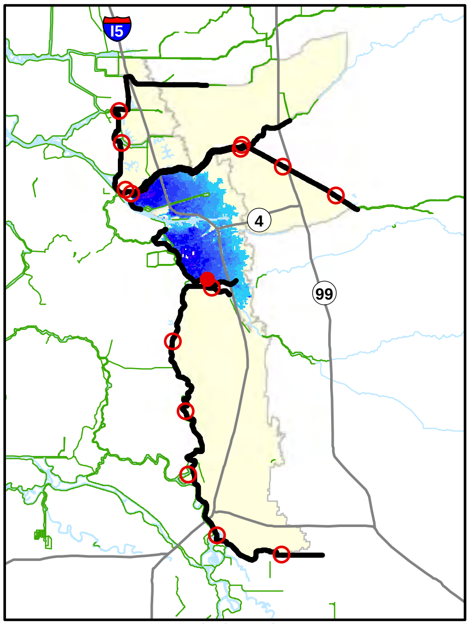
4% (1/25) ACE



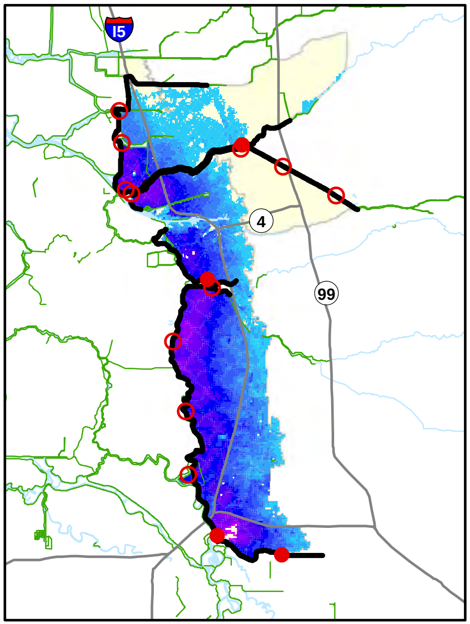
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Fails R&U Criteria
○ Meets R&U Criteria

Depth (FT)

🟦 0.01 - 2.00
🟦 2.01 - 4.00
🟦 4.01 - 6.00
🟦 6.01 - 8.00
🟦 8.01 - 10.00
🟦 10.01 - 12.00
🟪 12.01 - 14.00
🟪 14.01 - 16.00
🟪 16.01 - 18.00
🟪 18.01 - 20.00
🟪 20.01 - 22.00
🟪 22.01 - 24.00
Levees (CLD, NLD)
▭ LSJ Study Extent
Project Features

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

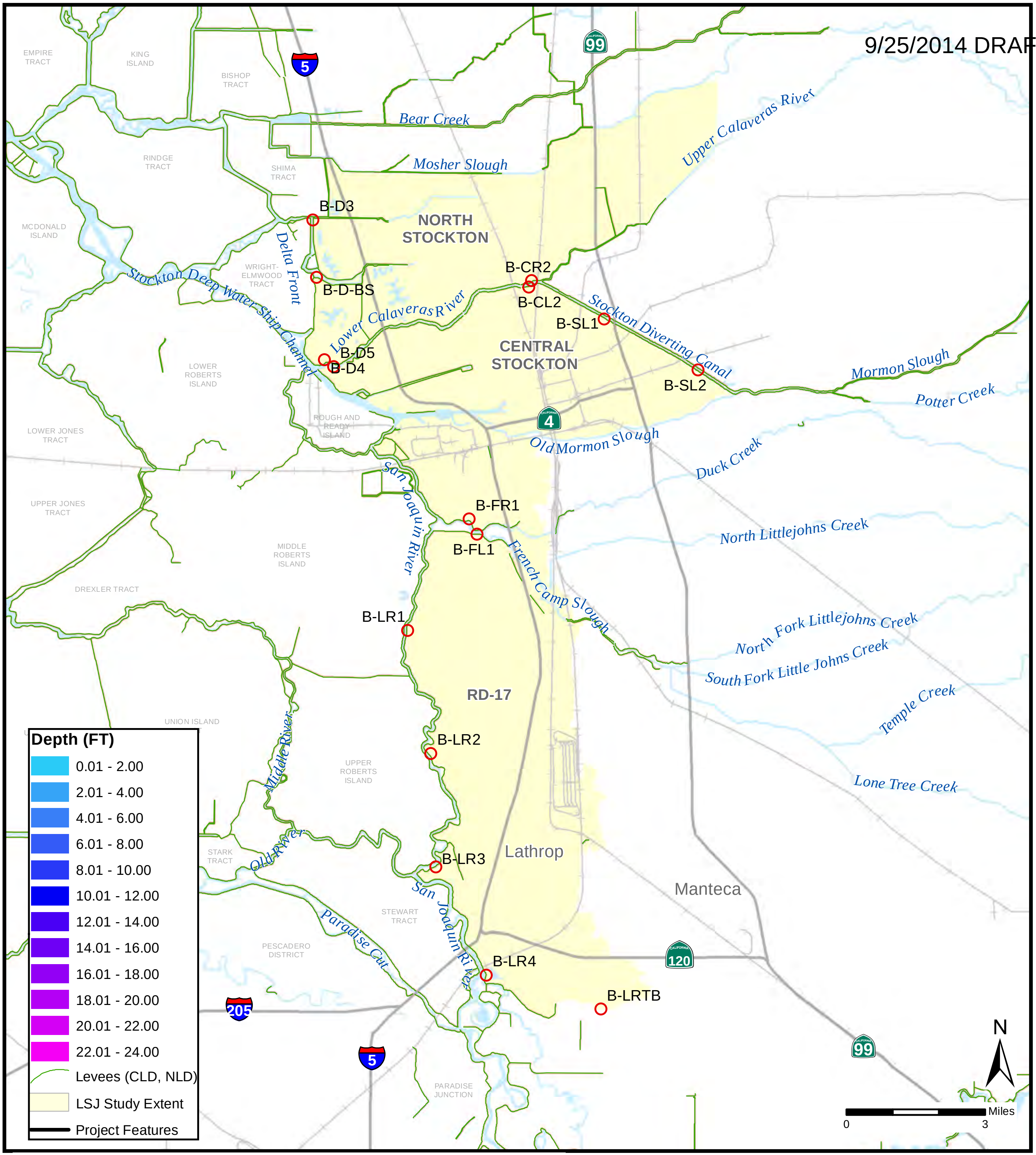
R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8B

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014

PLATE 79



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

Imagery Source: 2012 NAIP, 1m

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 8B
50% (1/2) ACE

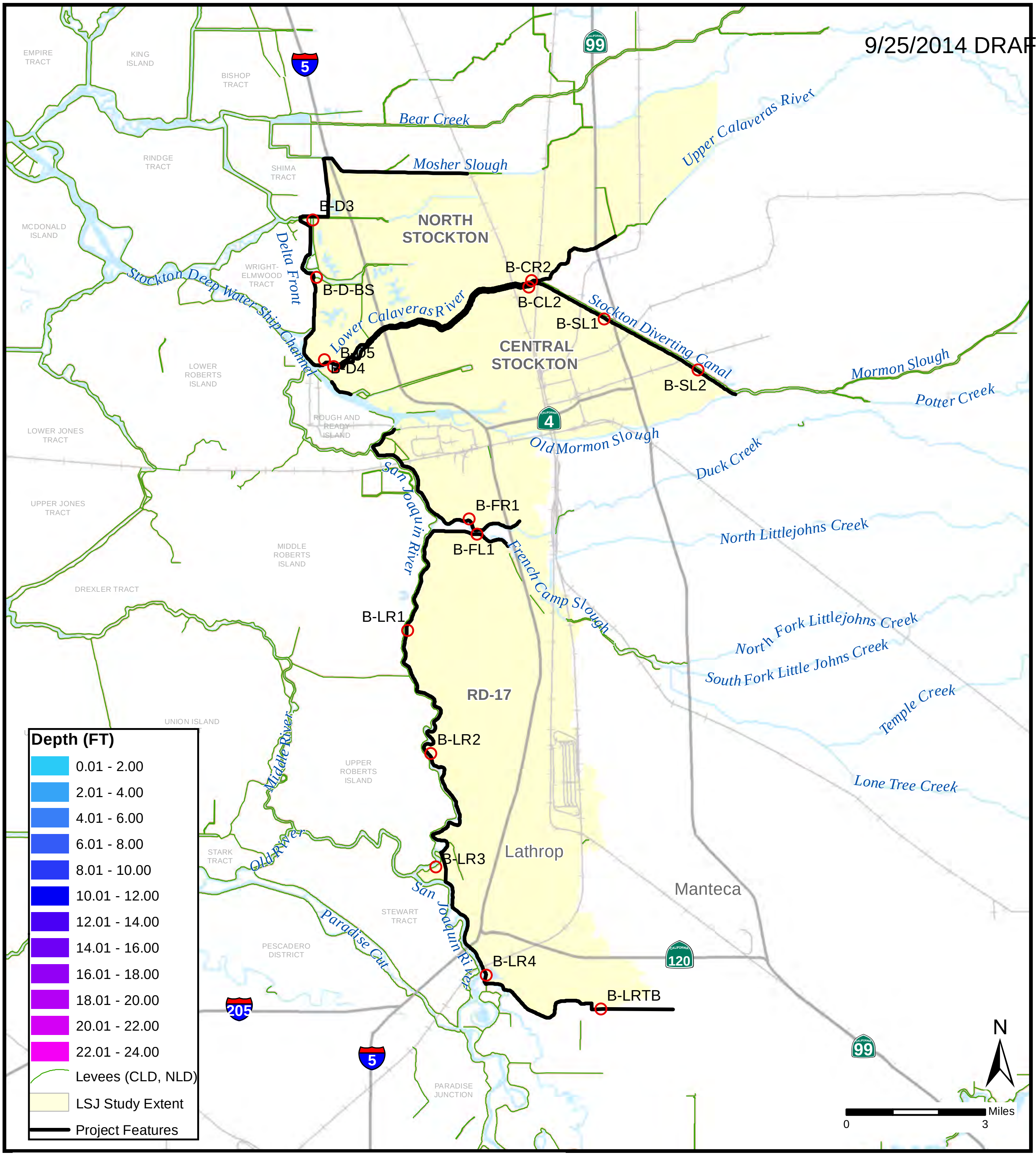
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 80

9/25/2014 DRAFT



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

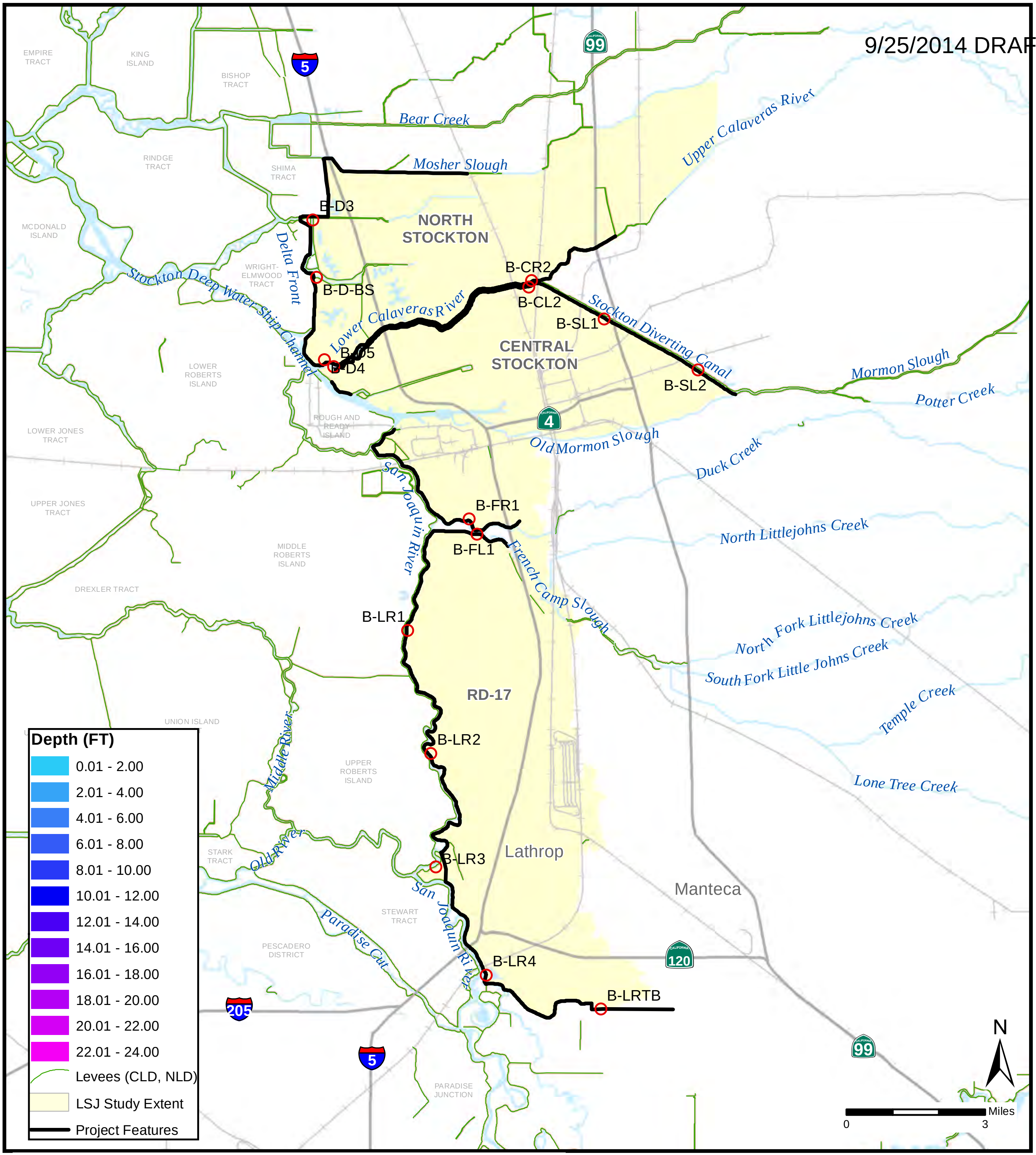
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 8B 10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

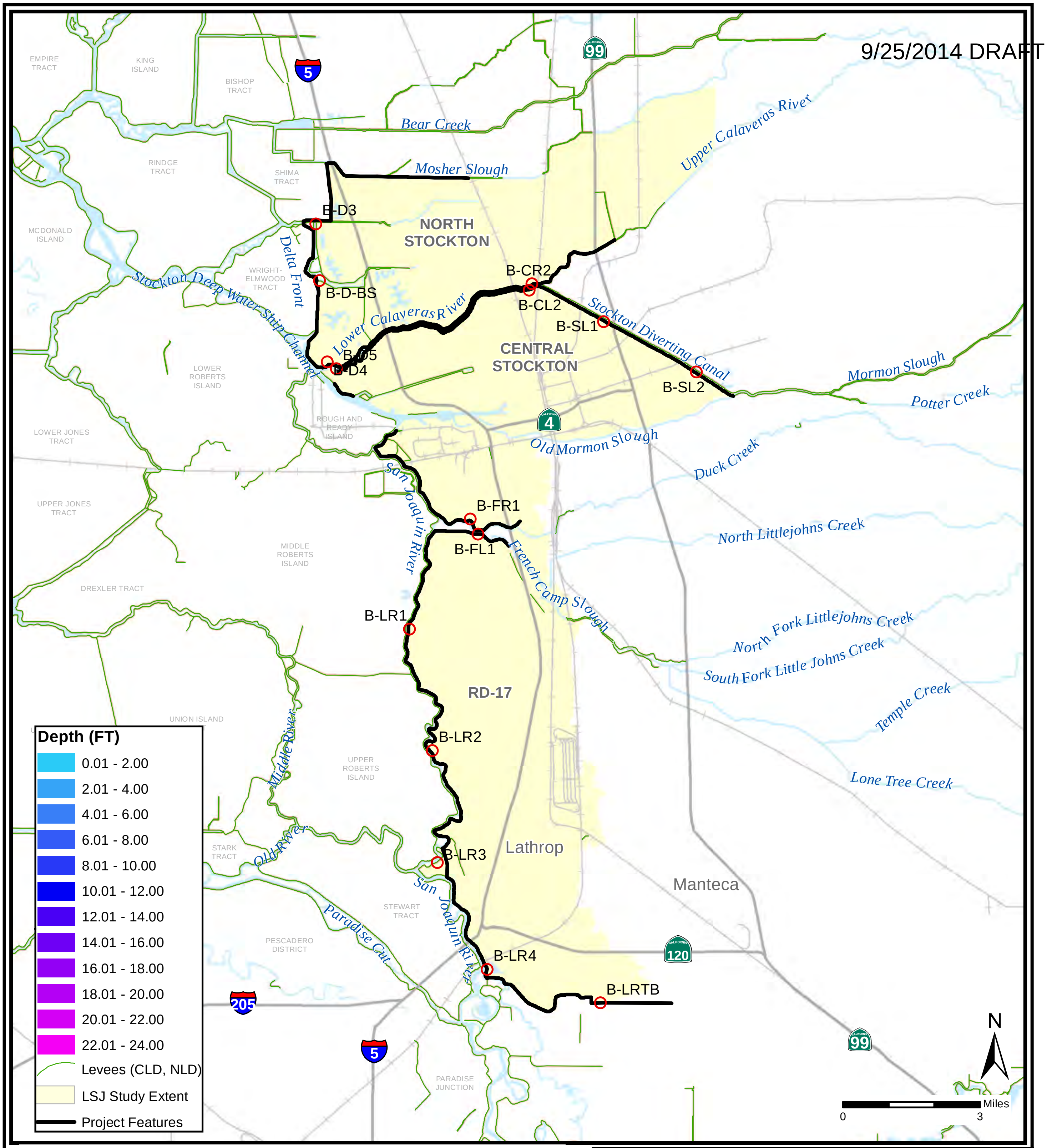
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 8B
4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

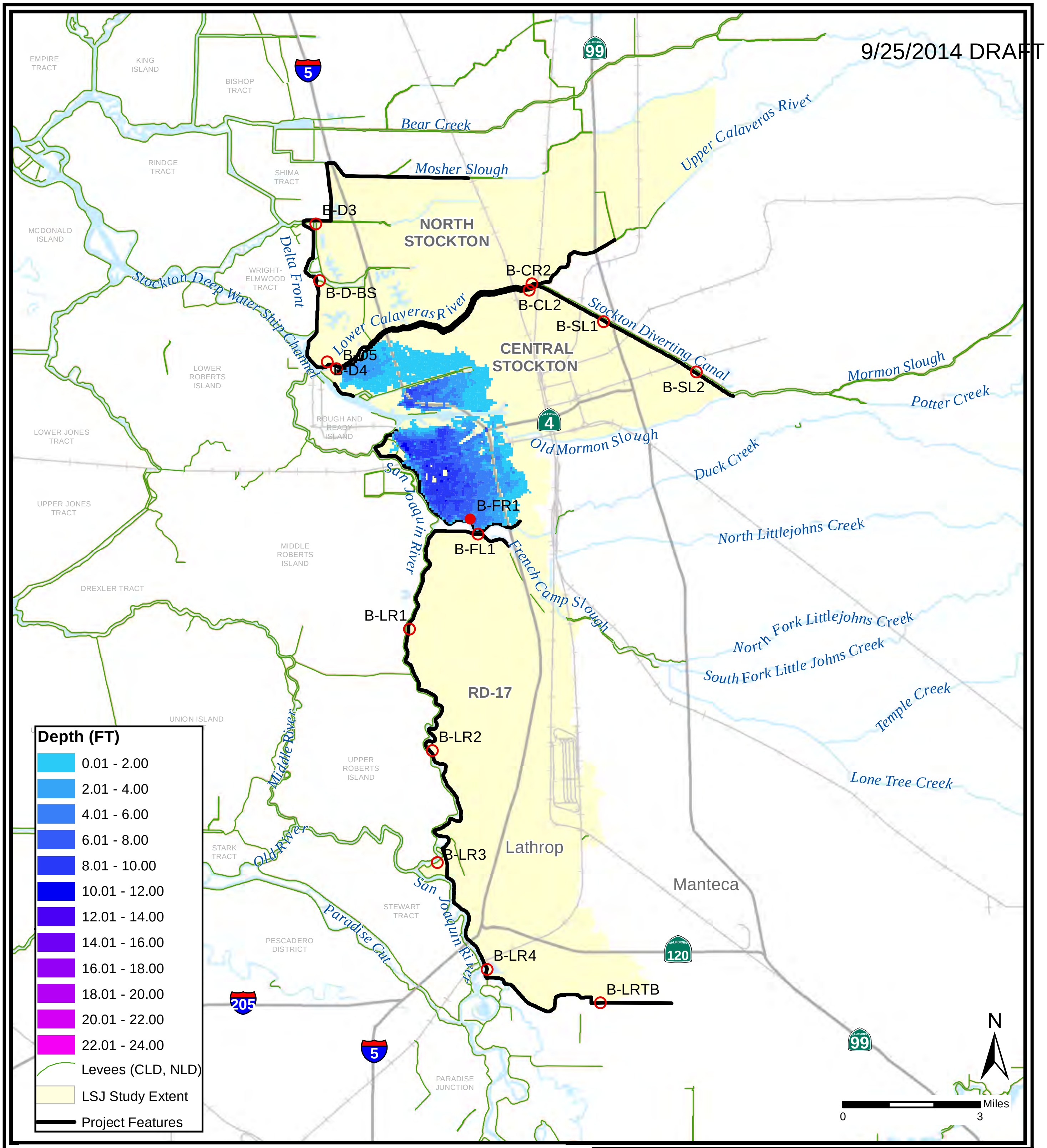
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 8B
2% (1/50) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 83



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

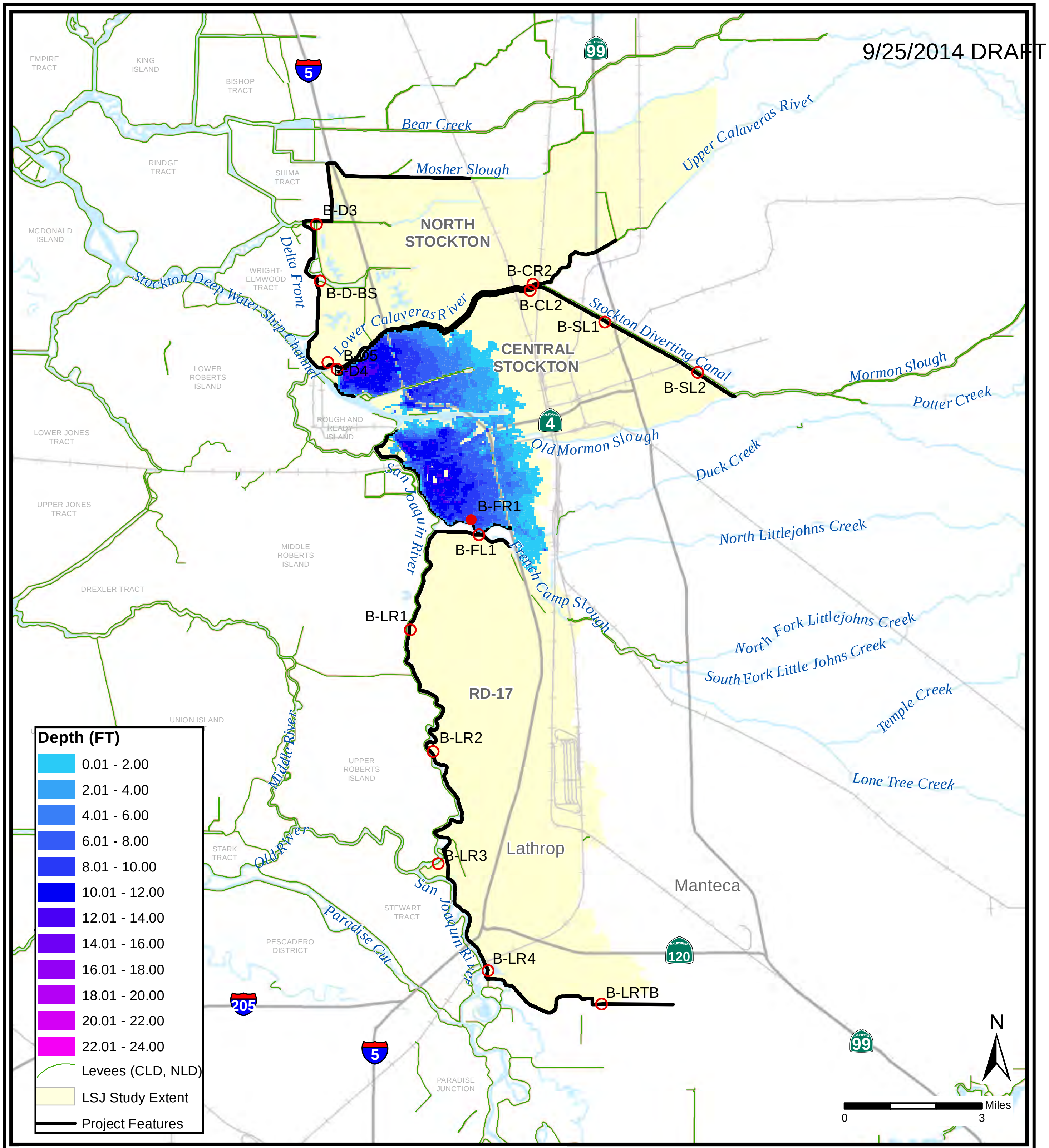
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 8B
1% (1/100) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 84



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

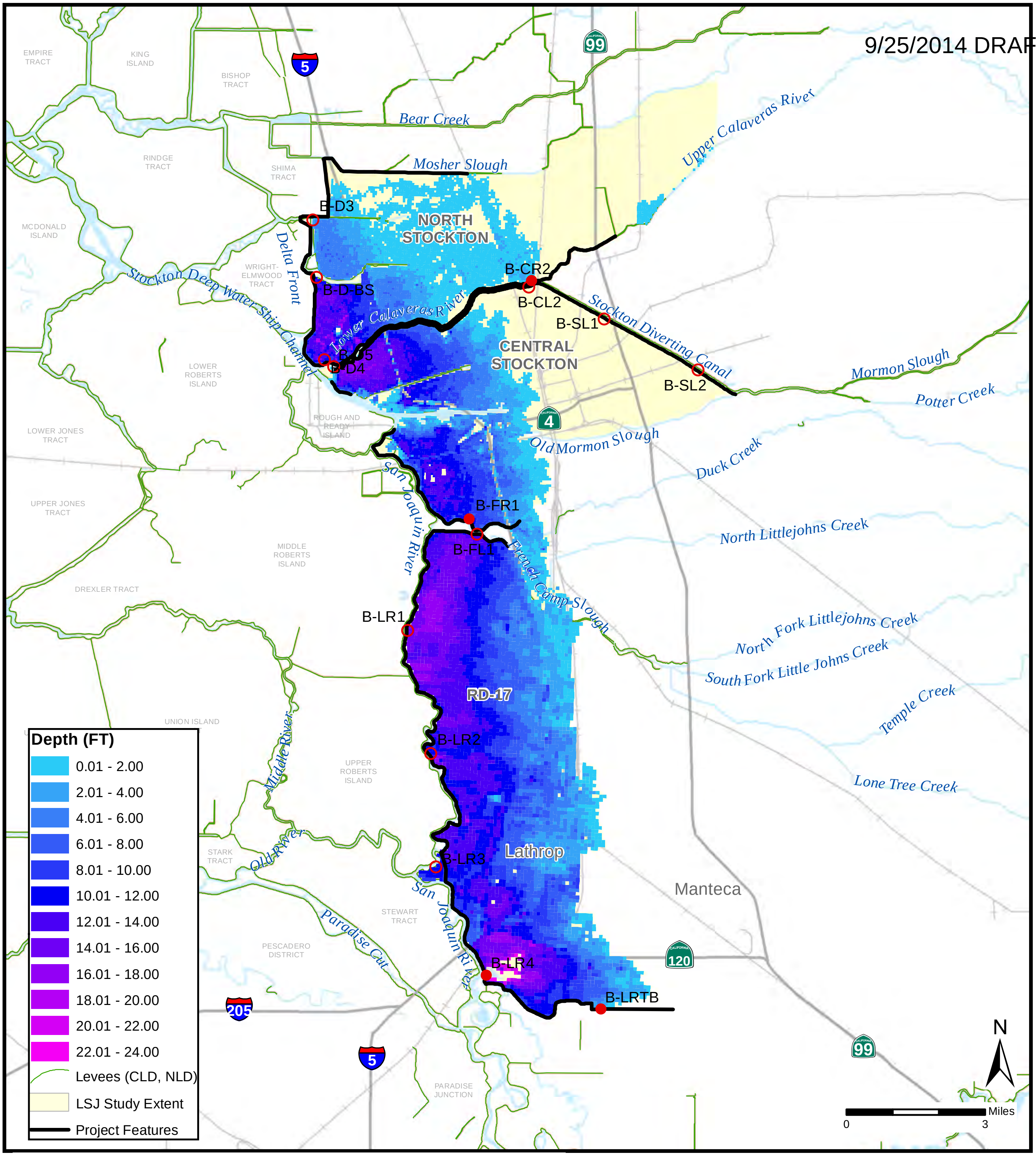
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 8B
0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014


9/25/2014 DRAFT

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

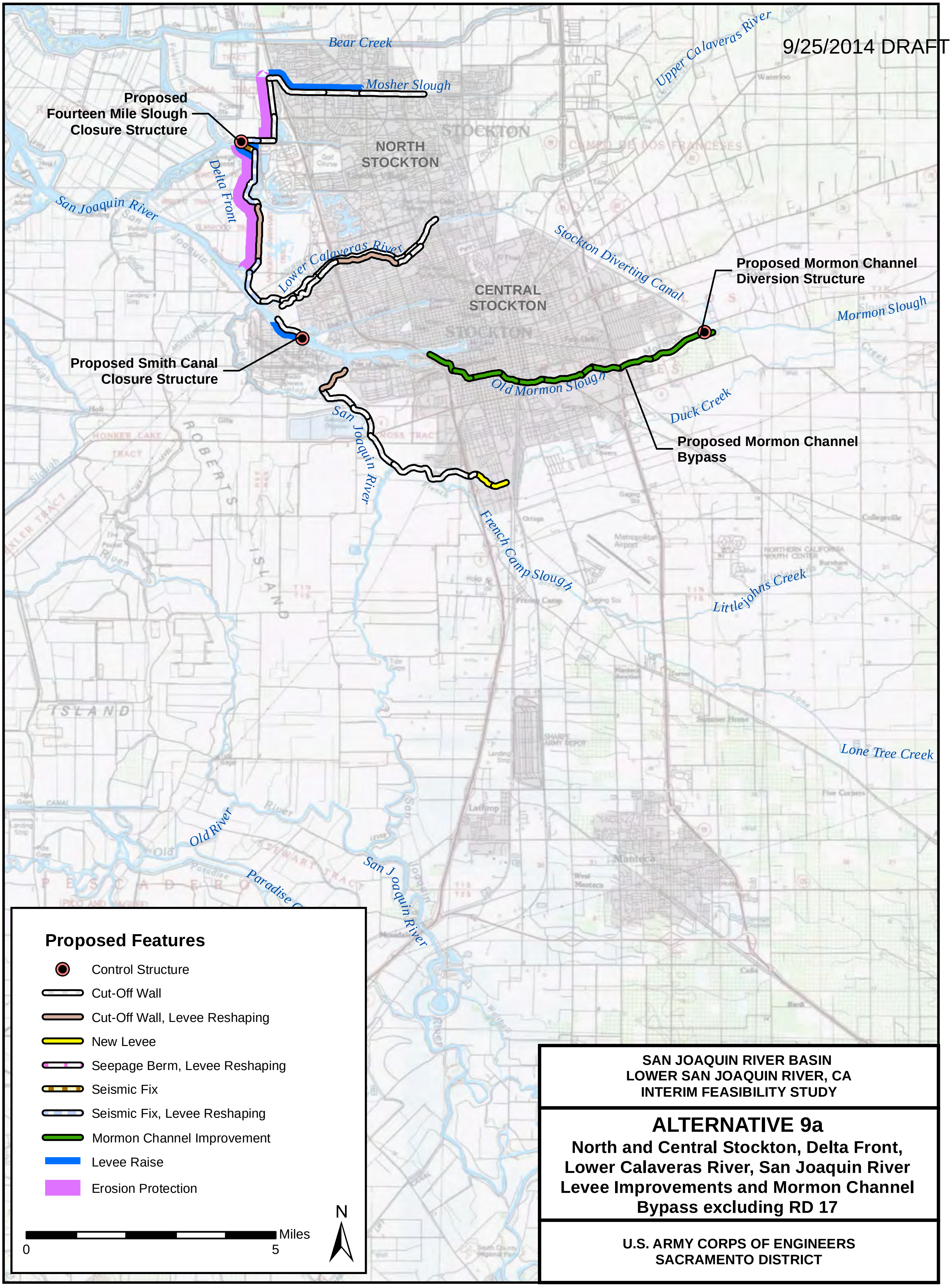
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 8B
0.2% (1/500) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

9/25/2014 DRAFT



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

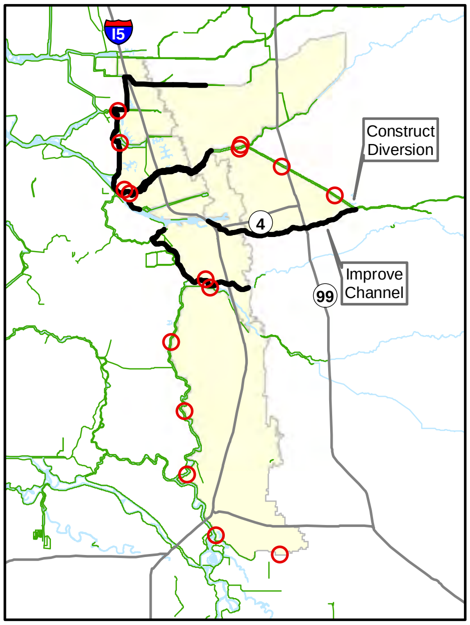
ALTERNATIVE 9a

North and Central Stockton, Delta Front, Lower Calaveras River, San Joaquin River Levee Improvements and Mormon Channel Bypass excluding RD 17

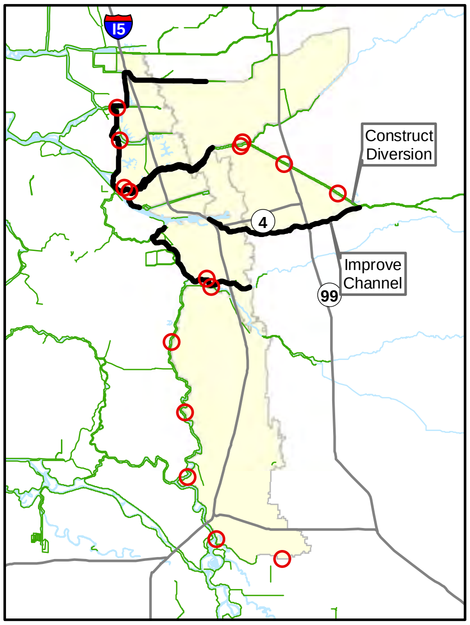
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

JUN 2014

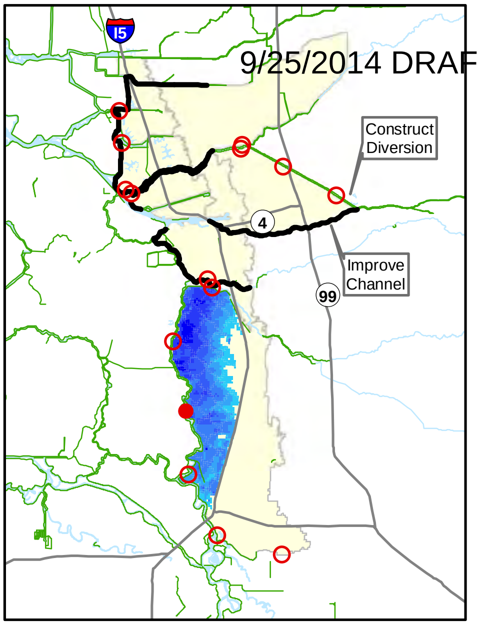
9/25/2014 DRAFT



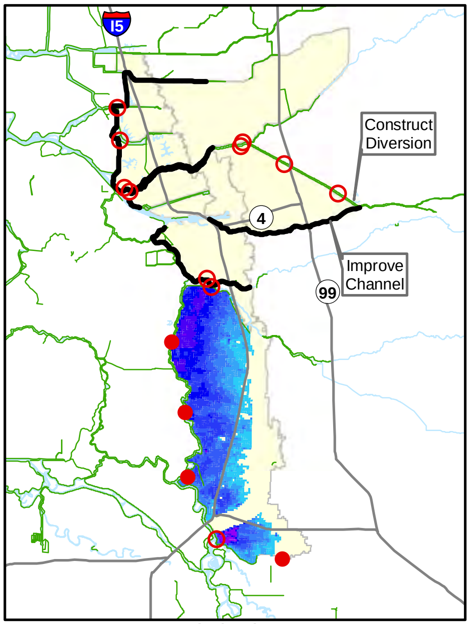
50% (1/2) ACE



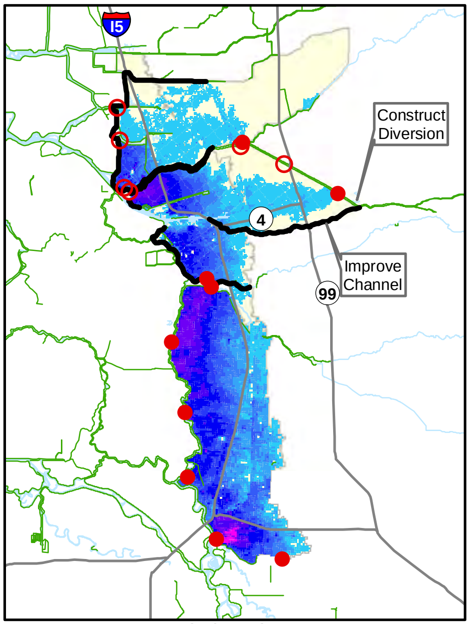
10% (1/10) ACE



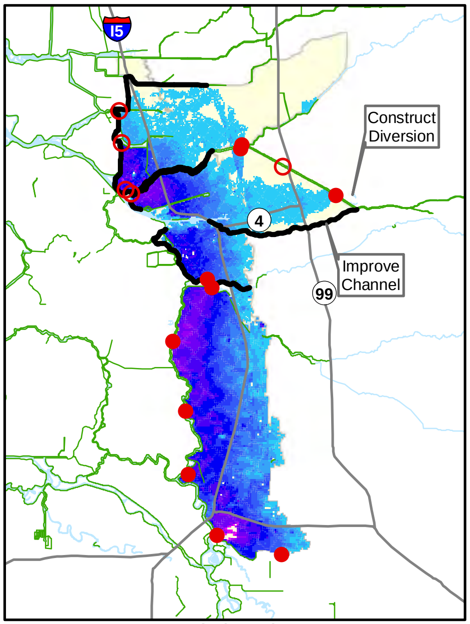
4% (1/25) ACE



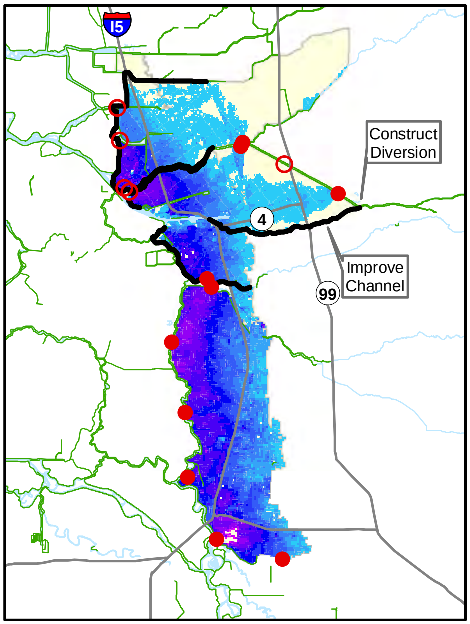
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Fails R&U Criteria
○ Meets R&U Criteria

Depth (FT)

- 🟦 0.01 - 2.00
- 🟦 2.01 - 4.00
- 🟦 4.01 - 6.00
- 🟦 6.01 - 8.00
- 🟦 8.01 - 10.00
- 🟦 10.01 - 12.00
- 🟪 12.01 - 14.00
- 🟪 14.01 - 16.00
- 🟪 16.01 - 18.00
- 🟪 18.01 - 20.00
- 🟪 20.01 - 22.00
- 🟪 22.01 - 24.00
- Levees (CLD, NLD)
- 🟨 LSJ Study Extent
- Project Features

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

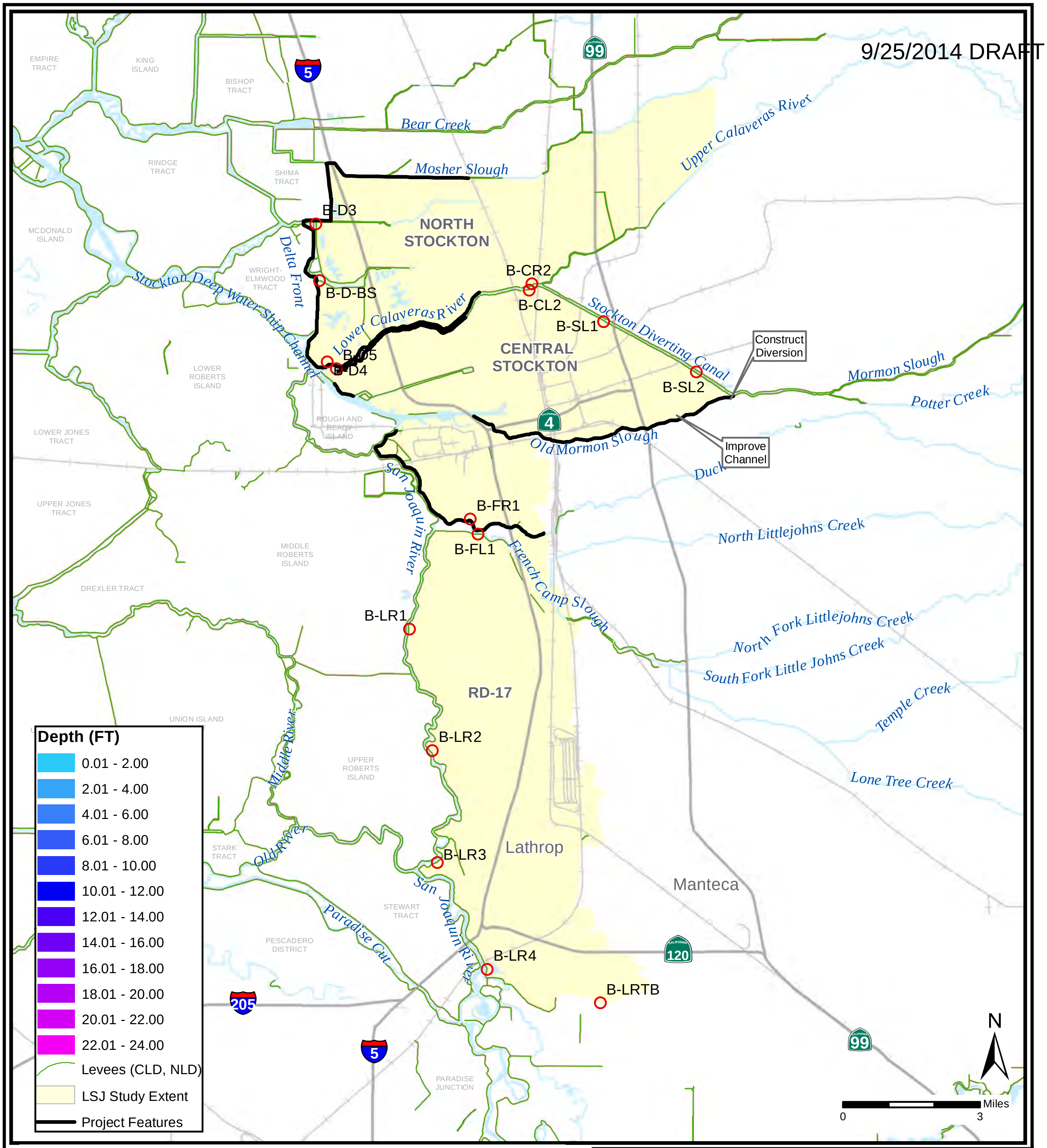
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9A

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by C. Young

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

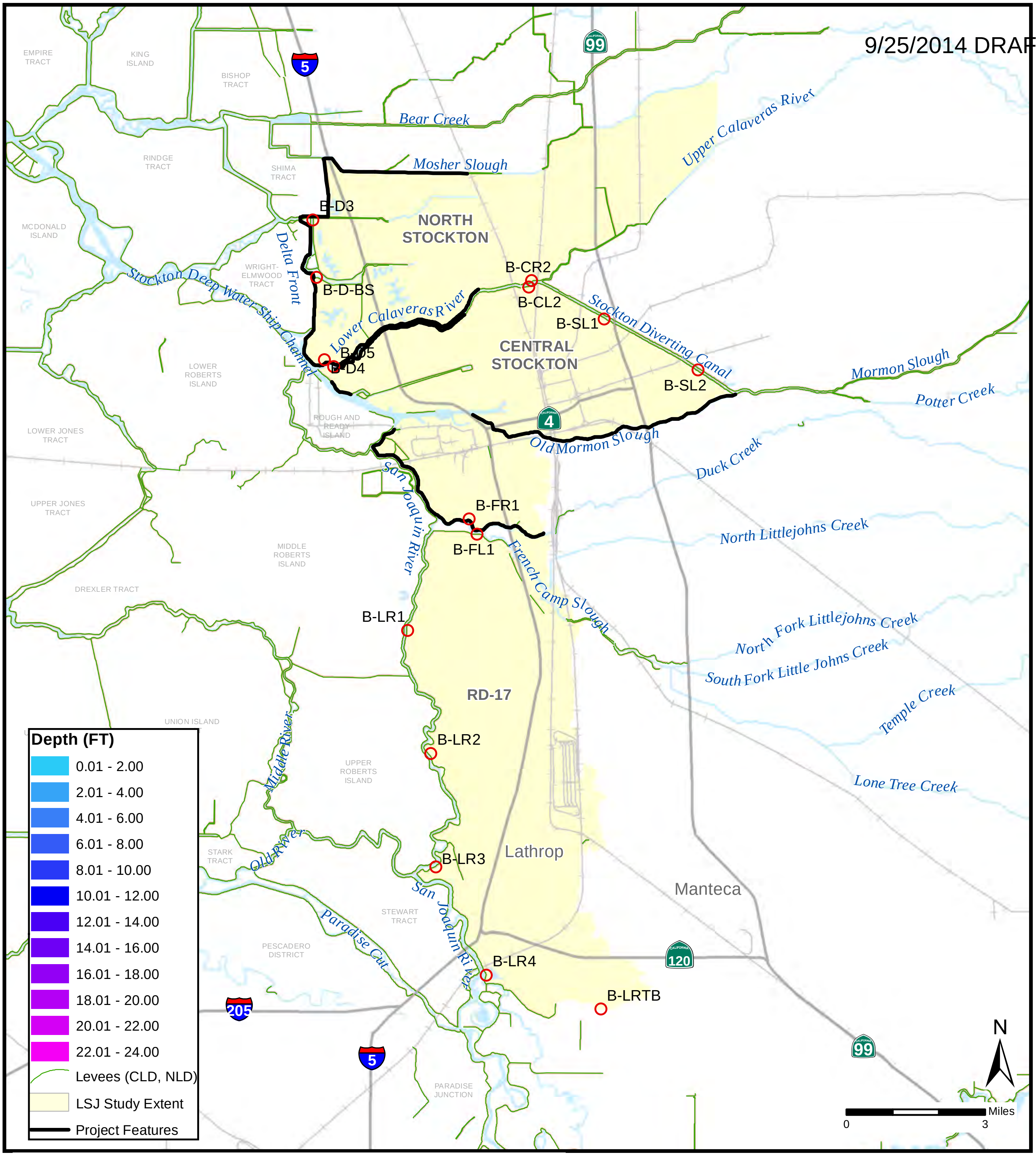
R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9A 50% (1/2) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 89



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

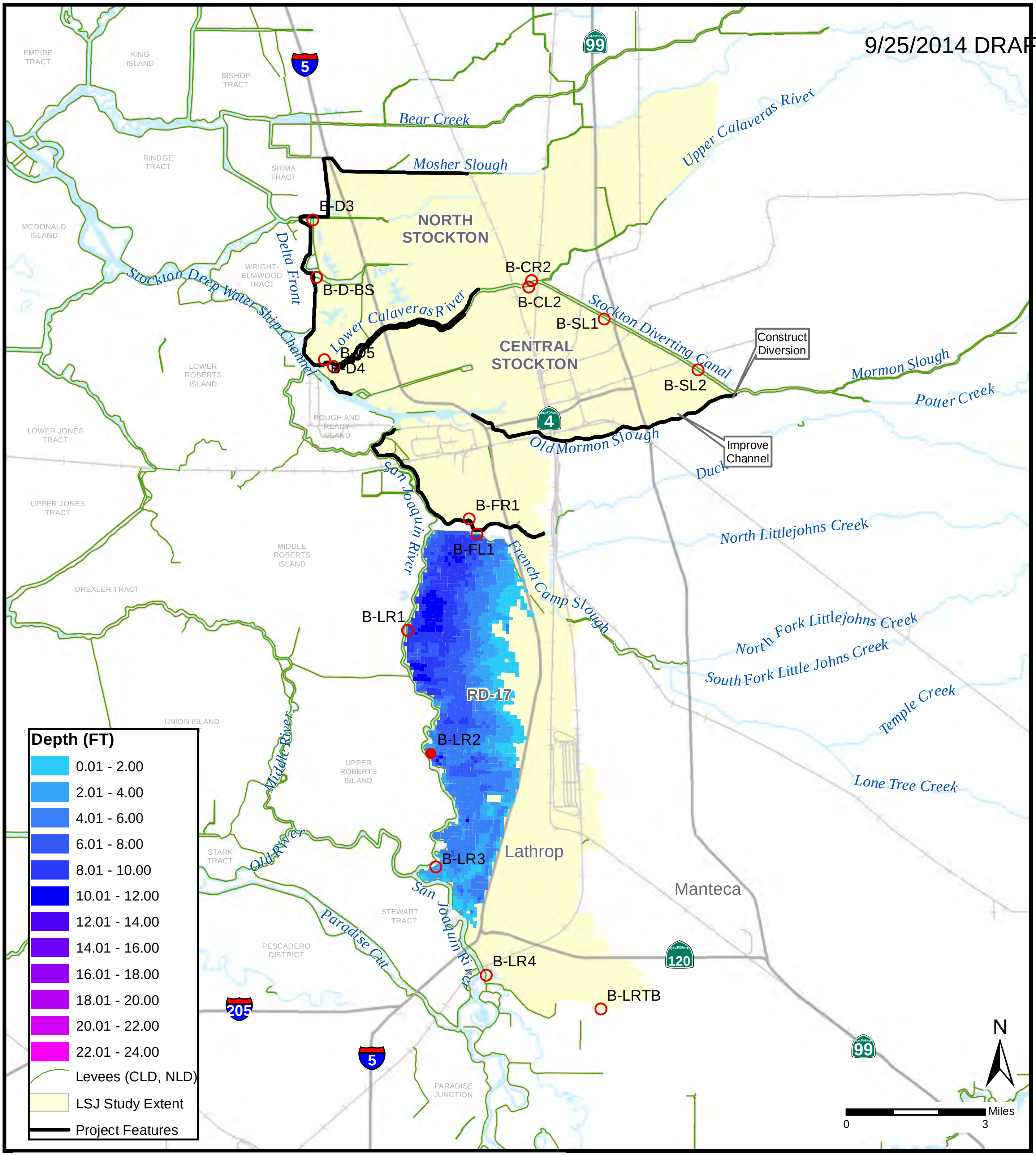
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9A
10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 90



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

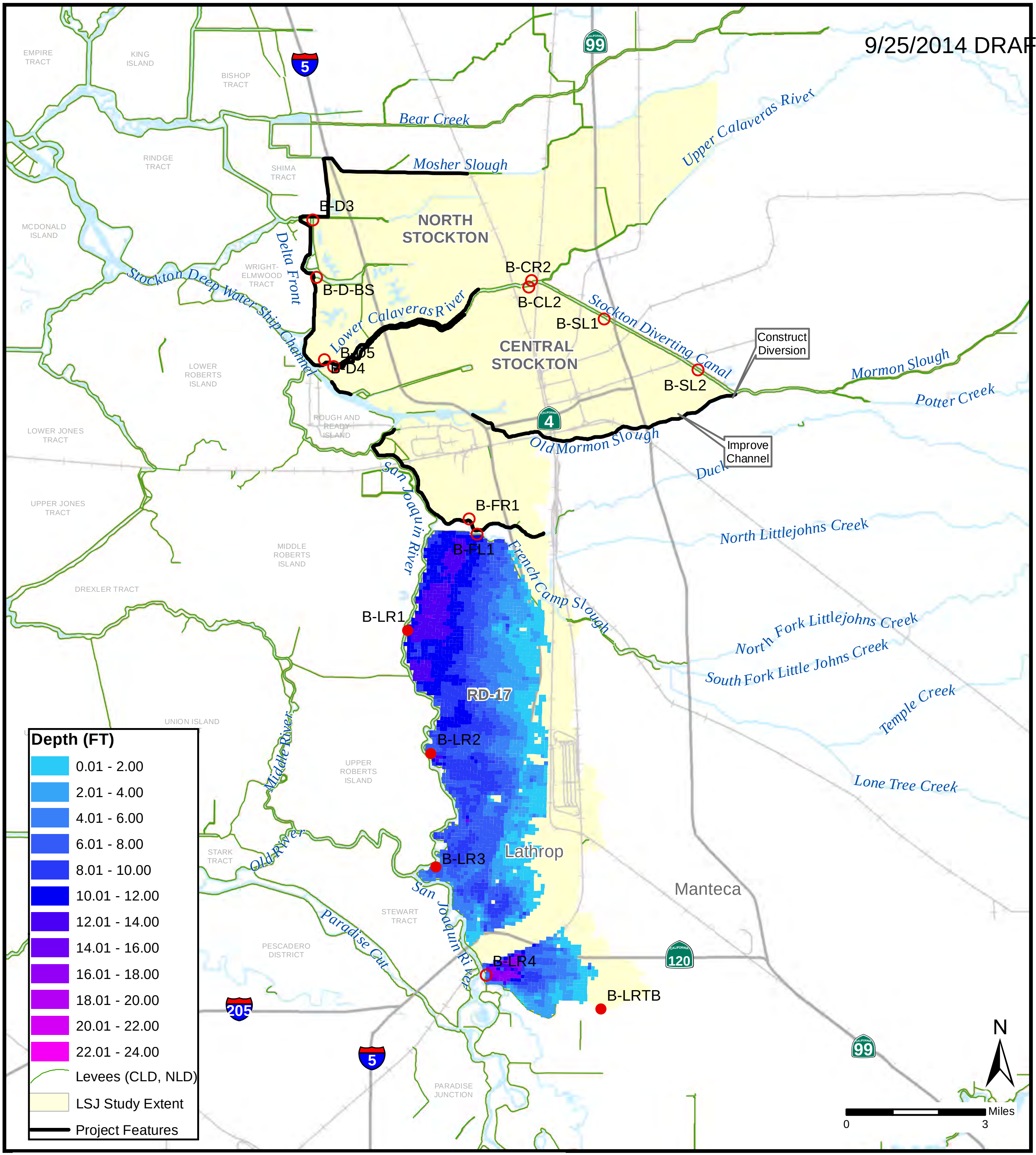
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9A
4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 91



9/25/2014 DRAFT

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

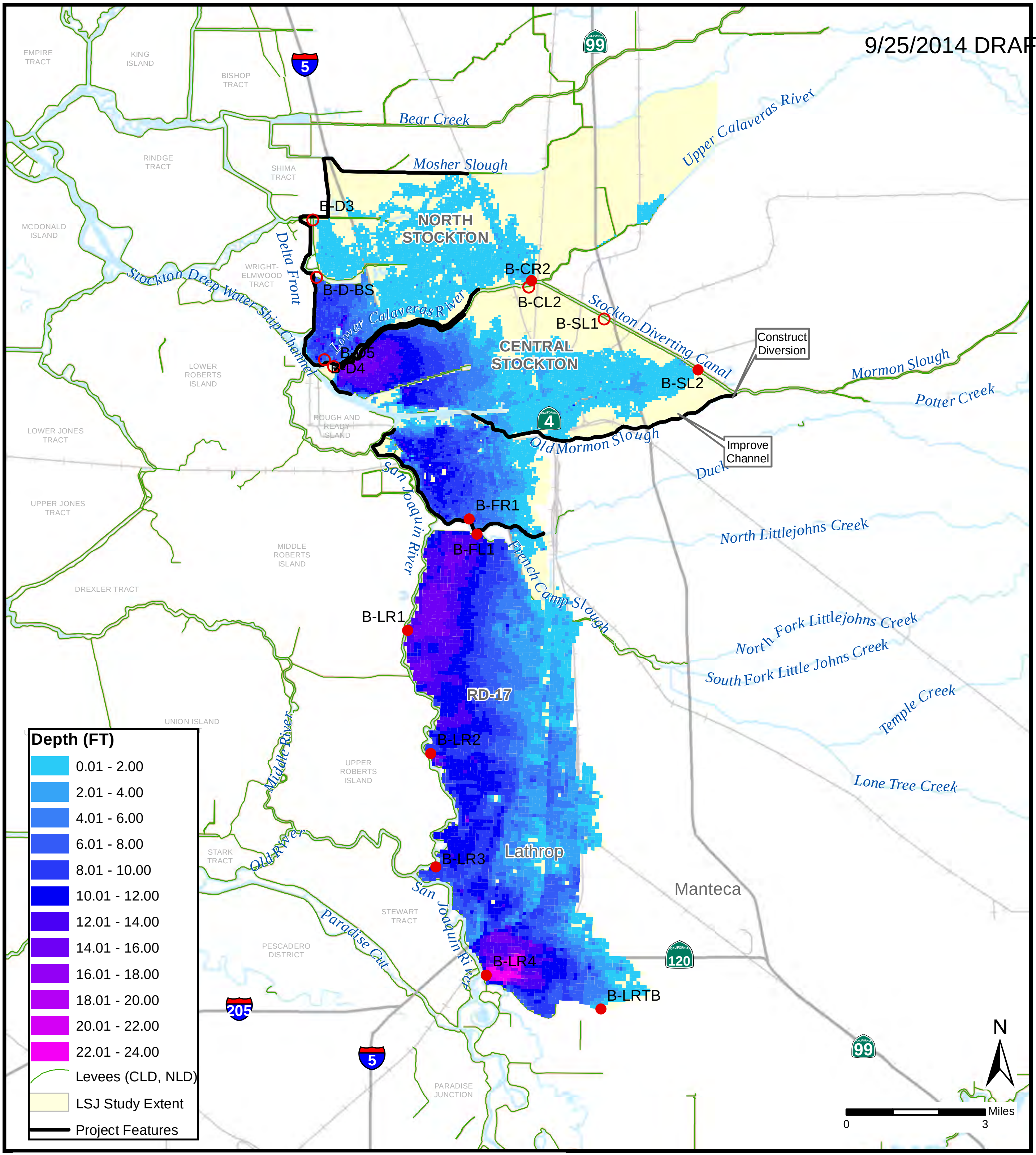
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9A
2% (1/50) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

9/25/2014 DRAFT



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

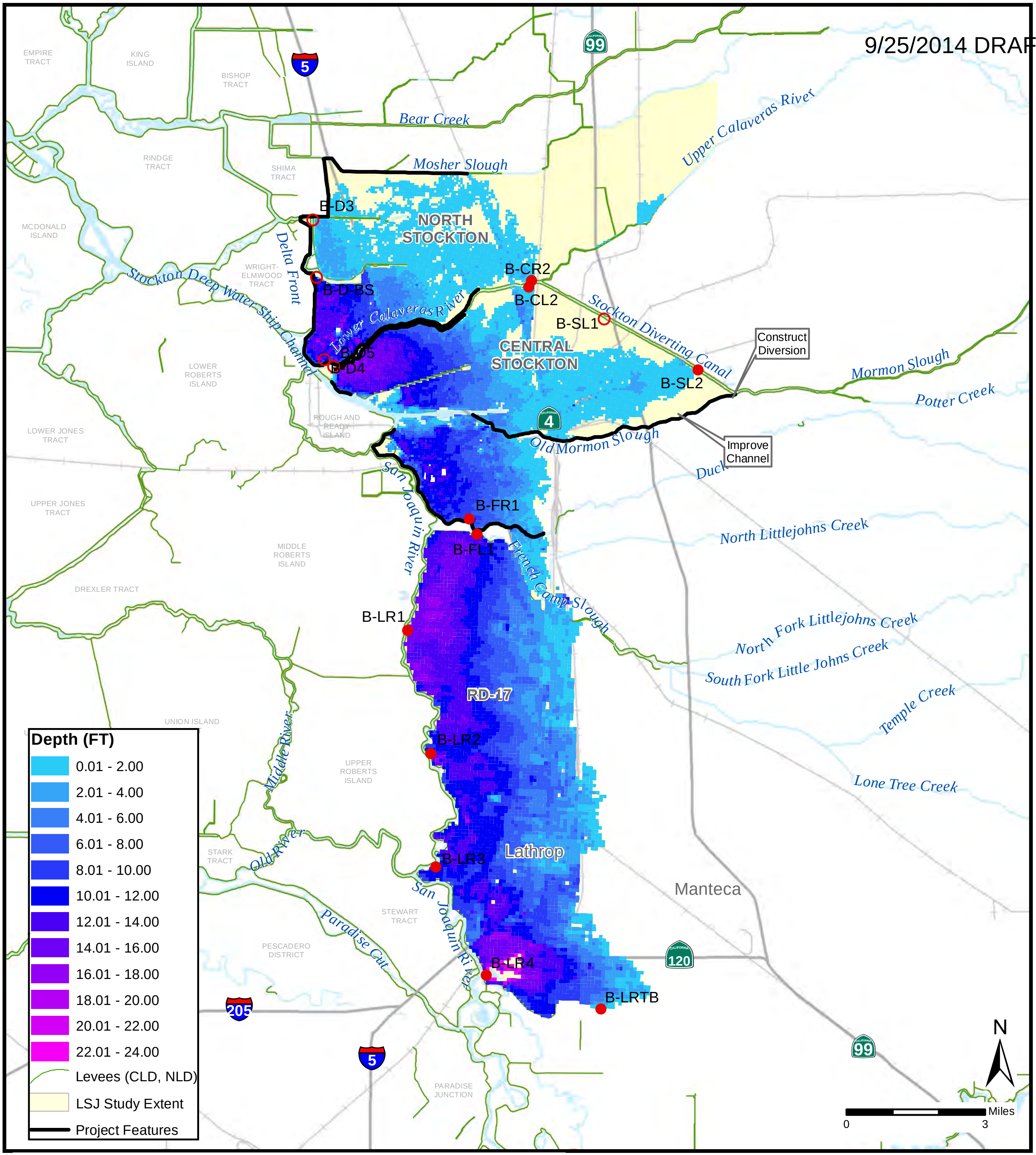
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9A
1% (1/100) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 93



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

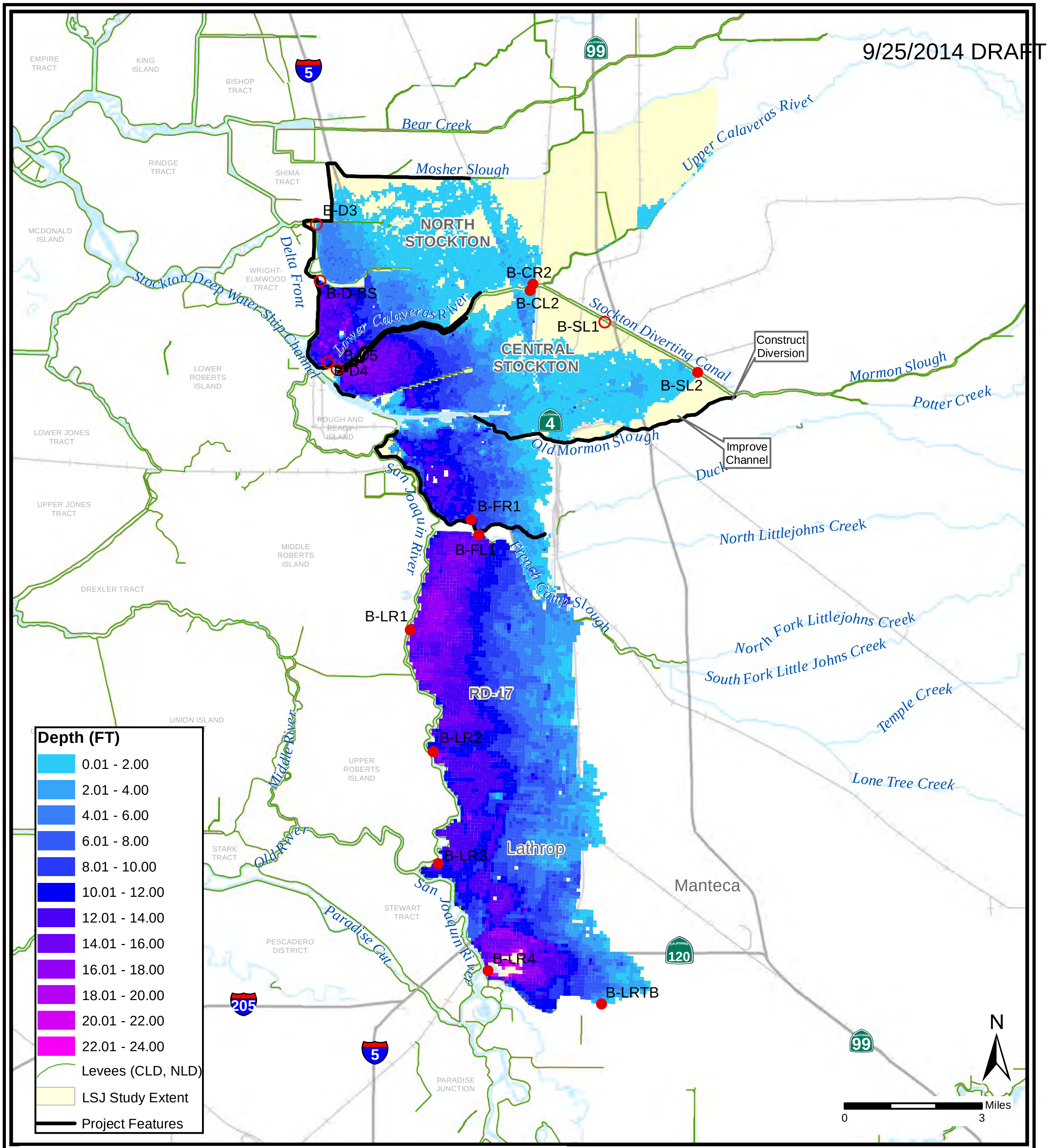
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9A
0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 94



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9A
0.2% (1/500) ACE

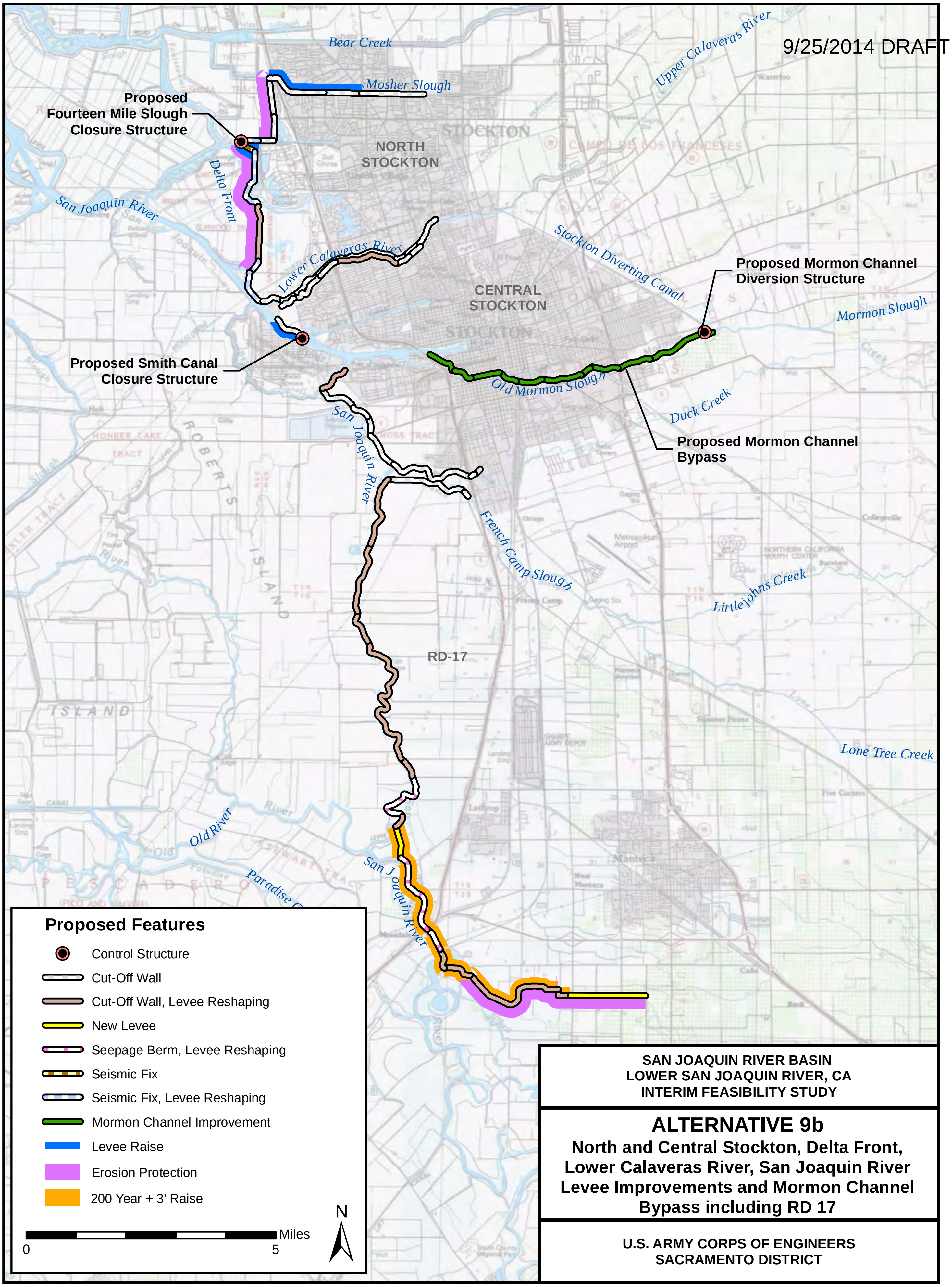
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 95

9/25/2014 DRAFT



SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

ALTERNATIVE 9b

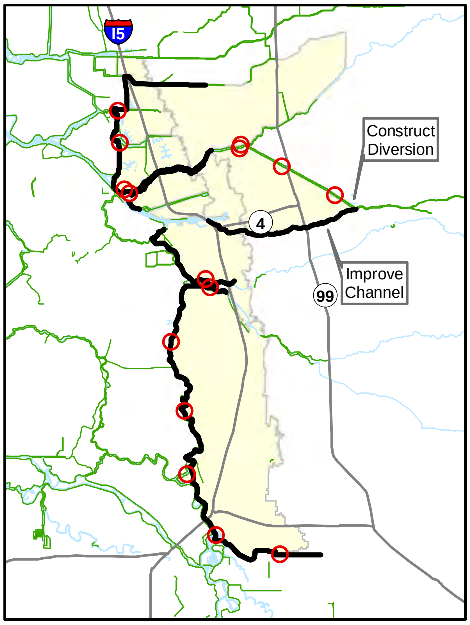
North and Central Stockton, Delta Front, Lower Calaveras River, San Joaquin River Levee Improvements and Mormon Channel Bypass including RD 17

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

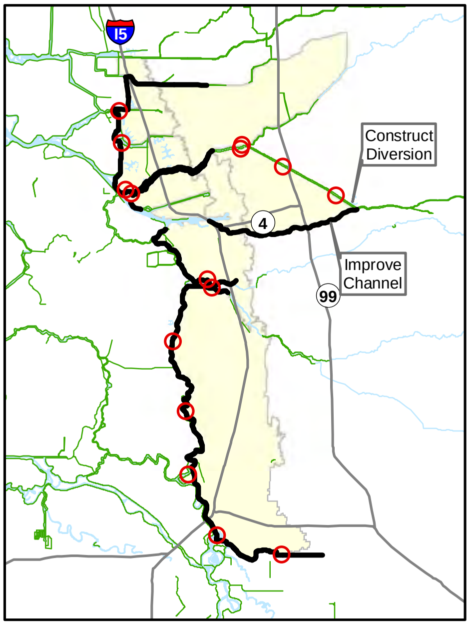
JUN 2014

PLATE 96

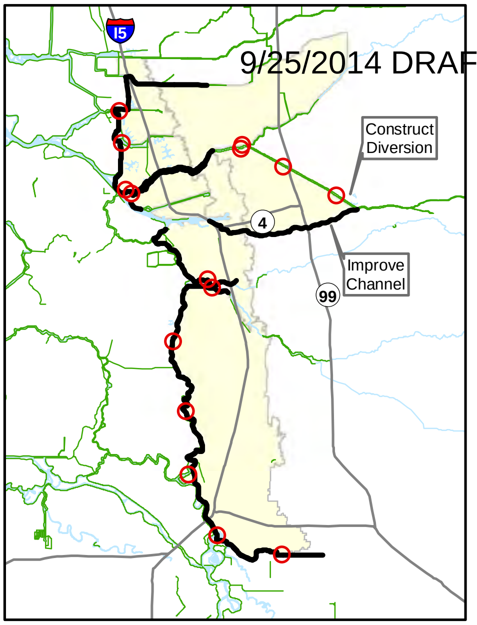
9/25/2014 DRAFT



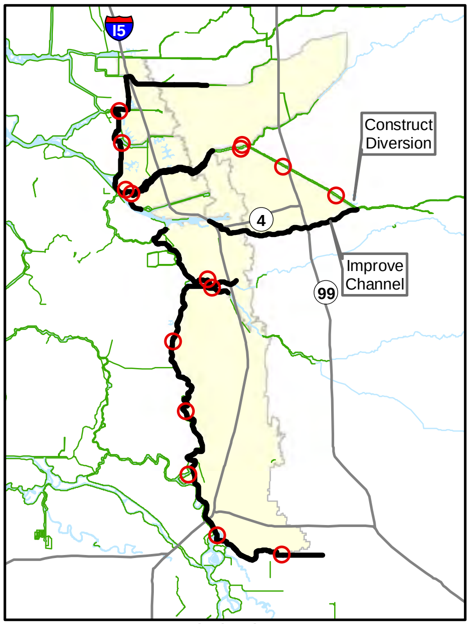
50% (1/2) ACE



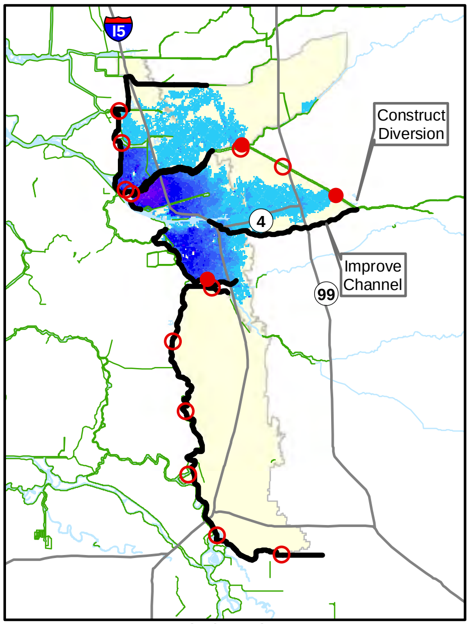
10% (1/10) ACE



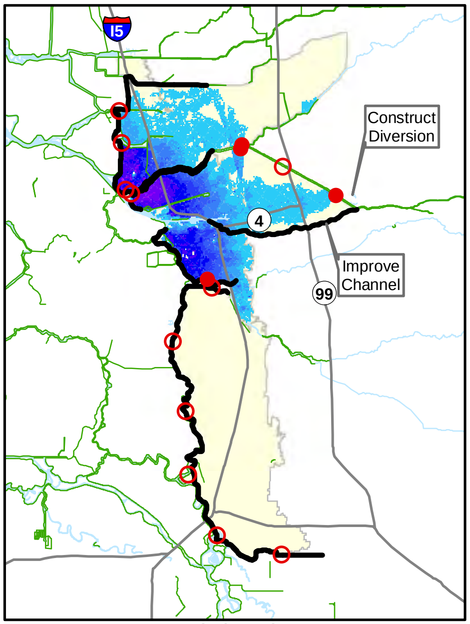
4% (1/25) ACE



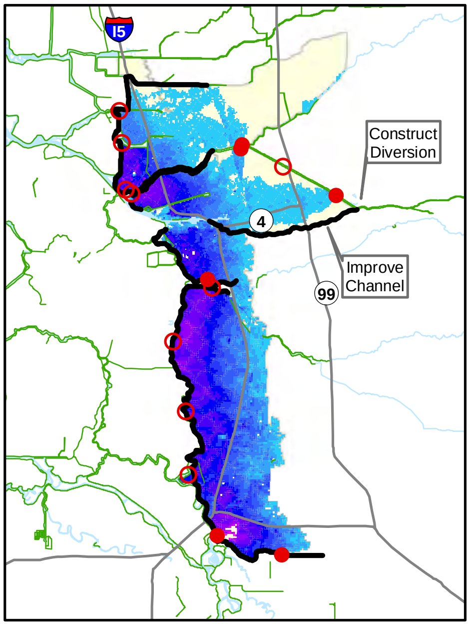
2% (1/50) ACE



1% (1/100) ACE



0.5% (1/200) ACE



0.2% (1/500) ACE

● Fails R&U Criteria

○ Meets R&U Criteria

Depth (FT)

- ■ 0.01 - 2.00
- ■ 2.01 - 4.00
- ■ 4.01 - 6.00
- ■ 6.01 - 8.00
- ■ 8.01 - 10.00
- ■ 10.01 - 12.00
- ■ 12.01 - 14.00
- ■ 14.01 - 16.00
- ■ 16.01 - 18.00
- ■ 18.01 - 20.00
- ■ 20.01 - 22.00
- ■ 22.01 - 24.00
- ～ Levees (CLD, NLD)
- ▭ LSJ Study Extent
- ━ Project Features

Miles
0 5

N

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

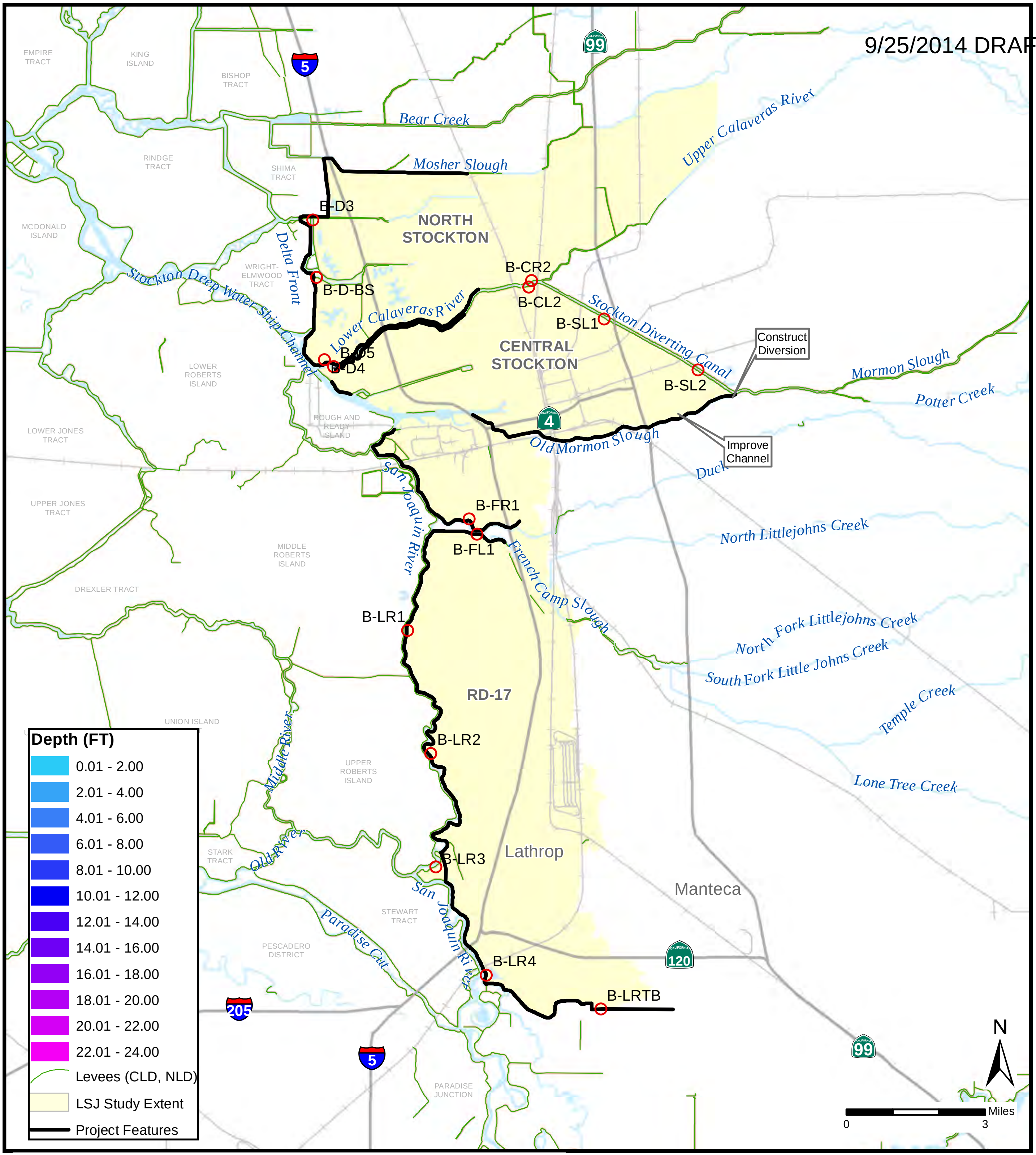
**SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY**

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9B

**U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT**

Prepared by C. Young

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

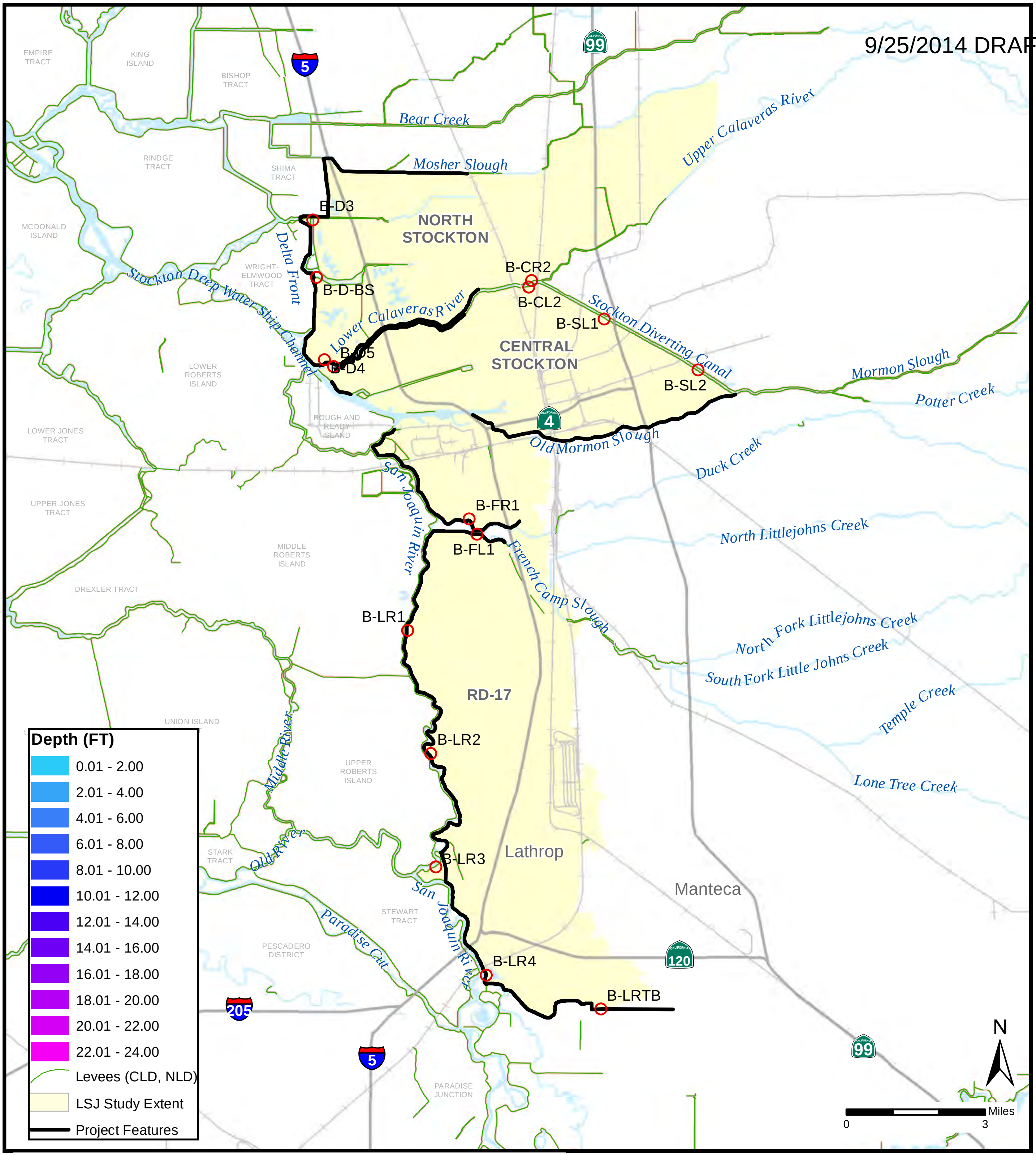
SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9B
50% (1/2) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

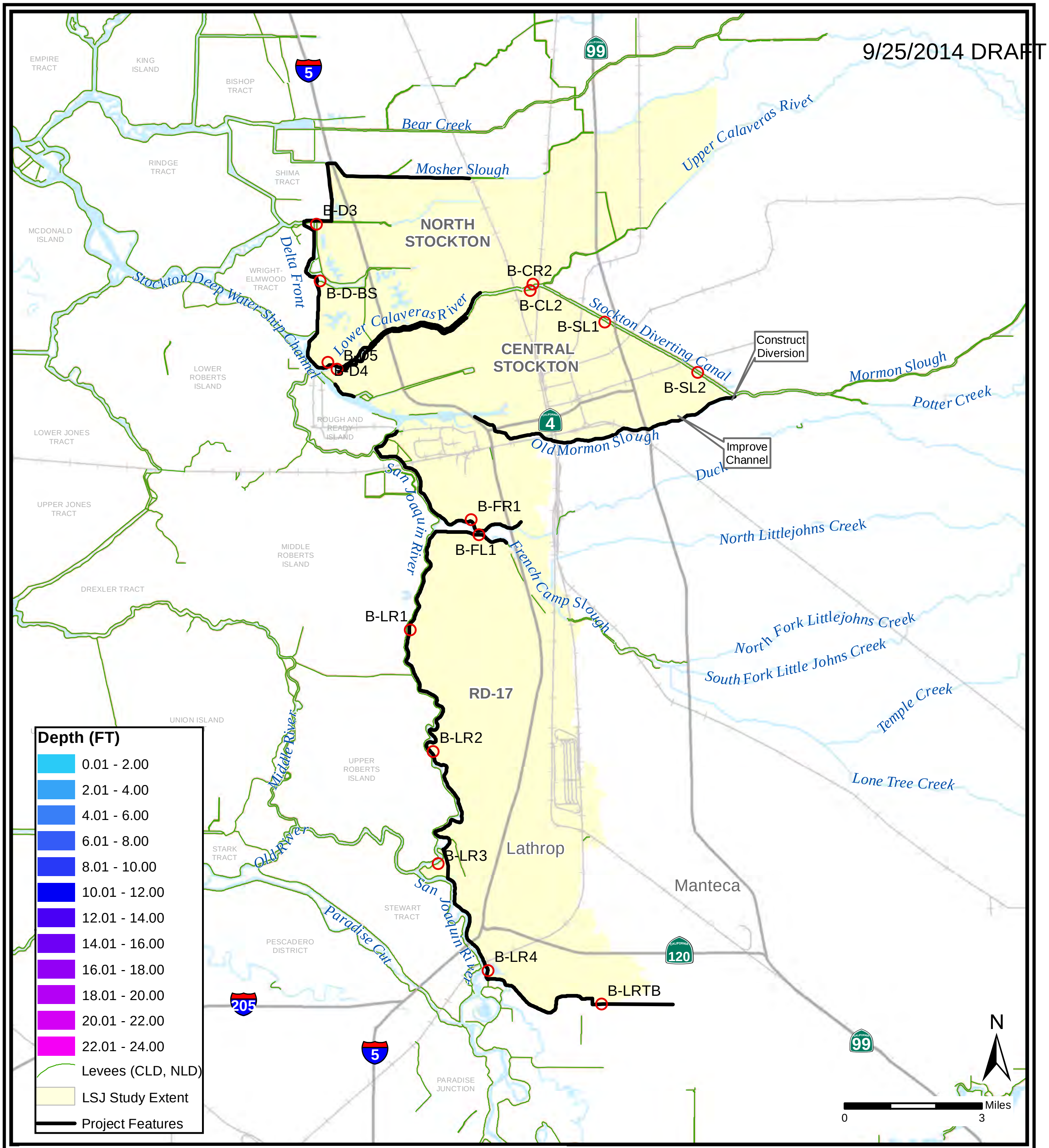
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9B
10% (1/10) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 99



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

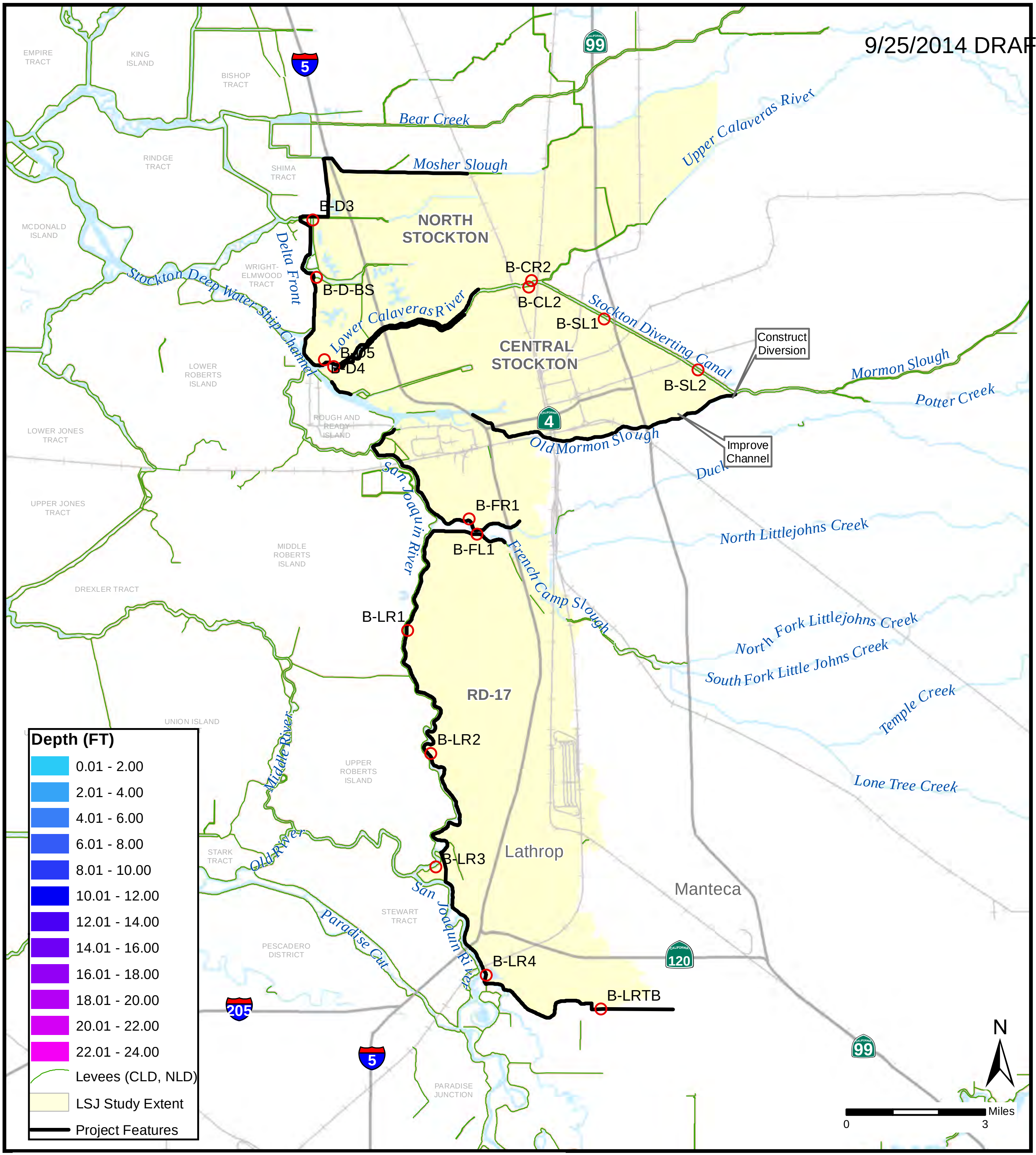
R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9B
4% (1/25) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 100



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9B 2% (1/50) ACE

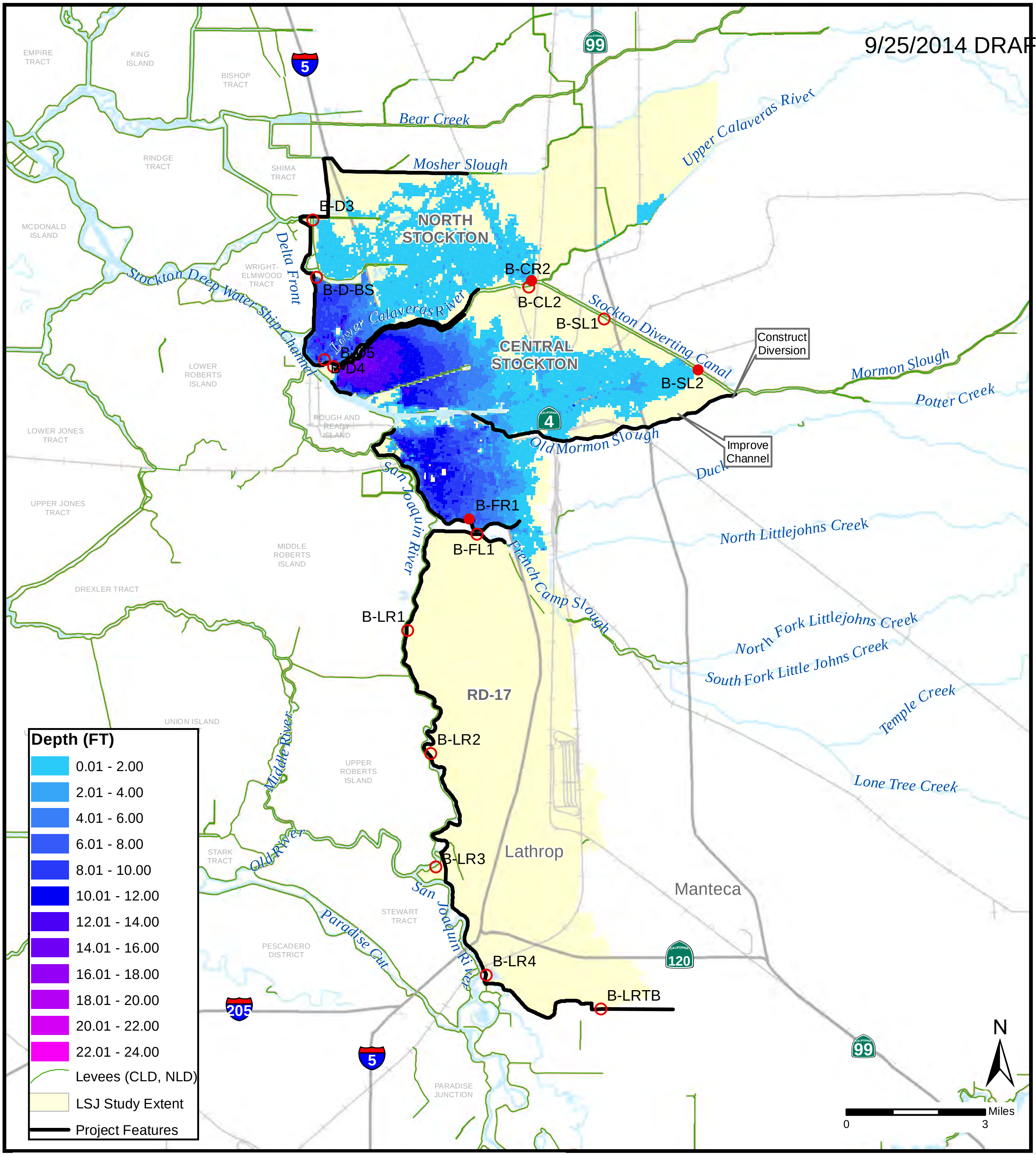
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 101

9/25/2014 DRAFT



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria
○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9B
1% (1/100) ACE

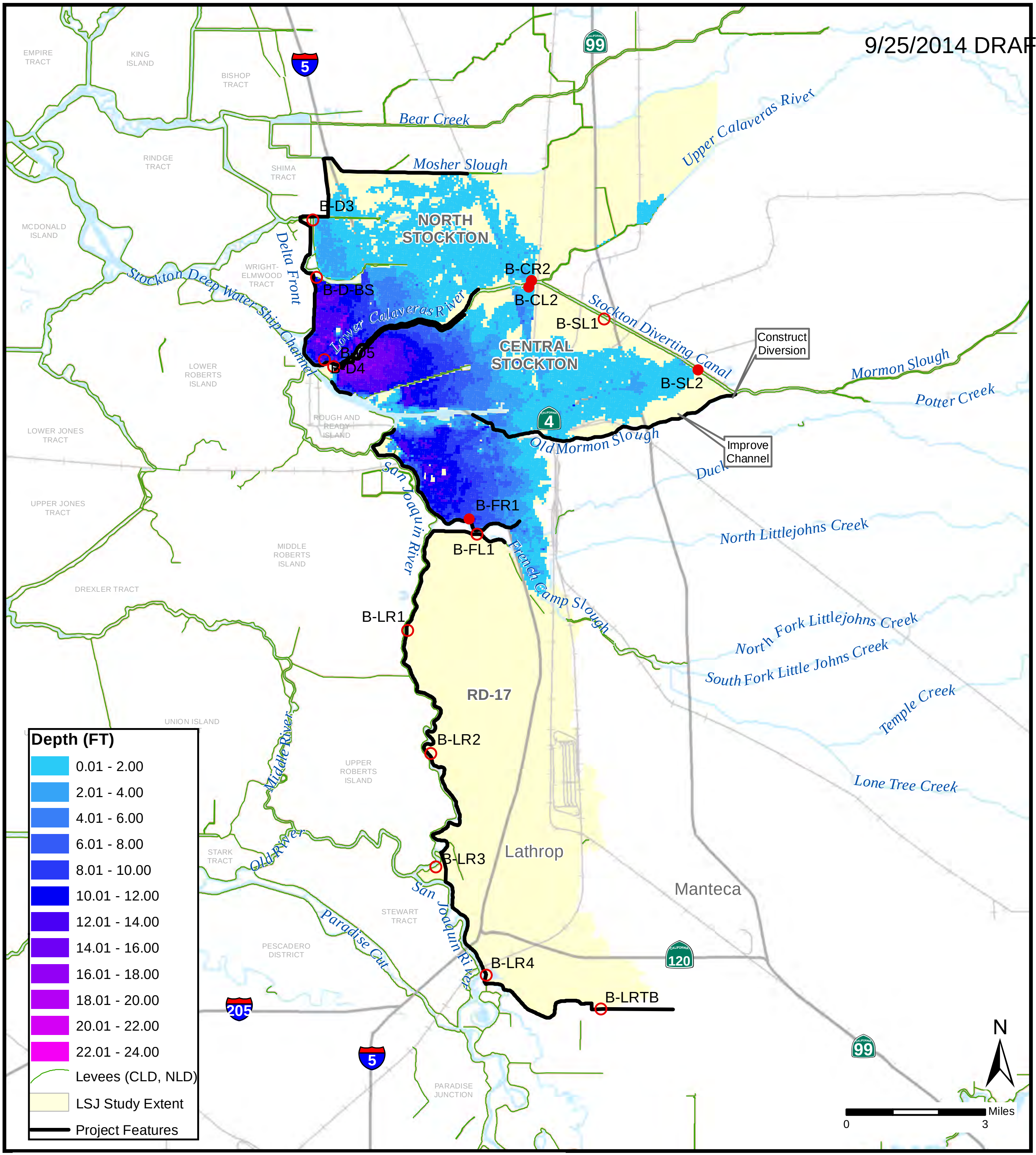
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 102

9/25/2014 DRAFT



NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

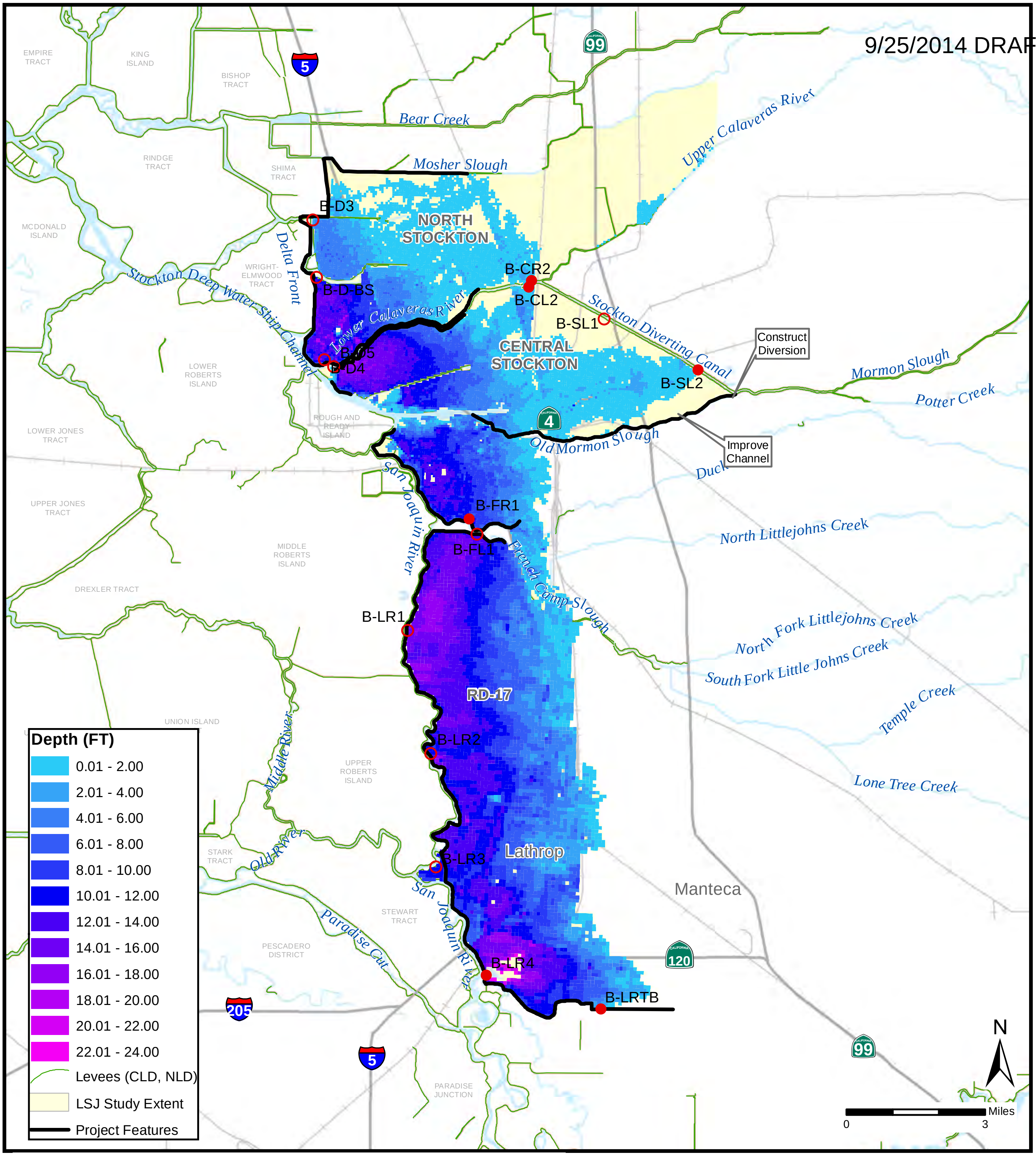
R&U COMPOSITE FLOODPLAIN ALTERNATIVE - 9B 0.5% (1/200) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 103



9/25/2014 DRAFT

NOTE: Breach simulation shown if levee does not pass assurance criteria.1) Assurance less than 90% the levee does not pass criteria 2) For assurance between 90 and 95% levee must have minimum of 3 feet of freeboard to pass criteria. 3) For assurance greater than 95% levee must have minimum of 2 feet of freeboard to pass criteria

Composite Floodplains only shown within Study Extent

● Fails R&U Criteria

○ Meets R&U Criteria

SAN JOAQUIN RIVER BASIN
LOWER SAN JOAQUIN RIVER, CA
INTERIM FEASIBILITY STUDY

R&U COMPOSITE FLOODPLAIN
ALTERNATIVE - 9B
0.2% (1/500) ACE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by V. Nino-Tapia

JUN 2014

PLATE 104

QUALITY CONTROL CERTIFICATE

Hydraulic Analysis Section, Engineering Division

PROJECT NAME: SAN JOAQUIN BASIN, INTERIM FEASIBILITY STUDY
PRODUCT: HYDRAULIC DESIGN APPENDIX TO FEASIBILITY STUDY REPORT
Actual Completion Date: 28-Aug-14

PROJECT MANAGER: JOANA SAVIGNON

Background: [Include project description, technical products, and review methodology]

District Quality Control was performed on the August 2014 report "Lower San Joaquin River Feasibility Report - Environmental Impact Report/Environmental Impacts Statement, San Joaquin County, California, and Hydraulic Design Appendix. Review comments, responses, and back checks on the report are located in the folder with the memorandum. Supporting memoranda including quality control documents are located in the memorandum folder.

HYDRAULIC LEAD

I have ensured that the above products were prepared in accordance with standard quality control practices. I have also incorporated or resolved all issues identified during District Quality Control (DQC) review.

Hydraulic Lead: Peter Blodgett — Title: Senior Hydraulic Engineer
PETER BLODGETT — Print name
Signature
8/29/2014 — Date

REVIEWERS

I have reviewed the products noted above and find them to be in accordance with project requirements, standards of the profession, and USACE policies and standards.

DQC Reviewer: Ethan Thompson — Title: Senior Hydraulic Engineer
Ethan Thompson — Print name
Signature
8/29/2014 — Date

RESOURCE PROVIDER

I have reviewed and resolved all critical and technical issues. I agree that all project requirements, standards of the profession, and USACE policies and standards have been met.

Section Chief: Jesse Schlunegger , Chief, Hydraulic Analysis Section
Print name
Jesse Schlunegger — Signature
9/18/2014 — Date

ATTACHMENT A

GEOTECHNICAL FRAGILITY CURVES

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

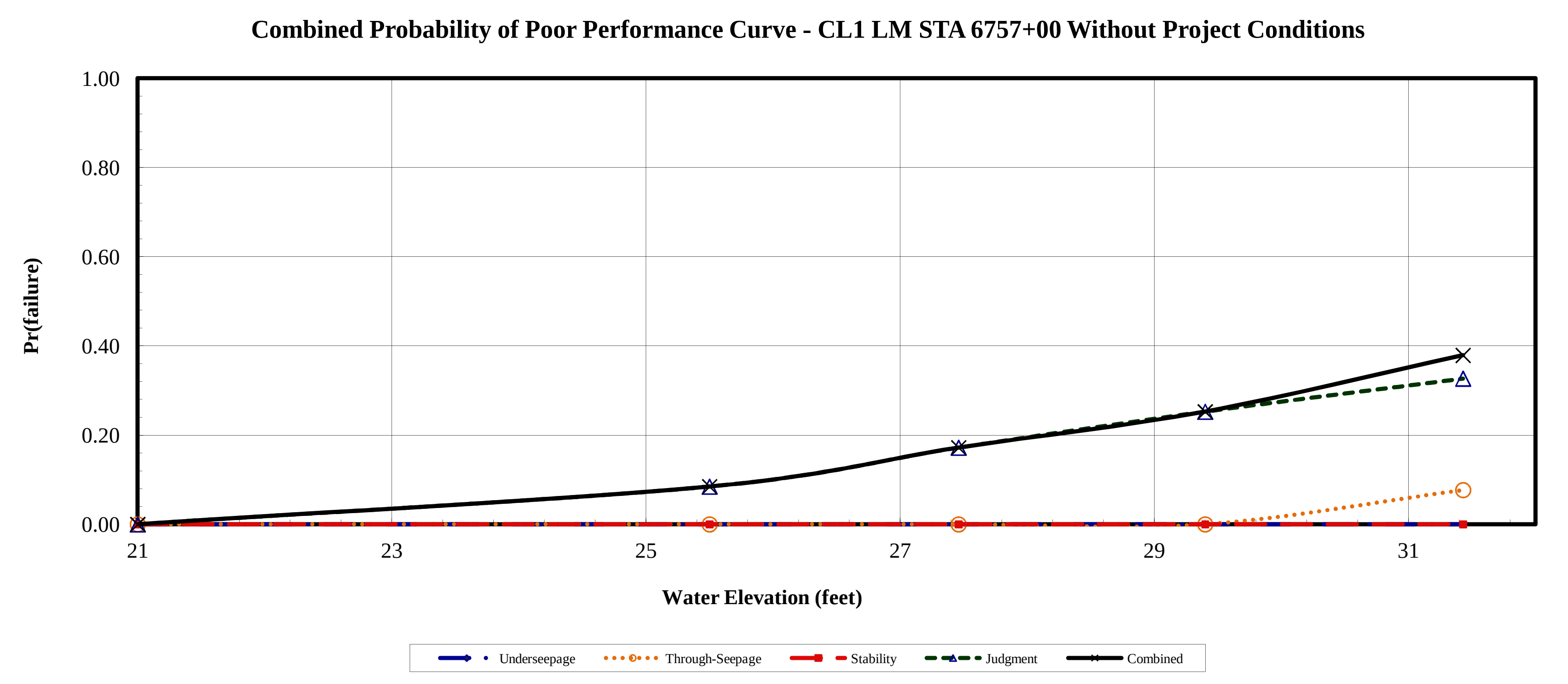
Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: CL1

Levee Mile: STA 6757+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 31.43
L/S Toe Elev.: 21.00
W/S Toe Elev.: 26.94

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hog
Date: 9/24/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
21.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
25.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0845	0.9155	0.0845	0.9155
27.46	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1719	0.8281	0.1719	0.8281
29.40	0.0001	0.9999	0.0000	1.0000	0.0000	1.0000	0.2526	0.7474	0.2527	0.7473
31.43	0.0004	0.9996	0.0769	0.9231	0.0001	0.9999	0.3268	0.6732	0.3790	0.6210


Combined Probability of Poor Performance Curve - CL1 LM STA 6757+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

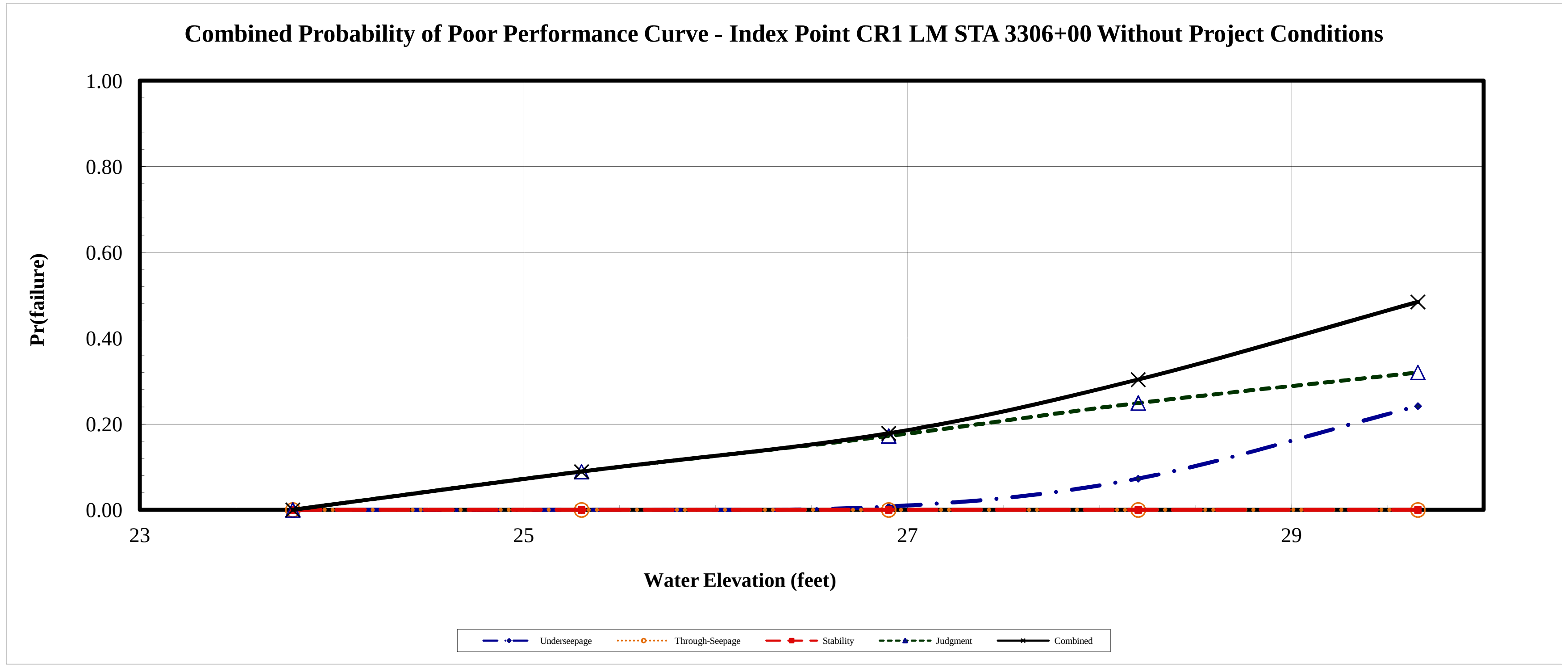
Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point CR1

Levee Mile: STA 3306+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 29.66
L/S Toe Elev.: 23.80
W/S Toe Elev.: 22.90

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hog
Date: 9/28/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
23.80	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
25.30	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0892	0.9108	0.0892	0.9108
26.90	0.0074	0.9926	0.0000	1.0000	0.0000	1.0000	0.1721	0.8279	0.1783	0.8217
28.20	0.0727	0.9273	0.0000	1.0000	0.0000	1.0000	0.2490	0.7510	0.3036	0.6964
29.66	0.2418	0.7582	0.0000	1.0000	0.0000	1.0000	0.3203	0.6797	0.4846	0.5154


Combined Probability of Poor Performance Curve - Index Point CR1 LM STA 3306+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

Project: Lower San Joaquin
Study Area: Right Bank Calaveras River
River Section: Index Point D4

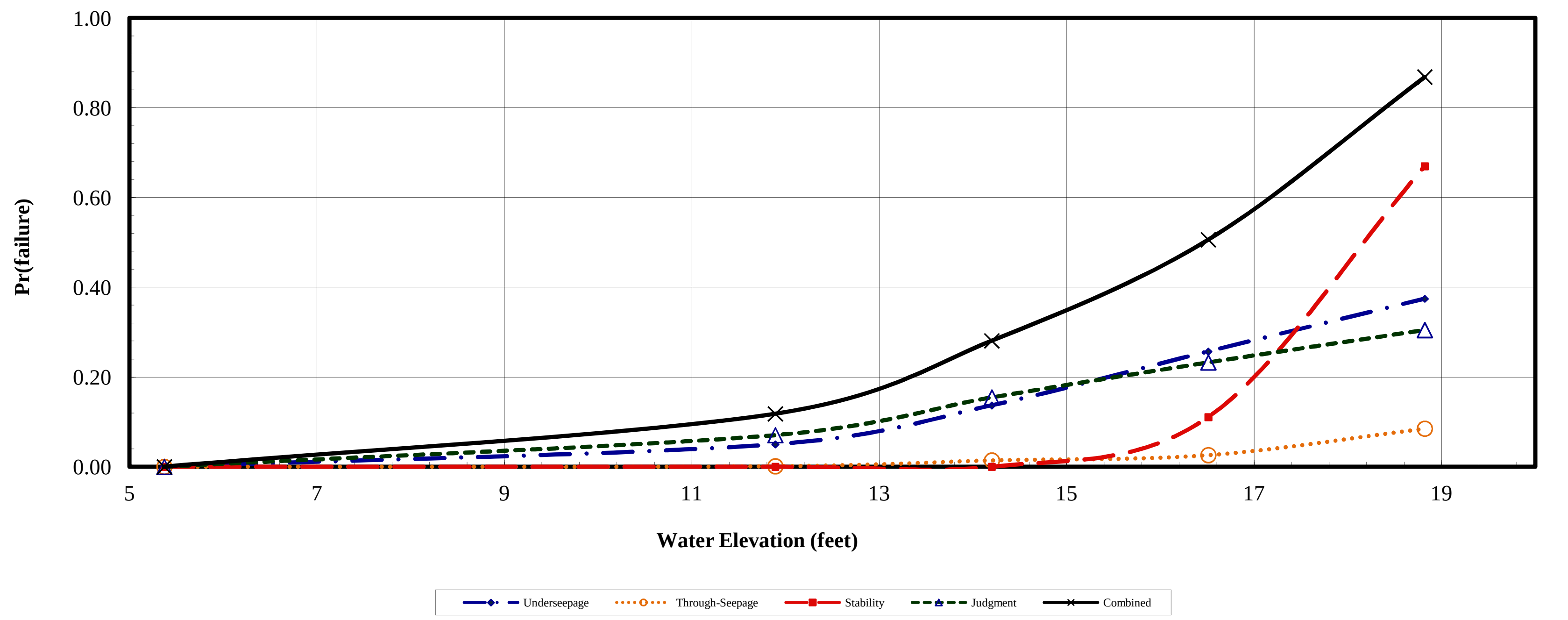
Levee Mile: STA 3092+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 18.82
L/S Toe Elev.: 5.37
W/S Toe Elev.: 3.18

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hog
Date: 9/25/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
5.37	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
11.89	0.0500	0.9500	0.0013	0.9987	0.0000	1.0000	0.0705	0.9295	0.1181	0.8819
14.20	0.1369	0.8631	0.0143	0.9857	0.0000	1.0000	0.1546	0.8454	0.2809	0.7191
16.51	0.2570	0.7430	0.0260	0.9740	0.1108	0.8892	0.2327	0.7673	0.5062	0.4938
18.82	0.3744	0.6256	0.0851	0.9149	0.6698	0.3302	0.3049	0.6951	0.8686	0.1314

Combined Probability of Poor Performance Curve - Index Point D4 LM STA 3092+00 Without Project Conditions



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

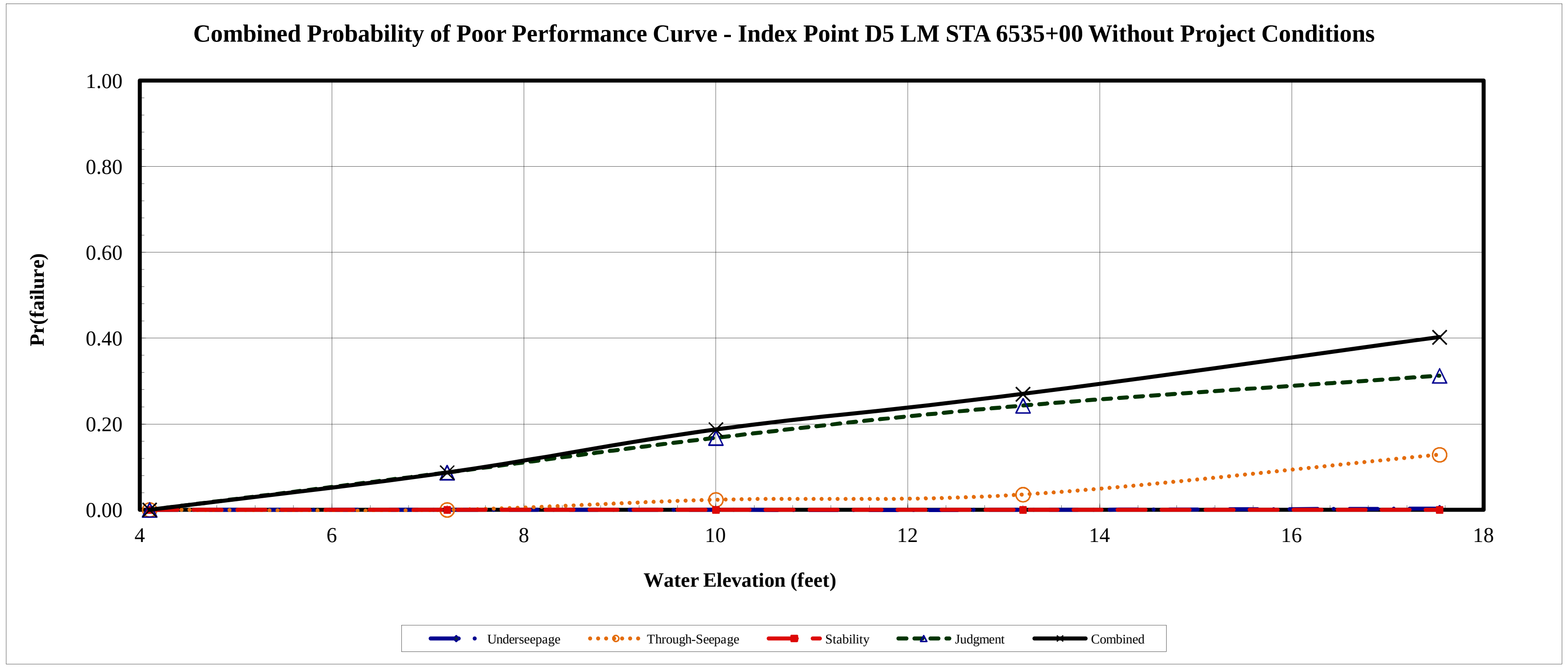
Project: Lower San Joaquin
Study Area: Left Bank Calaveras River
River Section: Index Point D5

Levee Mile: STA 6535+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 17.54
L/S Toe Elev.: 4.10
W/S Toe Elev.: -6.30

Analysis By: G. Johnson
Checked By: M. Perlea, J. Hog
Date: 9/19/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
4.10	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
7.20	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0869	0.9131	0.0869	0.9131
10.00	0.0000	1.0000	0.0235	0.9765	0.0000	1.0000	0.1677	0.8323	0.1872	0.8128
13.20	0.0001	0.9999	0.0356	0.9644	0.0000	1.0000	0.2427	0.7573	0.2698	0.7302
17.54	0.0028	0.9972	0.1284	0.8716	0.0000	1.0000	0.3124	0.6876	0.4023	0.5977


Combined Probability of Poor Performance Curve - Index Point D5 LM STA 6535+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

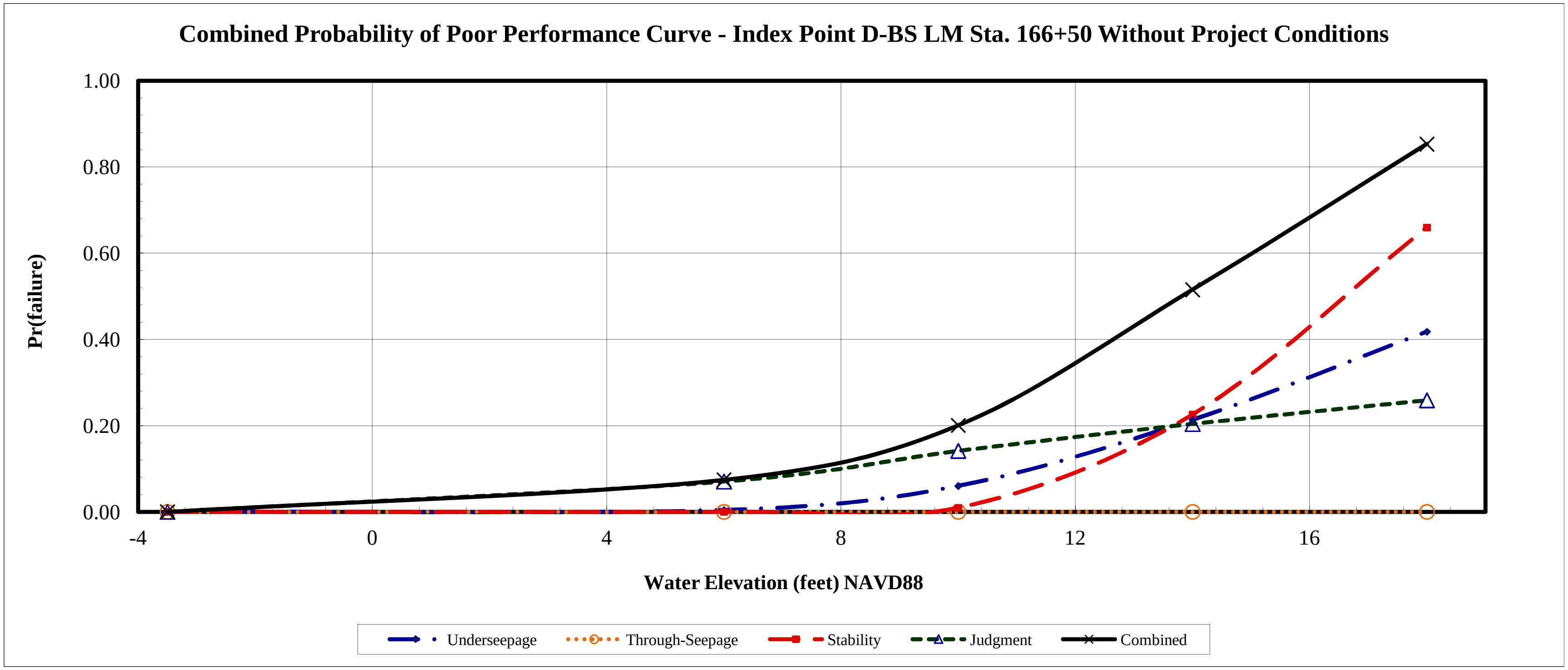
Project: Lower San Joaquin
Study Area: Delta Front Brookside Study Area
River Section: Index Point D-BS
Coordinates: State Plane (ft), N 2183200, E 6311320

Levee Mile: Sta. 166+50
River Mile: XXXX
Analysis Case: Without Project Conditions

Datum: NAVD 88
Crest Elev.: 18.00
L/S Toe Elev.: -3.50
W/S Toe Elev.: -7.50

Analysis By: G. Johnson
Checked By: J. Hogan, M. Per
Date: 3/14/2013

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
-3.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
6.00	0.0041	0.9959	0.0000	1.0000	0.0000	1.0000	0.0705	0.9295	0.0743	0.9257
10.00	0.0600	0.9400	0.0000	1.0000	0.0094	0.9906	0.1415	0.8585	0.2006	0.7994
14.00	0.2136	0.7864	0.0000	1.0000	0.2256	0.7744	0.2040	0.7960	0.5153	0.4847
18.00	0.4180	0.5820	0.0000	1.0000	0.6597	0.3403	0.2589	0.7411	0.8532	0.1468



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

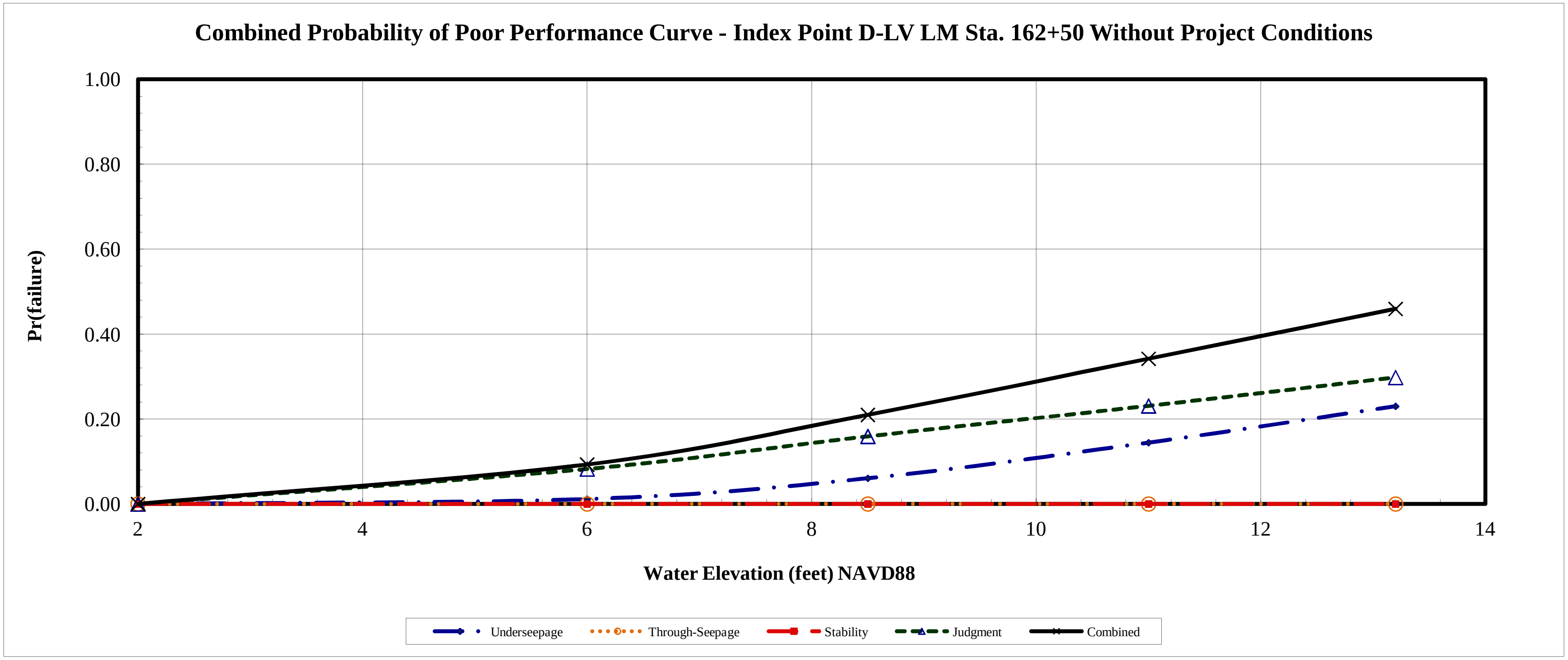
Project: Lower San Joaquin
Study Area: Delta Front Lincoln Village
River Section: Index Point D-LV
Coordinates: State Plane (ft), N 2185939, E 6315555

Levee Mile: Sta. 162+50
River Mile: XXXX
Analysis Case: Without Project Conditions

Datum: NAVD 88
Crest Elev.: 13.20
L/S Toe Elev.: 2.00
W/S Toe Elev.: 3.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perle
Date: 4/9/2013

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
2.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
6.00	0.0115	0.9885	0.0000	1.0000	0.0000	1.0000	0.0822	0.9178	0.0928	0.9072
8.50	0.0602	0.9398	0.0000	1.0000	0.0000	1.0000	0.1591	0.8409	0.2098	0.7902
11.00	0.1443	0.8557	0.0000	1.0000	0.0000	1.0000	0.2309	0.7691	0.3419	0.6581
13.20	0.2299	0.7701	0.0000	1.0000	0.0000	1.0000	0.2979	0.7021	0.4593	0.5407


Combined Probability of Poor Performance Curve - Index Point D-LV LM Sta. 162+50 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

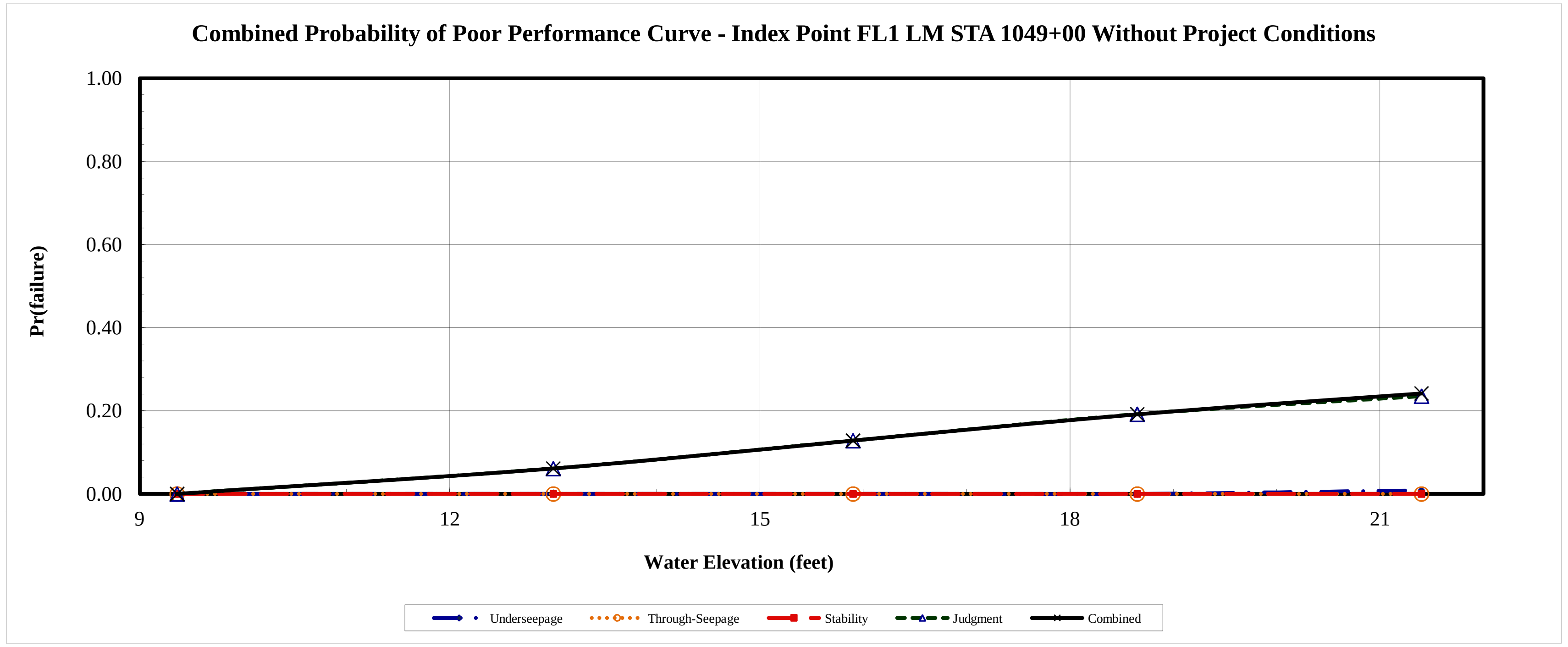
Project: Lower San Joaquin
Study Area: Left Bank French Camp Slough
River Section: Index Point FL1

Levee Mile: STA 1049+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 21.40
L/S Toe Elev.: 9.36
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/03/2012
Date: 11/28/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
9.36	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
13.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0610	0.9390	0.0610	0.9390
15.90	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1282	0.8718	0.1282	0.8718
18.65	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1917	0.8083	0.1917	0.8083
21.40	0.0087	0.9913	0.0000	1.0000	0.0000	1.0000	0.2351	0.7649	0.2418	0.7582


Combined Probability of Poor Performance Curve - Index Point FL1 LM STA 1049+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

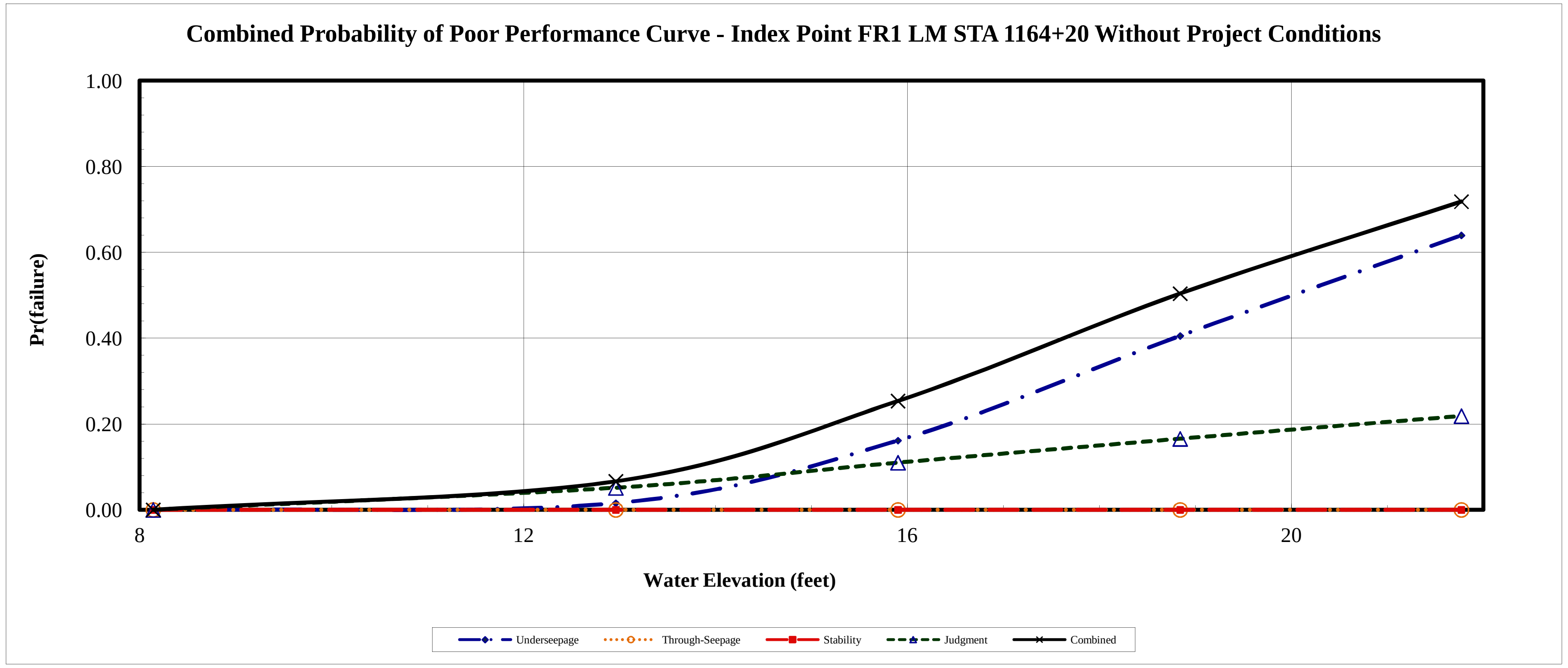
Project: Lower San Joaquin
Study Area: Right Bank French Camp Slough
River Section: Index Point FR1

Levee Mile: STA 1164+20
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 21.77
L/S Toe Elev.: 8.14
W/S Toe Elev.: 10.00

Analysis By: G. Johnson
Checked By: M. Perlea 12/12/2012
Date: 12/10/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
8.14	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
12.96	0.0157	0.9843	0.0000	1.0000	0.0000	1.0000	0.0514	0.9486	0.0663	0.9337
15.90	0.1615	0.8385	0.0000	1.0000	0.0000	1.0000	0.1099	0.8901	0.2537	0.7463
18.84	0.4054	0.5946	0.0000	1.0000	0.0000	1.0000	0.1656	0.8344	0.5039	0.4961
21.77	0.6396	0.3604	0.0000	1.0000	0.0000	1.0000	0.2185	0.7815	0.7183	0.2817


Combined Probability of Poor Performance Curve - Index Point FR1 LM STA 1164+20 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR1

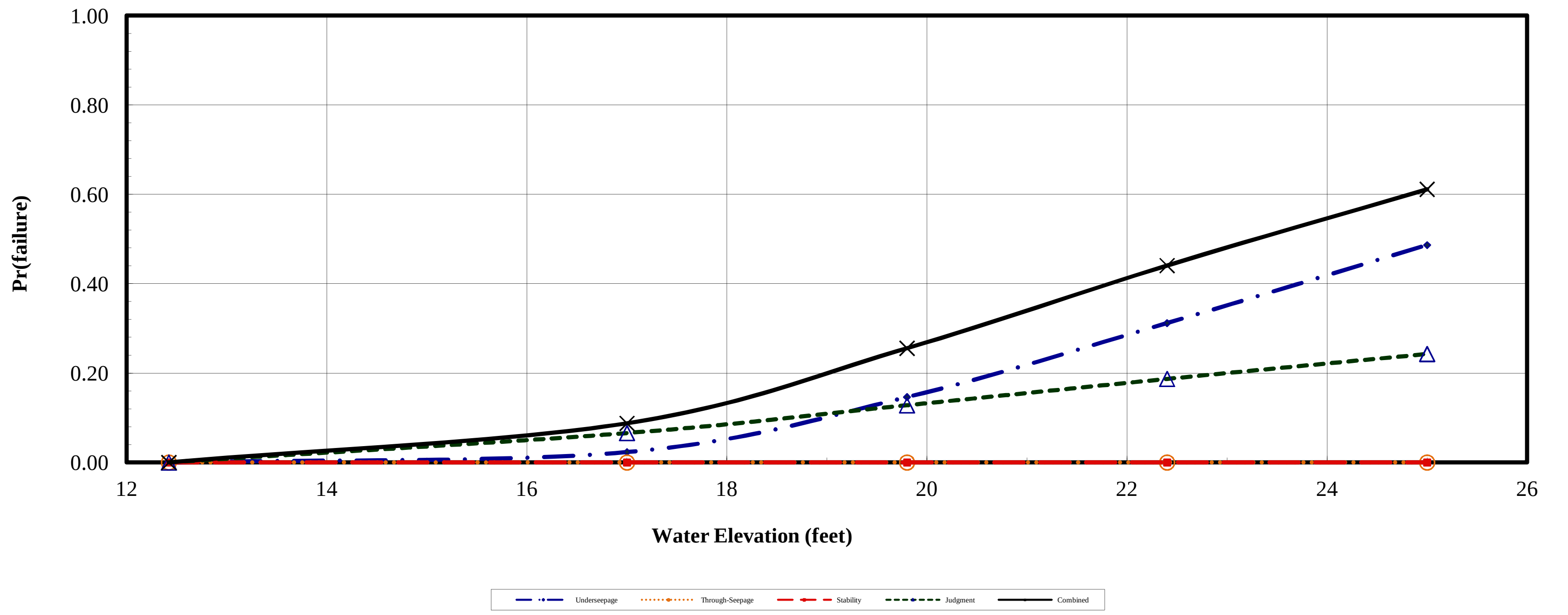
Levee Mile: 1292+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 25.00
L/S Toe Elev.: 12.42
W/S Toe Elev.: 11.00

Analysis By: G. Johnson
Checked By: J. Hogan, M. Per
Date: 12/18/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
12.42	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
17.00	0.0234	0.9766	0.0000	1.0000	0.0000	1.0000	0.0657	0.9343	0.0876	0.9124
19.80	0.1465	0.8535	0.0000	1.0000	0.0000	1.0000	0.1280	0.8720	0.2557	0.7443
22.40	0.3121	0.6879	0.0000	1.0000	0.0000	1.0000	0.1870	0.8130	0.4408	0.5592
25.00	0.4868	0.5132	0.0000	1.0000	0.0000	1.0000	0.2429	0.7571	0.6114	0.3886

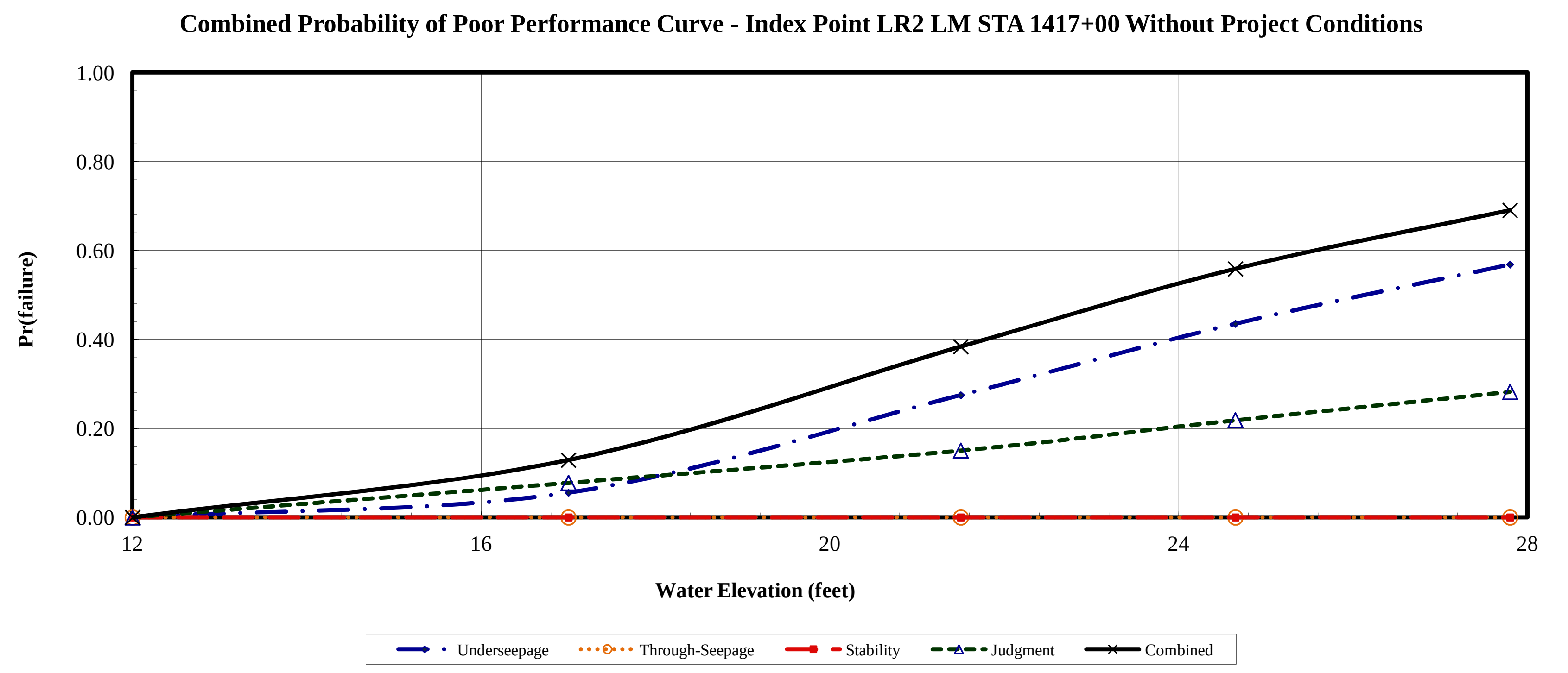
Combined Probability of Poor Performance Curve - Index Point LR1 LM 1292+00 Without Project Conditions



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

Project:	Lower San Joaquin	**Levee Mile:**	STA 1417+00	**Crest Elev.:**	27.80	**Analysis By:**	G. Johnson
Study Area:	Right Bank San Joaquin River	**River Mile:**	XX.XX	**L/S Toe Elev.:**	12.00	**Checked By:**	M. Perlea 12/03/2012
River Section:	Index Point LR2	**Analysis Case:**	Without Project Conditions	**W/S Toe Elev.:**	12.00	**Date:**	11/28/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
12.00	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
17.00	0.0555	0.9445	0.0000	1.0000	0.0000	1.0000	0.0775	0.9225	0.1287	0.8713
21.50	0.2749	0.7251	0.0000	1.0000	0.0000	1.0000	0.1503	0.8497	0.3839	0.6161
24.65	0.4353	0.5647	0.0000	1.0000	0.0000	1.0000	0.2185	0.7815	0.5587	0.4413
27.80	0.5685	0.4315	0.0000	1.0000	0.0000	1.0000	0.2823	0.7177	0.6903	0.3097



Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

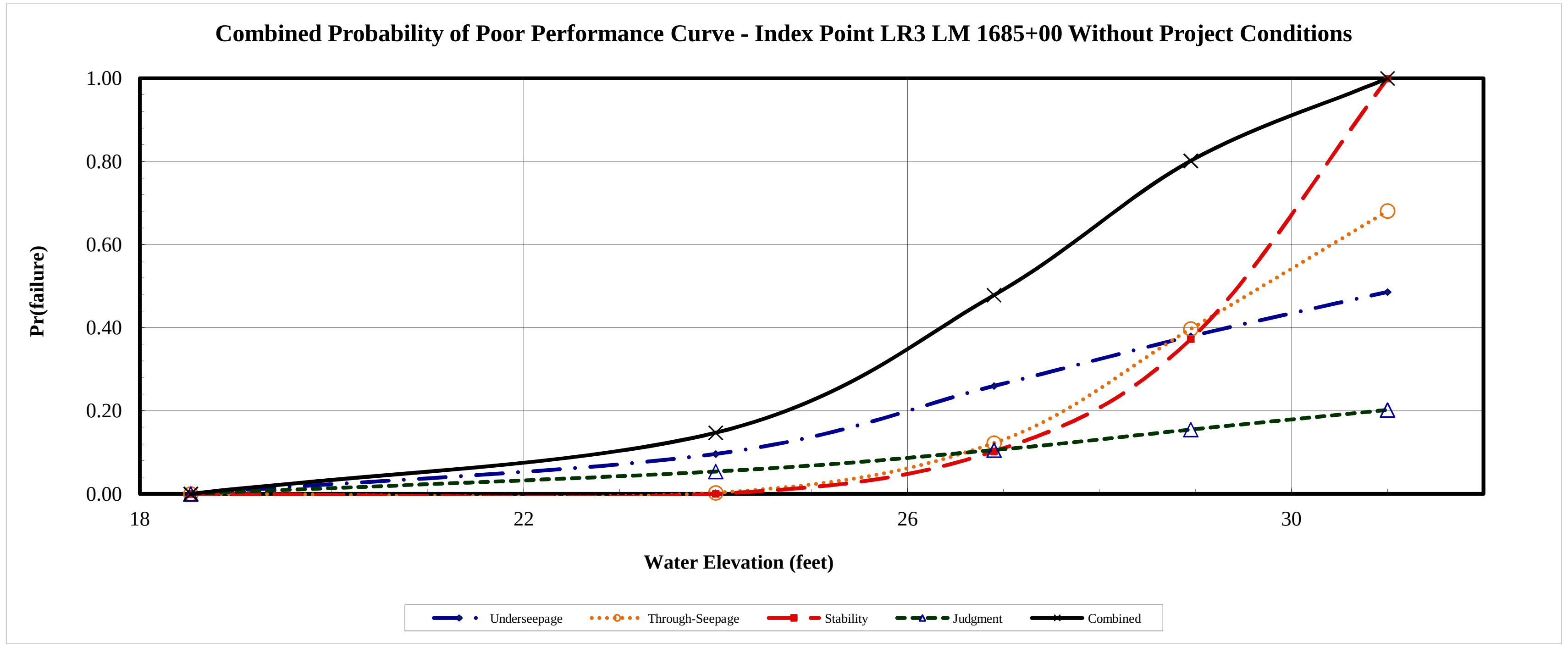
Project: Lower San Joaquin
Study Area: San Joaquin River
River Section: Index Point LR3

Levee Mile: 1685+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 31.00
L/S Toe Elev.: 18.53
W/S Toe Elev.: 17.80

Analysis By: G. Johnson
Checked By: J. Hogan, M. Perlea
Date: 12/19/2012

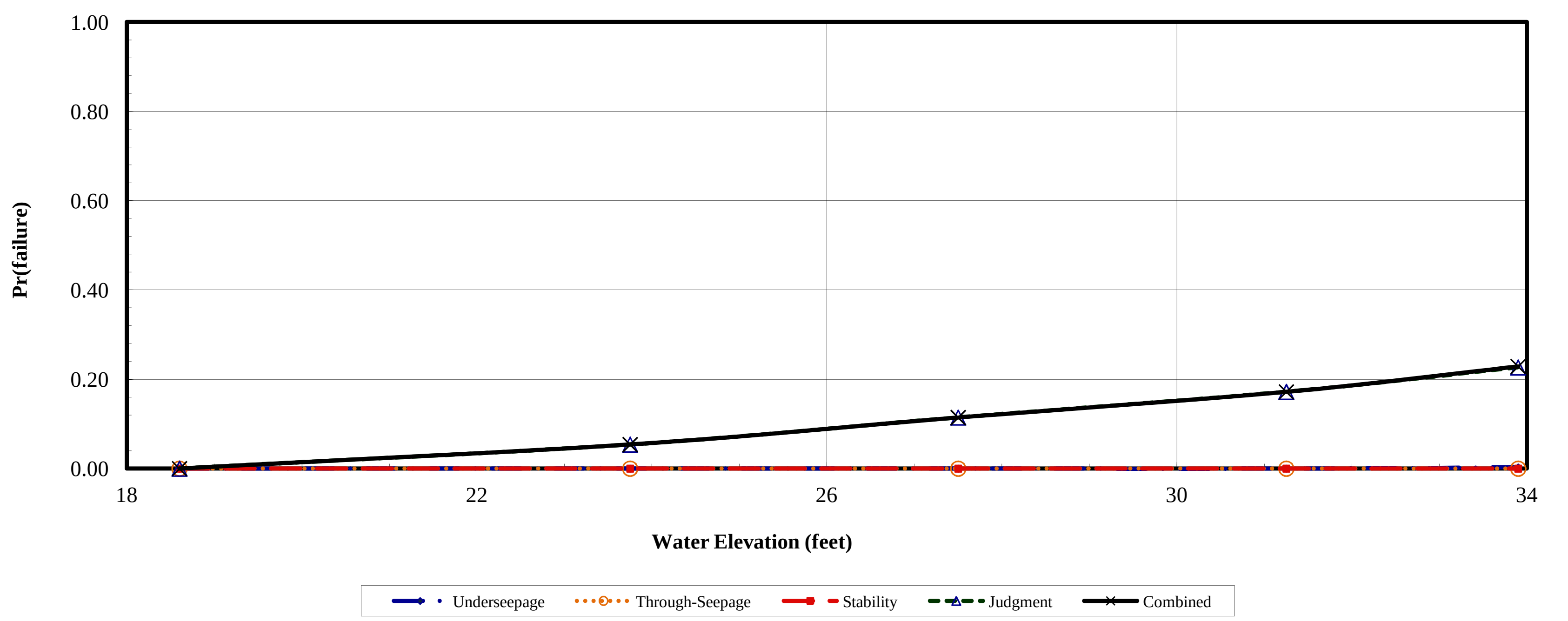
Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
18.53	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
24.00	0.0961	0.9039	0.0026	0.9974	0.0003	0.9997	0.0538	0.9462	0.1472	0.8528
26.90	0.2596	0.7404	0.1222	0.8778	0.1025	0.8975	0.1054	0.8946	0.4782	0.5218
28.95	0.3790	0.6210	0.3971	0.6029	0.3725	0.6275	0.1547	0.8453	0.8014	0.1986
31.00	0.4857	0.5143	0.6809	0.3191	0.9993	0.0007	0.2019	0.7981	0.9999	0.0001


Combined Probability of Poor Performance Curve - Index Point LR3 LM 1685+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

Project:	Lower San Joaquin	**Levee Mile:**	STA 1815+00	**Crest Elev.:**	33.90	**Analysis By:**	G. Johnson
Study Area:	Right Bank San Joaquin River	**River Mile:**	XX.XX	**L/S Toe Elev.:**	18.60	**Checked By:**	M. Perlea 12/13/2012
River Section:	Index Point LR4	**Analysis Case:**	Without Project Conditions	**W/S Toe Elev.:**	19.40	**Date:**	12/13/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
18.60	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
23.75	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0538	0.9462	0.0538	0.9462
27.50	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1144	0.8856	0.1144	0.8856
31.25	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.1719	0.8281	0.1719	0.8281
33.90	0.0030	0.9970	0.0000	1.0000	0.0001	0.9999	0.2265	0.7735	0.2289	0.7711



Combined Probability of Poor Performance Curve - Index Point LR4 LM STA 1815+00 Without Project Conditions

Geotechnical Risk and Uncertainty Analysis - Taylor Series Method
Combined Probability of Poor Performance Curve

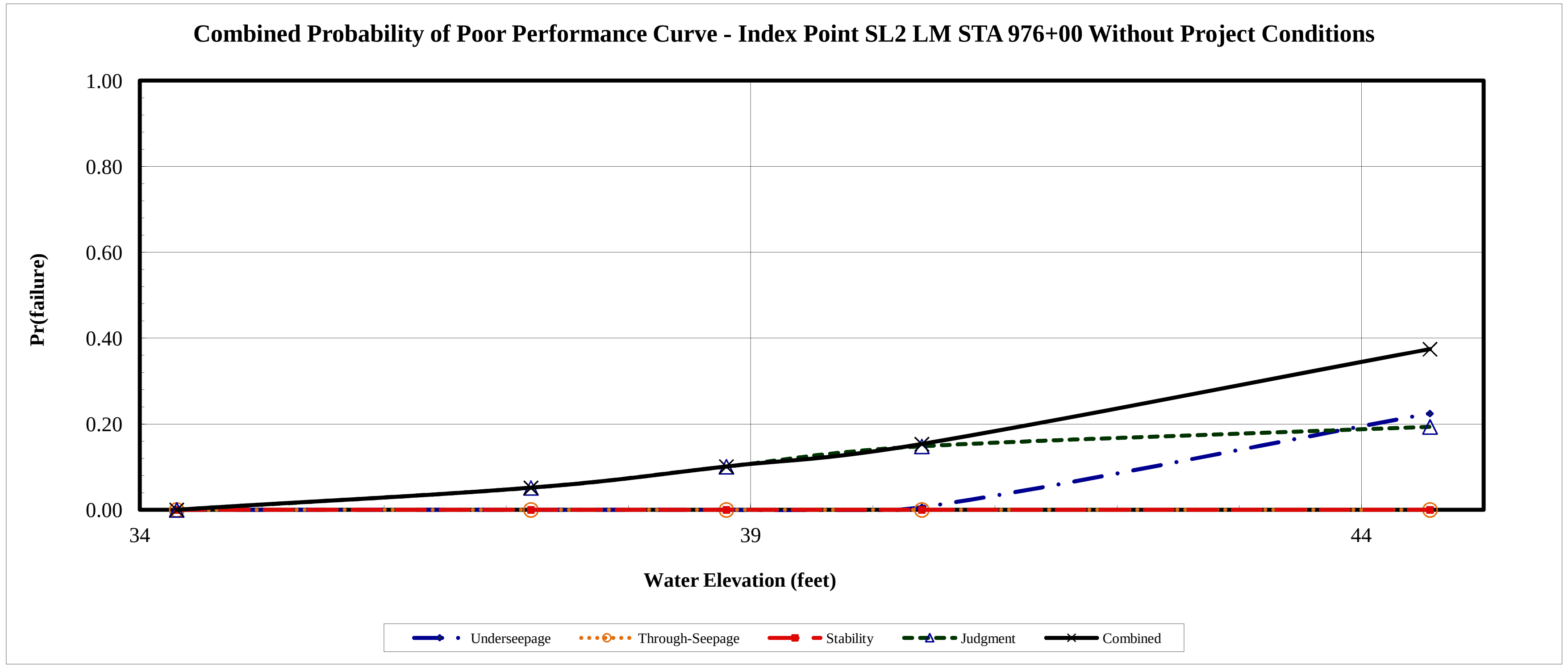
Project: Lower San Joaquin
Study Area: Left Bank Stockton Diverting Canal
River Section: Index Point SL2

Levee Mile: STA 976+00
River Mile: XX.XX
Analysis Case: Without Project Conditions

Crest Elev.: 44.56
L/S Toe Elev.: 34.30
W/S Toe Elev.: 34.79

Analysis By: J. Hogan
Checked By: M. Perlea, G. Joh
Date: 9/27/2012

Water Surface Elevation	Underseepage		Through-Seepage		Stability		Judgment		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
34.30	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000
37.20	0.0000	1.0000	0.0000	1.0000	0.0000	1.0000	0.0514	0.9486	0.0514	0.9486
38.80	0.0002	0.9998	0.0000	1.0000	0.0000	1.0000	0.1008	0.8992	0.1009	0.8991
40.40	0.0062	0.9938	0.0000	1.0000	0.0000	1.0000	0.1481	0.8519	0.1533	0.8467
44.56	0.2245	0.7755	0.0000	1.0000	0.0000	1.0000	0.1934	0.8066	0.3745	0.6255



US Army Corps of Engineers

Sacramento District

Lower San Joaquin River Feasibility Study

San Joaquin County, California

HYDROLOGY OFFICE SUMMARY REPORT

23 June 2014

This page left intentionally blank

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY
HYDROLOGY OFFICE REPORT, April 2014

Table of Contents

List of Tables

List of Figures and Plates

SEE NEXT PAGE

These additional plates are attached to the end of this Hydrologic Summary Report.

APPENDICES

1. **Upper Calaveras River watershed above Bellota**
2. **Upper Littlejohn Creek above Farmington, Ca**
3. **Bear Creek, Mosher Slough, Lower Calaveras River watershed below Bellota, and French Camp Slough**

This page was left Intentionally Blank

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

SAN JOAQUIN COUNTY, CALIFORNIA
HYDROLOGY OFFICE REPORT

February 2014

1.0 PURPOSE OF STUDY

The purpose of this hydrology report is to perform a hydrologic analysis of the lower San Joaquin River and tributaries that impact flooding in the Lathrop and Stockton urban areas. Due to the variety of watersheds in the study area, a number of methods were utilized for each watershed analysis.

The Lower San Joaquin River feasibility study will develop flood risk management (FRM) and ecosystem restoration (EC) plans along the Lower San Joaquin River, and the Bear Creek, Mosher Slough, Calaveras River and Mormon Slough, Littlejohn Creek, Duck Creek, and French Camp Slough. New Hogan Dam on the Calaveras River and Farmington Dam on Littlejohn Creek are both Corps owned and operated flood control projects that provide flood protection and water supply and recreation to the Stockton area. The authority for the U.S. Army Corps of Engineers (USACE or Corps) to study FRM and related water resources problems in the San Joaquin River Basin, including the study area in San Joaquin County, is provided in the Flood Control Act of 1962 (Public Law 87-874).

2.0. HOW TO NAVIGATE REPORT

Appendix 1 is the Calaveras River watershed above Bellota. Appendix 2 is the Littlejohn Creek above Farmington, Ca. Appendix 3 covers Bear Creek, Mosher Slough, lower Calaveras River watershed below Bellota, and French Camp Slough watershed below Farmington, Ca.

3.0. STUDY AREA

The study area from the Reconnaissance Report, Section 905(b) Analysis, for the LSJRFS is along the lower (northern) portion of the San Joaquin River system in the Central Valley of California. The San Joaquin River originates on the western slope of the, Sierra Nevada and emerges from the foothills at Friant Dam. The river flows west to the Central Valley, where it is joined by the Fresno, Chowchilla, Merced, Tuolumne, Stanislaus and Calaveras rivers, and smaller tributaries as it flows north to the Sacramento-San Joaquin Delta. The primary study area as described in the Section 905(b) Analysis includes the main stem of the San Joaquin River and its floodplains from the Mariposa Bypass downstream to the city of Stockton. This includes the distributor channels of the San Joaquin River in the southernmost reaches of the Delta: Paradise Cut and Old River as far north as Tracy Boulevard and Middle River as far north as Victoria Canal.

On the basis of continued coordination with local interests along the San Joaquin River, the primary study area for the LSJRFS will also include the Littlejohns Creek and Farmington Dam areas southeast of Stockton, the city of Stockton extending from the Calaveras River, Mormon Slough, and Bear Creek, and tributaries north of Stockton including the Lodi WWTP at Thornton Road and Interstate 5. An overview of the San Joaquin River Basin showing reservoirs and primary gaging station locations is included in plate 1.

The overall study area includes those areas adjacent to the primary study area which could be influenced by potential actions to address the identified problems and needs.

The study area was decreased in size to the area shown in plate 2 in 2011. The area south of the Stanislaus River confluence with the San Joaquin River was excluded because the Corps is prohibited from promoting development in floodplains which is the criteria on wise use of floodplains. Some of the area to the west of the San Joaquin River is part of the Sacramento – San Joaquin River Delta and overlaps the Delta Islands Feasibility study.

A map of the study area is shown in plate 2. Plate 3 shows the boundary of San Joaquin county. It shows that the entire study area is within the San Joaquin County boundary. Plate 4 shows the boundary of the San Joaquin Area Flood Control Agency (SJAFCA). The study area extends to the south to the Stanislaus River, to the east to Jack Tone Road, and outside the SJAFCA boundary north to the Lodi WWTP. The study area covers approximately 306 square miles and is approximately 15 miles east-west and 25 miles north-south. The study area includes the communities of Stockton, Manteca, Lathrop, Lockeford, and the census designated places (CDP) of Lincoln Village, French Camp, and parts of Lodi, and Ripon. Table 1 showing the population from the 2010-2000 US census is shown below. A plot of the San Joaquin County and City of Stockton population from 1960 to 2010 and projected population to 2070 is shown in plate 6.

Table 1. 2000 and 2010 Population and Projections

2010 - 2000 Census Population within study area			
Community	2010 Population	2000 Population	Change from 2000
French Camp, CDP	3,376	4,109	-17.8%
Lathrop	18,023	10,445	72.6%
Lincoln Village, CDP	4,381	4,216	3.9%
Lodi	62,134	56,999	9.0%
Manteca	67,096	49,258	36.2%
Ripon	14,297	10,146	40.9%
Stockton	291,707	243,771	19.7%
Unincorporated County	224,292	184,654	21.5%
San Joaquin County	685,306	563,598	21.6%
Source: US Census Bureau. CDP = Census Designated Place			

Table 2. Interim Projections For California and Counties

Interim Projections for California and Counties: July 1, 2015 to 2050 in 5-year Increments. Source: CA Dept of Finance, Demographics										
County	**Estimates**		**Projections**							
	2000	**2010**	**2015**	**2020**	**2025**	**2030**	**2035**	**2040**	**2045**	**2050**
San Joaquin	567,753	686,651	739,224	795,631	862,496	935,709	1,015,876	1,100,119	1,190,107	1,288,854

4.0. STUDY AREA BASINS – GENERAL DESCRIPTION

A list of the flood control dams and reservoirs above the Stockton metro area is shown in the table 10 below entitled “Dams and Lakes in the San Joaquin River Basin”.

Table 12 shows the drainage areas within the San Joaquin River basin. Flood control projects and principle control points are described below with the percentage of the total drainage area controlled. This table shows that there is approximately 56-percent of the basin controlled at Vernalis.

Flow frequency of New Hogan dam (NHG), the Bellota control point (MRS), and Farmington dam (FRM) and the at Farmington control point (FRG) were estimated by detailed study methods using gage records on the Calaveras River for New Hogan dam and Bellota, and on Littlejohn Creek for Farmington dam and at Farmington. Frequency curves and hydrographs of unregulated flow were developed for the 50% (1/2) ACE to 0.2% (1/200) ACE events. Additional details of the Calaveras River above Bellota and Littlejohn Creek above Farmington control points may be found in the Calaveras River and Littlejohn Creek frequency analysis and hydrographs by David Ford Consulting Engineers (Ford) in June 2011 for the Lower San Joaquin River Feasibility Study [6 & 7].

Flow frequency for stream reaches downstream of the Bellota control point on the Calaveras River, and below the Farmington control point on Littlejohn Creek were developed by detailed methods using an HEC-HMS rainfall-runoff model calibrated to specific flood events. That includes the Mormon Slough which is tributary to the Calaveras River. And, the HEC-HMS model of the Littlejohn Creek watershed also includes, Duck Creek, Lone Tree Creek, and French Camp Slough. HEC-HMS models were also developed for Bear Creek and Mosher Slough watersheds, which are unregulated watersheds, and are tributary to the Delta. Additional details of the Calaveras River below Bellota and Littlejohn Creek below Farmington control points may be found in the F3 Hydrology Appendix for the Lower San Joaquin River Feasibility Study done by Peterson-Brustad, Inc Consulting Engineers (PBI) as work-in-kind for the San Joaquin Area Flood Control Agency (SJAFCA).

4.1. Bear Creek HEC-HMS Modeling General

Bear Creek is located near the city of Stockton in San Joaquin County, California plates 29 and 30 (Figure 3-2 and 3-12). The watershed runs east from the city of Stockton into the Sierra Nevada foothills in Calaveras County and includes a total area of approximately 115 square miles. The uppermost portion of the watershed achieves maximum elevations of 1,000 feet and is not subject to snowmelt. It then descends through moderate slopes to the lower portion of the watershed at sea-level. The HEC-HMS model described in this memorandum has an outlet on Bear Creek at Disappointment Slough and includes Bear Creek, Upper Mosher Creek, Paddy Creek and Pixley Slough. See figure 3-12 for subbasins and index points.

4.2. Mosher Slough HEC-HMS Modeling General

Mosher Slough is located near the city of Stockton in San Joaquin County, California (Figure 2-1). The majority of the watershed is located in the urbanized area of Stockton between Interstate-5 and Highway 99 with the watershed area totaling approximately 16 square miles. The watershed's terrain has moderate slopes and reaches a maximum elevation of 65 feet above the modeled outlet at the confluence of Mosher Slough and Bear Creek just west of Interstate-5.

The HEC-HMS model described in this report includes only the lower portion of Mosher Slough which begins immediately below the diversion that routes the entirety of Upper Mosher Creek to Bear Creek (see plate 31, Figure 4- 2). The hydrology for Upper Mosher Creek is included in the Bear Creek HEC-HMS model as described in Section 3.0 of the LSJRFS Hydrology Report. See plate 32 (figure 4-10) for subbasins and index points.

4.3 Calaveras River HEC-HMS Modeling General

The Calaveras River watershed is located near the city of Stockton in San Joaquin County, California (Plates 33 and 34, Figure 5-2 and 5-12). The watershed runs east from the city of Stockton into the Sierra Nevada foothills in Calaveras County. The Calaveras River watershed can be split into two sections: above New Hogan Dam and below New Hogan Dam. The PBI - F3 Hydrology Appendix [4] focuses on the section of the Calaveras River below the dam whereas the section above the dam is part of a separate reservoir operations study [6].

The watershed includes a total area of 597 square miles with 352 square miles of this tributary area flowing into New Hogan Reservoir. The watershed discussed in this TM (below New Hogan Reservoir) includes the remaining 245 square miles and achieves maximum elevations of 1,500 feet. It then descends through moderate slopes to the lower portion of the watershed which lies at sea-level. Flow in the stream system is largely affected by releases from New Hogan Reservoir. The entire watershed is low enough in elevation to be rainfall dominant. The HEC-HMS model described in this memorandum includes the Calaveras River, Cosgrove Creek, Mormon Slough, Potter Creek, and the Stockton Diverting Canal systems and discharges to the San Joaquin River to the west of Interstate-5. See plate 34 (figure 5-12) for subbasins and index points.

4.3.1. General Characteristics of the Calaveras River Basin

The area associated with operation of the New Hogan Lake Project is basically the entire Calaveras River Basin, including its distributary channels, flood plain, and service area. The following information is taken from the New Hogan Water Control Manual, USACE, 1983).

The Calaveras River Basin above New Hogan Dam is relatively low-lying, consisting of 363 square miles on the western slope of the Sierra Nevada in Calaveras County, California. The basin is fan-shaped in plan, with the principal tributaries. Esparanza Creek and Jesus Maria Creek, which together form the North Fork of the Calaveras; and Calaveritas Creek, San Antonio Creek, and San Domingo Creek which form the South Fork. The North and South Forks join about 7 miles above the dam, within the limits of the reservoir.

Below New Hogan Dam, the Calaveras flows westerly to emerge from the foothills at Bellota, where the channel divides into two branches. A control structure provides for diversion of water when desired into the old Calaveras River channel, which is narrow and overgrown with dense vegetation. Otherwise flows enter Mormon Slough which was enlarged in the late 1960's to convey 12,500 cubic feet per second. Mormon Slough extends 13 miles southwesterly across the valley floor to the Stockton Diverting Canal, which continues northerly on the east side of Stockton to rejoin the Calaveras channel. From there, the Calaveras extends westerly through the City of Stockton to the San Joaquin River on the west side of Stockton. A General Map of the basin is presented on Plate 5 (reference plate 2) and plate 33 (figure 5-2).

4.3.2. Climate

Climate in the Calaveras River basin is characterized by cool, wet winters and hot, dry summers. Temperatures on the valley floor normally range from a winter low of about 30°F to a summer high of about 105°F and are typical of the entire basin except for the extreme upper elevations.

Normal annual precipitation (NAP) for the watershed above New Hogan Dam is 33.3 inches, and ranges from about 24 inches at New Hogan Dam to nearly 50 inches in the upper basin. In dry years, annual basin precipitation can amount to less than 11 inches and in wet years more than 40 inches. Plate 22 (reference plate 12) shows isohyetal lines of NAP over the basin.

More than 90 percent of the annual precipitation occurs from November through April. Winter storms, which account for the greatest share of annual basin precipitation, originate over the Pacific Ocean and are associated with frontal systems containing masses of moist air mov1ng inland against mountain barriers. Precipitation usually occurs as rain below 4,000 feet elevation. Above 4,000 feet, precipitation may occur as snow, although winter storms often bring rain above 4,000 feet. Intensities are moderate, but rain generally continues for three or four days and is often followed by additional storm fronts. As much as half of the normal annual precipitation may fall in a single storm period.

Precipitation during summer is from thunderstorms and is mainly confined to relatively small areas at higher elevations.

Average monthly precipitation for three representative stations are shown on Table 3.

Table 3. Precipitation Data at Selected Stations

	Average Monthly Precipitation					
	Stockton WSO Airport		Camp Pardee		Calaveras Big Trees	
Month	Inches	%	Inches	%	Inches	%
July	0.01	0.1%	0.01	0.0%	0.06	0.1%
August	0.03	0.2%	0.04	0.2%	0.13	0.2%
September	0.17	1.2%	0.18	0.9%	0.51	0.9%
October	0.72	5.1%	1.15	5.5%	2.78	5.0%
November	1.72	12.1%	2.80	13.4%	6.79	12.3%
December	2.68	18.9%	3.50	16.8%	10.17	18.4%
January	2.91	20.5%	3.85	18.5%	10.60	19.1%
February	2.11	14.9%	2.91	14.0%	8.24	14.9%
March	1.96	13.8%	3.17	15.2%	7.99	14.4%
April	1.37	9.7%	2.25	10.8%	5.25	9.5%
May	0.42	3.0%	0.80	3.8%	2.22	4.0%
June	0.07	0.5%	0.20	1.0%	0.64	1.2%
Total	14.17	100.0%	20.86	100.0%	55.38	100.0%
Nov - Apr	12.75	90.0%	18.48	88.6%	49.04	88.6%
Years of Record	27		49		35	
Elevation (feet, msl)	22		658		4695	
Basin Mean NAP	33.0 inches					
Source: NOAA NWS 1941-70						

5.0. FRENCH CAMP SLOUGH HEC-HMS MODELING GENERAL

The French Camp Slough watershed is located near the city of Stockton in San Joaquin County, California (Plates 35 and 36, Figure 6-1 and 6-2). The watershed runs east from the city of Stockton into the Sierra Nevada foothills in Calaveras County. It achieves maximum elevations of 2,100 feet and includes a total area of 430 square miles. It then descends through moderate slopes to the lower portion of the watershed which lies at sea-level. None of the watershed experiences snowfall; all floods are rainfall-induced.

The HEC-HMS model described in this memorandum includes the Duck Creek, Lone Tree Creek, Temple Creek, Rock Creek, Webb Creek, Littlejohn Creek, and the French Camp Slough systems and discharges to the San Joaquin River to the west of Interstate-5. See plate 36 (figure 6-11) for subbasins and index points.

5.1. Littlejohn Creek Watershed Characteristics

The following information is taken from the Farmington Dam Water Control Manual, USACE, 2004.

5.1.1 General Characteristics.

The basin encompassing the Littlejohn Creek Stream Group – bounded on the north and south by the Calaveras and Stanislaus river basins, respectively – is about 15 miles (24.1 km) wide from north to south and 40 miles (64.4 km) long from east to west. Runoff from its approximately 415 square mile drainage area flows westward to the San Joaquin River via French Camp Slough. Of the many creeks comprising the Littlejohn Creek Stream Group, three are considered major: Littlejohn, Duck, and Lone Tree, and of these, Littlejohn is the principal stream system.

Above Farmington Dam, the watershed portion of the project is a wing-shaped area extending 20 miles (32.0 km) upstream into the foothills on the western slope of the Sierra Nevada. Principal streams contributing to the reservoir are Littlejohn, Rock and Hoods creeks. These streams drain a combined area of 212 square miles at the dam. Above the diversion structure, across Duck Creek, the drainage area is 28 square miles. Basin features are shown on the General Map, plates 28, 35 and 36 (figures 2-1, 6-2 and 6-11).

Vegetative cover varies within the basin. Above Farmington Dam, the steep hillsides in the upper basin are sparsely covered by deciduous brush, small stands of trees, and a grassland understory. A discontinuous bank of riparian growth stretches through much of the upper basin. Along portions of Rock and Littlejohn creeks, the banks are completely devoid of riparian vegetation and badly eroded. The existing riparian vegetation is primarily valley oak, Fremont cottonwood, willow and white alder. Shrubs include willow, elderberry, and coyote brush. Annual grassland, such as grasses and forbs, is the predominant vegetation type within the reservoir area. Below Farmington Dam, the lower basin consists primarily of intensely developed agricultural lands and unimproved pastureland. Along lower basin stream channels, native vegetation has diminished, with some light brush and a few scattered oaks remaining.

5.1.2. Climate

a. General. The climate of the Littlejohn Creek Basin is classified as dry and sub-humid, characterized by two well-defined seasons: long, hot dry summers with very little rain, and short, mild wet winters with frequent rain but very little snow. The location of climatological stations and normal annual precipitation isohyets are shown on plates 24 and 26 (Plate 4-5.1 and 4-5.2).

b. Temperature. Average temperatures within the basin range between 45°F and 77°F, with a yearly average of 61.5°F. Summer highs can reach 115°F and winter lows can drop to near freezing. At Stockton, extreme temperatures have ranged from 114°F during the summer to 16°F during the winter months.

c. <u>Precipitation.</u> Normal annual precipitation (NAP) varies throughout the Littlejohn Creek drainage area, ranging from 12 inches on the valley floor to about 30 inches in the higher areas as shown on plates 24 and 26 (Plate 4-5.1 and 4-5.2). Normal annual precipitation above Farmington Dam is about 17 inches, while downstream it is about 14 inches. The mean monthly and annual distribution of precipitation at selected stations is given in Table 4.

TABLE 4

MEAN MONTHLY PRECIPITATION

MONTH	STOCKTON WSO AIRPORT [+] (Elev 22')		KNIGHTS FERRY 2ESE [‡] (Elev 315')		COPPEROPOLIS [‡] (Elev 970')		FLOWERS MOUNTAIN (Elev 1480')	
	in	%	in	%	in	%	in	%
Jan	2.85	20.4	2.88	16.9	4.52	19.4	4.07	19.2
Feb	2.27	16.3	2.55	15.0	4.08	17.6	3.99	18.8
Mar	2.04	14.6	2.49	14.6	3.83	16.5	3.51	16.5
Apr	1.13	8.1	1.74	10.2	1.80	7.7	1.60	7.5
May	0.41	2.9	0.39	2.3	0.46	2.0	0.82	3.9
Jun	0.08	0.6	0.15	0.9	0.19	0.8	0.21	1.0
Jul	0.03	0.2	0.10	0.6	0.06	0.3	0.09	0.4
Aug	0.04	0.3	0.15	0.9	0.08	0.3	0.08	0.4
Sep	0.28	2.0	0.29	1.7	0.31	1.3	0.18	0.9
Oct	0.69	5.0	0.96	5.6	1.06	4.6	1.29	6.1
Nov	1.81	13.0	2.65	15.5	3.20	13.8	2.53	11.9
Dec	2.31	16.6	2.69	15.8	3.66	15.7	2.85	13.4
Average Annual	13.94	100.0	17.04	100.0	23.25	100.0	21.22	100.0
Nov-Mar	11.28	80.9	13.26	77.8	19.29	83.0	16.95	79.5
Source:	NOAA 1941-2004		NOAA 1960-1972 1974-1976		USACE 1955-1995		USACE 1972-2003	

[+] Climatological Data Summary. Monthly Average Temperatures (updated June 2004) retrieved 12 July 2004 from Western Regional Climate Center, Desert Research Institute Web site: <http://www.wrcc.dri.edu/> [‡]Gage discontinued.

About 80 percent of the precipitation runoff occurs during the months of November through March. Snow rarely falls on the area and is not a significant factor in runoff from large storms.

6.0. DESIGN STORMS

Except for Bear Creek (storm balanced to multiple durations), design storms for hydrologic analysis of the Mosher Slough, Calaveras River below Bellota, and Littlejohn and French Camp system below the town of Farmington were created using 72-hour duration NOAA14 depths and areal reduction for the 1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, and 1/500 AEP events as input to the LSJRFS HEC-HMS models. As discussed in Section 6.3, the 72-hour storm pattern provides a storm event that is high in both peak flow and volume which is important for levee breach scenarios.

6.1. Rainfall Zones

LSJRFS subbasins were aggregated into seven rainfall zones with uniform rainfall characteristics. Seven rainfall gages were selected to form the basis of this subbasin aggregation. The selected gages are distributed throughout the study area and have available rainfall data at short-interval timesteps which can be used for storm patterning (see Section 6.3).

GIS software was used to draw Thiessen polygons around the selected rainfall gages and subbasins lying within each Thiessen polygon were aggregated to create the rainfall zones Plate 28 (Plate 2-1).

6.2. Design Storm Depths

The National Oceanic and Atmospheric Administration (NOAA) published its Atlas 14 Precipitation Frequency Study for California1 in April 2011 (NOAA, 2011) which includes estimates for design rainfall depths in an ASCII grid file format for use in GIS. A shapefile with seven defined rainfall zone boundaries was projected on top of the NOAA14 ASCII grid files to calculate average point rainfall depths within each rainfall zone for 96 different frequency-duration combinations.

The output from the NOAA14 GIS data acquisition process includes depth-duration-frequency tables for each rainfall zone. These depth-duration-frequency tables are included for each watershed in their respective attachments.

6.3. Design Storm Pattern

The design storm pattern used for the LSJRFS is based on an observed storm event that was recorded at various rainfall gages within the study area.

The December 31, 1996-January 3, 1997 rainfall event (1997 Event) and the April 2, 2006-April 5, 2006 rainfall event (2006 Event) were considered for the basis of design storm patterning. These events represent two of the largest storms in recent history.

Data records were checked for these events at all known precipitation gages within the vicinity of the study area. Some gages only had recorded data at monthly or daily intervals and were excluded from the gage selection process based on their inadequate time step. Other gages were excluded due to lack of data for the specific dates listed; many of the available rainfall gages did not contain data for the 2006 Event.

The 1997 Event is often considered an industry standard for rainfall events and was ultimately selected as the pattern used to temporally distribute the design storms. The storm temporal pattern is shown below in figure 5.1.

Data from the New Hogan (NHG) gage location represents a typical 72-hour hyetograph pattern for the 1997 Event and is shown below.

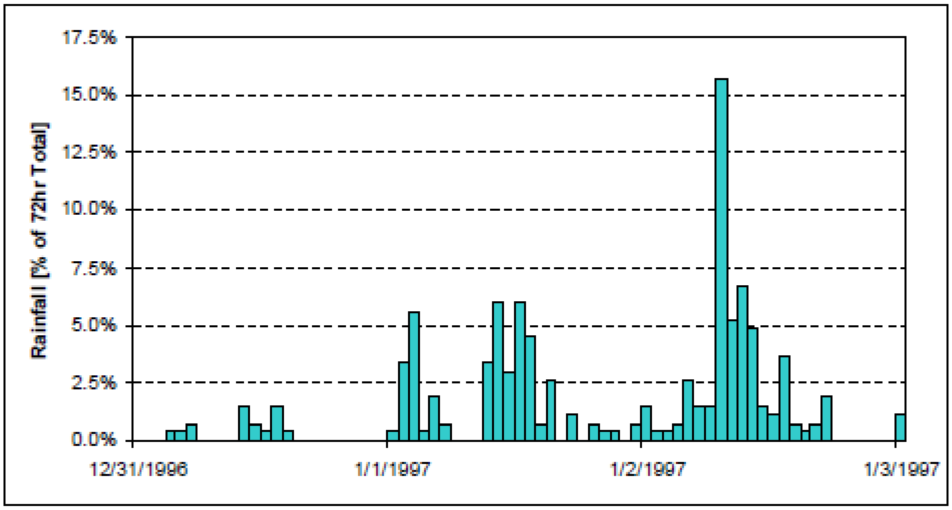


Figure 5.1. Typical Rainfall Pattern for the 1997 Event.

The 72-hour storm pattern provides a storm event that is high in volume which is important for levee breach scenarios. For the LSJRFS, it is also desirable to preserve the high peak flows that would result from a standard, 24-hour design storm. Therefore, additional analyses were conducted for Mosher and French Camp Sloughs to run a SCS Type 1 storm, an industry standard 24-hour event, to confirm that the peak flows resulting from either type storm were comparable. For the lower Calaveras River watershed below Bellota, a 97 pattern balanced to 1-, 3-, 6-, 12-, 24-, 48-, and 72 hour NOAA14 depths and areal reduction factors was compared to the 97 pattern balanced only to a 72-hour depth and one areal reduction factor. The results were highly comparable in volume and peak (see Appendix 2).

All flows were comparable except for those in the Bear Creek watershed. To correct this, Bear Creek hyetographs were balanced to the 3-, 6-, 12-, 24-, 48-, and 72-hour NOAA14 storm depths. After balancing the hyetographs, Bear Creek models produced high-volume hydrographs with peak flows that are comparable to those resulting from a standard 24-hour design storm.

6.4. Storm Centering Approach

The LSJRFS utilizes a storm centering approach to consider depth area reduction of design storms falling over the study area. This area reduction is typically disregarded for small watersheds where one point precipitation depth can be applied to the entire tributary area, however given the size of the watersheds in the LSJRFS it is necessary to apply area reduction factors to the point rainfall design storm depths.

Area reduction factors were calculated using a procedure that was developed by the USACE Sacramento District for the hydrology of their Downtown Guadalupe River Project in November 2009 [9]. This procedure takes into account various storm centerings by ranking the rainfall zones according to their distance from the storm centering location and determining the cumulative drainage area for each location in the watershed. HMR 59 was source of factors.

7. EXISTING CONDITIONS

Existing conditions are those at the time the study is conducted and form the basis for extrapolations to other conditions. Existing conditions within the study area are discussed below.

7.1 Flow Frequency Estimates

Flood waters potentially threatening the study area originate from several sources.

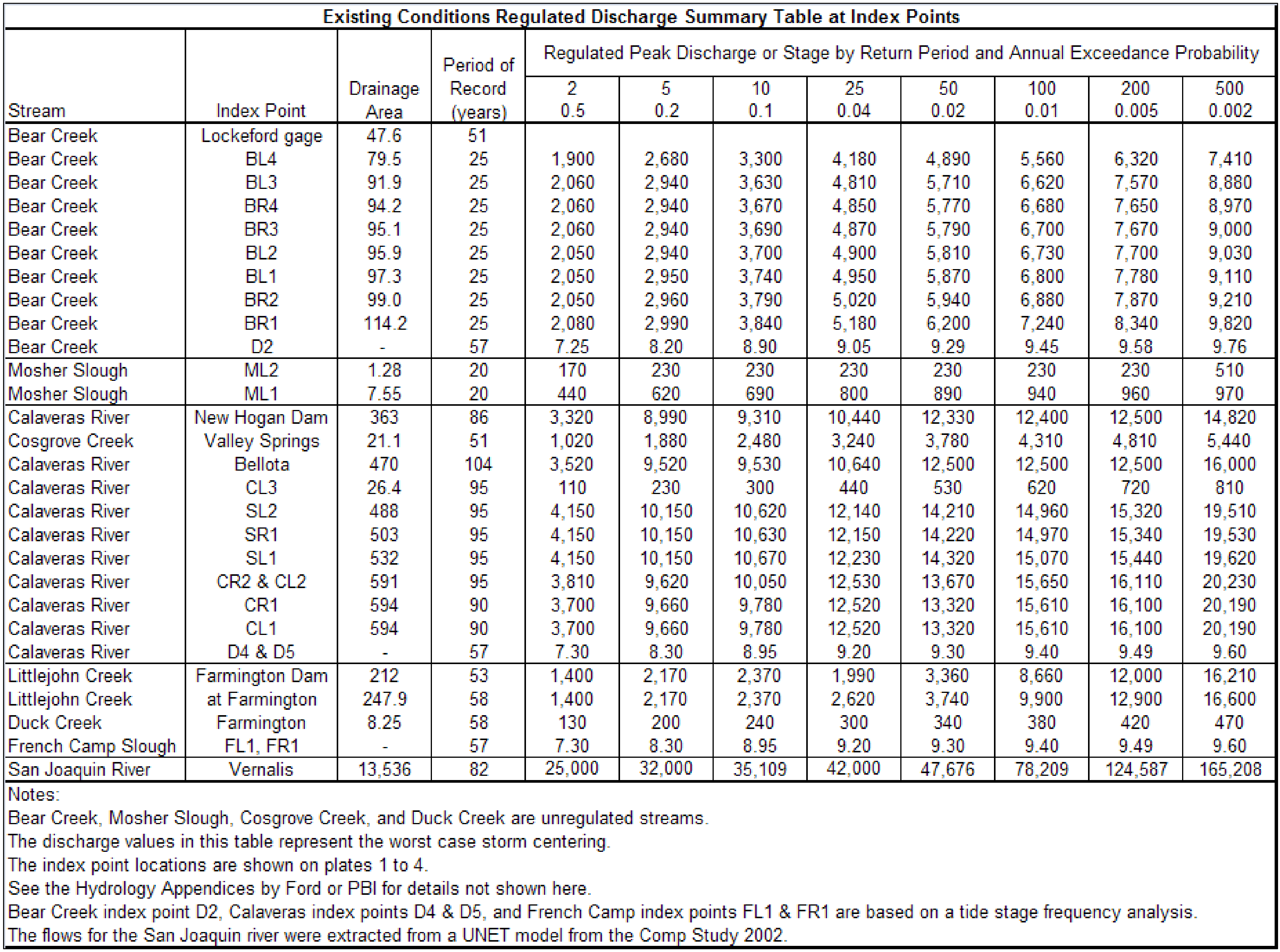
Those sources include:

- The San Joaquin river mainstem (flood control projects are shown in table 10 below);
- The east side tributaries including:
    - Bear Creek,
    - Mosher Slough,
    - Calaveras River and Mormon Slough,
    - Littlejohn Creek, Duck Creek, and French Camp Slough;
- The Sacramento-San Joaquin Delta, including the Sacramento, San Joaquin, Cosumnes and Mokelumne Rivers, and ocean tides.

The discharges by index point for annual exceedance probabilities of 0.5 (1/2) to 0.002 (1/500) are shown in table 5 below. Plates 30, 32, 34, and 36 (figures 3-12, 4-10, 5-12, and 6-11), at the end of this memo, show the location of the index points.

The existing and future without project conditions are considered the same. In addition, the future with project condition is essentially the same as the existing without project condition. Therefore, the table of existing conditions flow values will be used for all conditions.

Table 5. Existing Conditions Regulated Flows (CFS)

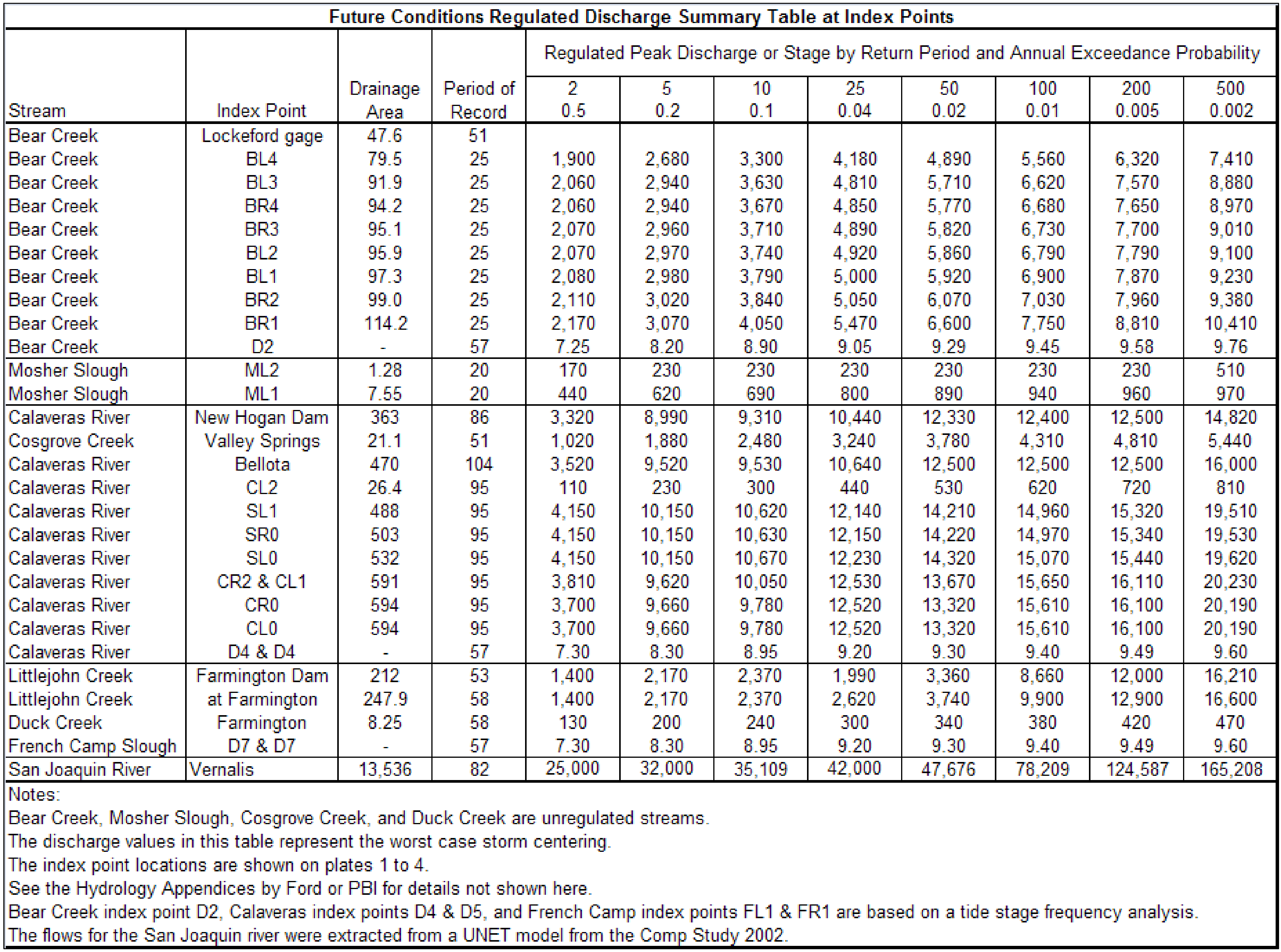


Existing Conditions Regulated Discharge Summary Table at Index Points

				Regulated Peak Discharge or Stage by Return Period and Annual Exceedance Probability							
Stream	Index Point	Drainage Area	Period of Record (years)	2 0.5	5 0.2	10 0.1	25 0.04	50 0.02	100 0.01	200 0.005	500 0.002
Bear Creek	Lockeford gage	47.6	51								
Bear Creek	BL4	79.5	25	1,900	2,680	3,300	4,180	4,890	5,560	6,320	7,410
Bear Creek	BL3	91.9	25	2,060	2,940	3,630	4,810	5,710	6,620	7,570	8,880
Bear Creek	BR4	94.2	25	2,060	2,940	3,670	4,850	5,770	6,680	7,650	8,970
Bear Creek	BR3	95.1	25	2,060	2,940	3,690	4,870	5,790	6,700	7,670	9,000
Bear Creek	BL2	95.9	25	2,050	2,940	3,700	4,900	5,810	6,730	7,700	9,030
Bear Creek	BL1	97.3	25	2,050	2,950	3,740	4,950	5,870	6,800	7,780	9,110
Bear Creek	BR2	99.0	25	2,050	2,960	3,790	5,020	5,940	6,880	7,870	9,210
Bear Creek	BR1	114.2	25	2,080	2,990	3,840	5,180	6,200	7,240	8,340	9,820
Bear Creek	D2	-	57	7.25	8.20	8.90	9.05	9.29	9.45	9.58	9.76
Mosher Slough	ML2	1.28	20	170	230	230	230	230	230	230	510
Mosher Slough	ML1	7.55	20	440	620	690	800	890	940	960	970
Calaveras River	New Hogan Dam	363	86	3,320	8,990	9,310	10,440	12,330	12,400	12,500	14,820
Cosgrove Creek	Valley Springs	21.1	51	1,020	1,880	2,480	3,240	3,780	4,310	4,810	5,440
Calaveras River	Bellota	470	104	3,520	9,520	9,530	10,640	12,500	12,500	12,500	16,000
Calaveras River	CL3	26.4	95	110	230	300	440	530	620	720	810
Calaveras River	SL2	488	95	4,150	10,150	10,620	12,140	14,210	14,960	15,320	19,510
Calaveras River	SR1	503	95	4,150	10,150	10,630	12,150	14,220	14,970	15,340	19,530
Calaveras River	SL1	532	95	4,150	10,150	10,670	12,230	14,320	15,070	15,440	19,620
Calaveras River	CR2 & CL2	591	95	3,810	9,620	10,050	12,530	13,670	15,650	16,110	20,230
Calaveras River	CR1	594	90	3,700	9,660	9,780	12,520	13,320	15,610	16,100	20,190
Calaveras River	CL1	594	90	3,700	9,660	9,780	12,520	13,320	15,610	16,100	20,190
Calaveras River	D4 & D5	-	57	7.30	8.30	8.95	9.20	9.30	9.40	9.49	9.60
Littlejohn Creek	Farmington Dam	212	53	1,400	2,170	2,370	1,990	3,360	8,660	12,000	16,210
Littlejohn Creek	at Farmington	247.9	58	1,400	2,170	2,370	2,620	3,740	9,900	12,900	16,600
Duck Creek	Farmington	8.25	58	130	200	240	300	340	380	420	470
French Camp Slough	FL1, FR1	-	57	7.30	8.30	8.95	9.20	9.30	9.40	9.49	9.60
San Joaquin River	Vernalis	13,536	82	25,000	32,000	35,109	42,000	47,676	78,209	124,587	165,208

Notes:
Bear Creek, Mosher Slough, Cosgrove Creek, and Duck Creek are unregulated streams.
The discharge values in this table represent the worst case storm centering.
The index point locations are shown on plates 1 to 4.
See the Hydrology Appendices by Ford or PBI for details not shown here.
Bear Creek index point D2, Calaveras index points D4 & D5, and French Camp index points FL1 & FR1 are based on a tide stage frequency analysis.
The flows for the San Joaquin river were extracted from a UNET model from the Comp Study 2002.

Table 6. Future Conditions Regulated Flows (CFS)



Future Conditions Regulated Discharge Summary Table at Index Points

Stream	Index Point	Drainage Area	Period of Record	Regulated Peak Discharge or Stage by Return Period and Annual Exceedance Probability							
				2 0.5	5 0.2	10 0.1	25 0.04	50 0.02	100 0.01	200 0.005	500 0.002
Bear Creek	Lockeford gage	47.6	51								
Bear Creek	BL4	79.5	25	1,900	2,680	3,300	4,180	4,890	5,560	6,320	7,410
Bear Creek	BL3	91.9	25	2,060	2,940	3,630	4,810	5,710	6,620	7,570	8,880
Bear Creek	BR4	94.2	25	2,060	2,940	3,670	4,850	5,770	6,680	7,650	8,970
Bear Creek	BR3	95.1	25	2,070	2,960	3,710	4,890	5,820	6,730	7,700	9,010
Bear Creek	BL2	95.9	25	2,070	2,970	3,740	4,920	5,860	6,790	7,790	9,100
Bear Creek	BL1	97.3	25	2,080	2,980	3,790	5,000	5,920	6,900	7,870	9,230
Bear Creek	BR2	99.0	25	2,110	3,020	3,840	5,050	6,070	7,030	7,960	9,380
Bear Creek	BR1	114.2	25	2,170	3,070	4,050	5,470	6,600	7,750	8,810	10,410
Bear Creek	D2	-	57	7.25	8.20	8.90	9.05	9.29	9.45	9.58	9.76
Mosher Slough	ML2	1.28	20	170	230	230	230	230	230	230	510
Mosher Slough	ML1	7.55	20	440	620	690	800	890	940	960	970
Calaveras River	New Hogan Dam	363	86	3,320	8,990	9,310	10,440	12,330	12,400	12,500	14,820
Cosgrove Creek	Valley Springs	21.1	51	1,020	1,880	2,480	3,240	3,780	4,310	4,810	5,440
Calaveras River	Bellota	470	104	3,520	9,520	9,530	10,640	12,500	12,500	12,500	16,000
Calaveras River	CL2	26.4	95	110	230	300	440	530	620	720	810
Calaveras River	SL1	488	95	4,150	10,150	10,620	12,140	14,210	14,960	15,320	19,510
Calaveras River	SR0	503	95	4,150	10,150	10,630	12,150	14,220	14,970	15,340	19,530
Calaveras River	SL0	532	95	4,150	10,150	10,670	12,230	14,320	15,070	15,440	19,620
Calaveras River	CR2 & CL1	591	95	3,810	9,620	10,050	12,530	13,670	15,650	16,110	20,230
Calaveras River	CR0	594	95	3,700	9,660	9,780	12,520	13,320	15,610	16,100	20,190
Calaveras River	CL0	594	95	3,700	9,660	9,780	12,520	13,320	15,610	16,100	20,190
Calaveras River	D4 & D4	-	57	7.30	8.30	8.95	9.20	9.30	9.40	9.49	9.60
Littlejohn Creek	Farmington Dam	212	53	1,400	2,170	2,370	1,990	3,360	8,660	12,000	16,210
Littlejohn Creek	at Farmington	247.9	58	1,400	2,170	2,370	2,620	3,740	9,900	12,900	16,600
Duck Creek	Farmington	8.25	58	130	200	240	300	340	380	420	470
French Camp Slough	D7 & D7	-	57	7.30	8.30	8.95	9.20	9.30	9.40	9.49	9.60
San Joaquin River	Vernalis	13,536	82	25,000	32,000	35,109	42,000	47,676	78,209	124,587	165,208

Notes:

Bear Creek, Mosher Slough, Cosgrove Creek, and Duck Creek are unregulated streams.

The discharge values in this table represent the worst case storm centering.

The index point locations are shown on plates 1 to 4.

See the Hydrology Appendices by Ford or PBI for details not shown here.

Bear Creek index point D2, Calaveras index points D4 & D5, and French Camp index points FL1 & FR1 are based on a tide stage frequency analysis.

The flows for the San Joaquin river were extracted from a UNET model from the Comp Study 2002.

Table 7. Existing Conditions Unregulated Flows (CFS)

Existing Conditions Unregulated Discharge Summary Table at Index Points

Stream	Location	Drainage Area (sq mi)	Period of Record (years)	Unregulated 1-day Discharge by Return Period and Annual Exceedance Probability							
				2	5	10	25	50	100	200	500
				0.5	0.2	0.1	0.04	0.02	0.01	0.005	0.002
San Joaquin River	Maze Road		82	19,203	44,753	68,988	108,667	145,171	187,885	237,393	314,324
San Joaquin River	Vernalis	13,536	82	24,126	56,984	88,444	140,317	188,312	244,715	310,343	412,740
Littlejohn Creek	Farmington Dam	212	58	2,471	5,682	8,061	11,034	13,118	15,044	16,810	18,903
Littlejohn Creek	at Farmington	247.9	58	2,730	7,015	10,438	14,930	18,192	21,282	24,173	27,668
Duck Creek	Farmington	8.25	58	128	196	241	297	339	379	419	472
Calaveras River	New Hogan Dam	363	104	5,627	13,000	18,618	25,855	31,081	36,039	40,701	46,391
Cosgrove Creek	Valley Springs	21.1	51	339	614	804	1,039	1,208	1,369	1,523	1,716
Calaveras River	Bellota	470	104	6,909	15,401	21,677	29,582	35,185	40,426	45,293	51,153

Notes:

The discharge values in this table represent the worst case storm centering.

The index point locations are shown on plate 5.

See the Calaveras River and Littlejohn Creek Frequency Reports by David Ford Consulting Engineers for details on those streams.

See the Sacramento-San Joaquin Comprehensive Study for details on the San Joaquin River.

Flow frequency estimates for the San Joaquin River are based on analysis described in the Sacramento and San Joaquin River Basins Comprehensive Study documentation. Flow frequency curves and hydrographs of unregulated flow were developed for the 50% (1/2) to 0.2% (1/500) Annual Chance Exceedance probability (ACE) frequencies. Regional synthetic hydrology presented in these studies represents the best available data for the large flood sources (San Joaquin River) of the Lower San Joaquin River Feasibility Study. These hydrologic analyses have also been used as the foundation for several other feasibility studies in the region, such as the Sutter Basin Feasibility Study. DWR and USACE are in the process of developing new hydrologic frequency estimates for existing conditions; however, the results are not available until mid-2014. Therefore, this study utilizes the results from the Sacramento and San Joaquin River Basins Comprehensive Study hydrologic analysis.

Synthetic hydrology of the Sacramento and San Joaquin River Basins Comprehensive Study was based on transformation of unregulated hydrologic conditions to regulated conditions. This was accomplished by developing balanced unregulated hydrographs based upon historically patterned storm events. Balanced hydrographs have the same annual exceedance frequency for all flood durations. For example a 10% (1/10) ACE hydrograph contains the 10% (1/10) ACE 1-day flow, 10% (1/10) ACE 3-day average flow, 10% (1/10) ACE 5-day average flow etc. These balanced hydrographs were then transformed to regulated hydrographs using an HEC-5 reservoir operations model of the system. The HEC-5 model, also developed and calibrated for the Sacramento and San Joaquin River Basins Comprehensive Study, simulates reservoir operations and produces regulated hydrographs. The comprehensive study transferred the hydrographs from the HEC-5 model at ‘handoff’ points and modeled in more hydraulic detail using UNET. The portion of the UNET model downstream of the San Joaquin River at Newman was replaced by an HEC-RAS unsteady model developed for this study (see hydraulics section). Hydrographs at San Joaquin River at Newman were obtained from the UNET model. All other hydrograph boundary conditions were obtained from the HEC-5 model. This process is shown on plate 19 (reference plate 6).

The Sacramento and San Joaquin River Basins Comprehensive Study hydrology utilized a runoff centering approach to evaluate possible hydrologic scenarios. A centering is multiple and varying frequency hydrographs positioned (centered) over a watershed to produce flow rates or stages of one specific frequency at a specific location (like Vernalis). Multiple centering scenarios are possible due to the diverse spectrum of floods that can occur from different combinations of concurrent storms on tributaries, orographic influences, and other factors that influence regional rainfall runoff events. The Comprehensive Study evaluated a suite of recorded flood centerings and generally tried to mimic general characteristics of those that historically produced the higher flows at a given location. For the Lower San Joaquin Feasibility study area, the Sacramento and San Joaquin River Basins Comprehensive Study results were reviewed and narrowed to one possible centering. The San Joaquin at Vernalis storm centering predominantly applies to the San Joaquin River downstream of Vernalis and the Stockton area.

7.2 Risk and Uncertainty Parameters

Uncertainties that Most Influence the Alternative Selection

For this study, Corps risk assessment procedures, incorporating uncertainty analysis, were followed. These procedures incorporate the best-available hydrologic, hydraulic, geotechnical, and economic information to compute expected annual damage (EAD), accounting explicitly for uncertainty in the information.

Each aspect of the flood risk assessment must account for uncertainty. For hydrologic and hydraulic analysis, the principle variables are discharge and water surface elevation. Uncertainty in discharge exists because record lengths are often short or do not exist where needed, precipitation-runoff computation methods are inaccurate, and the effectiveness of flood flow regulation measures is not known precisely. Uncertainty factors that affect water surface elevation include conveyance roughness, cross-section geometry, debris accumulation, ice effects, sediment transport, flow regime, and bed form. For geotechnical and structural analyses, the principle source of uncertainty is the structural performance of an existing levee due to its physical characteristics and construction quality. Uncertainty also arises from a lack of information about the relationship between depth and inundation damage, lack of accuracy in estimating structure and content values and locations, and the lack of ability to predict how the public will respond to a flood. These specific variables were explicitly accounted for in this risk assessment and via a sensitivity analysis the uncertainty in the hydrology most influence the damage and engineering performance outputs and thus the alternative selection. However, variables not explicitly evaluated that could influence future performance include climate change, or unforeseen changes in the watershed conditions such as unplanned growth or dramatic changes in agricultural practices.

Risk is defined as the probability that an event will occur, and the consequence of that outcome. Uncertainty is defined as a measure of insufficient knowledge of parameters and functions used to describe the hydraulic, hydrologic, geotechnical and economic aspects of a project plan. Risk analysis is an approach to evaluation and decision-making that explicitly incorporates estimates of risk and uncertainty in a flood damage reduction study. The annual exceedance probability or AEP is the probability that a flood event will occur in any given year, considering the full range of possible annual floods.

Unregulated flow frequency curves for Mormon Slough at Bellota, Farmington Dam, Littlejohn Creek at Farmington, and the San Joaquin River at Vernalis were developed by the direct analytical approach. A reservoir routing model was then used to regulate unregulated hydrographs. The direct analytical approach is used when a sample of stream gauge annual discharge values are available and the data can be fit with a statistical distribution. The median function is used in the risk based analysis. The derived function may then be used to predict specified exceedance probabilities. The approach generally follows USACE guidance including EM 1110-2-1415 and ER 1110-2-1450. The confidence limits will be computed within the HEC-FDA program from the period-of-record provided with the flow frequency statistics. An unregulated to regulated transform will be linked with the unregulated flow frequency curve in FDA. The lower Calaveras River watershed downstream of Mormon Slough at Bellota was modeled using a rainfall runoff model to produce concurrent local flow runoff when an a specific frequency event occurs at Bellota. Since approximately 75% or more of the total flow contained in the watershed’s levees comes from sources upstream of Bellota, a decision was made to use the unregulated 1-day frequency curve statistics with equivalent period of record for all downstream index points (except those impacted by Delta tides). An unregulated to regulated peak flow transform is linked to the unregulated 1-day frequency curve in FDA, with regulated peak based on the peak of the various frequency rainfall runoff model hydrographs produced at each index location.

The flood flow frequency estimates for Bear Creek, Mosher Slough, and for French Camp Slough downstream of Littlejohn Creek at Farmington were developed as hypothetical frequency events in a rainfall runoff model. In this case unique discharge hydrographs due to storms of specified probabilities and temporal and areal distributions are computed with a rainfall-runoff model. Flow frequency curves from rainfall runoff models are typically expressed as a graphical function. The graphical approach uses plotting positions to define the relationship with the actual function fitted by “eye” through the plotting position points. The confidence limits for flood flow estimates developed by use of rainfall-runoff models will be by equivalent record length guidelines as shown in table 8 below. Table 8 was extracted from EM 1110-2-1619, table 4-5.

Delta gage stage frequency curves and associated periods of record were used for tidally influenced points on the lower Bear Creek, lower Calaveras River, and French Camp Slough.

The final assessment of equivalent record length for each location is presented in tables 5 and 6.

TABLE 8

Equivalent Record Length Guidelines

Method of Frequency Function Estimation	**Equivalent Record Length**[1]
Analytical distribution fitted with long-period gauged record available at site	Systematic record length
Estimated from analytical distribution fitted for long-period gauge on the same stream, with upstream drainage area within 20% of that of point of interest	90% to 100% of record length of gauged location
Estimated from analytical distribution fitted for long-period gauge within same watershed	50% to 90% of record length
Estimated with regional discharge-probability function parameters	Average length of record used in regional study
Estimated with rainfall-runoff-routing model calibrated to several events recorded at short-interval event gauge in watershed	20 to 30 years
Estimated with rainfall-runoff-routing model with regional model parameters (no rainfall-runoff-routing model calibration)	10 to 30 years
Estimated with rainfall-runoff-routing model with handbook or textbook model parameters	10 to 15 years

[1] Based on judgment to account for the quality of any data used in the analysis, for the degree of confidence in models, and for previous experience with similar studies.

This table was developed after table 4-5 in EM 1110-2-1619, Risk based analysis for flood damage reduction studies.

Bear Creek hydrology is based on a rainfall-runoff model calibrated to an observed event at a short-interval runoff gage.

Mosher Slough is based on a rainfall runoff model. The model wasn’t calibrated to an observed event, however, because stream flows are largely dependent on pumped flows, the degree of uncertainty is judged to be equivalent to a calibrated model.

The Mormon Slough at Bellota index point equivalent record is based on “half” the period of record of the 1-day unregulated flow frequency curve at that location. It was reduced in half because of uncertainty about how efficiently the dam can operate to local flow conditions. This equivalent record was also adopted for multiple index points downstream of Bellota since approximately 75% or more of the total flow in the downstream levees is from sources upstream of Bellota.

The equivalent record length for French Camp Slough is based on the period of record of the tide gages analyzed for this location. Backwater from the San Joaquin River and the Delta (not discharges from the French Camp Slough watershed) determine the highest stages at this location. Littlejohn Creek at Farmington equivalent record is based on the period of record of the unregulated flow frequency curves at that location. There were no gages to calibrate the Duck Creek portion of the rainfall runoff model. The entire French Camp Slough rainfall runoff model (used to produce concurrent local flow contributions downstream of Littlejohn Creek at Farmington, Ca including Duck Creek) wasn’t calibrated to an observed event; however the soil loss rates were adjusted based on the calibration of the neighboring Calaveras River model.

The equivalent period of records that are used in HEC-FDA to establish the confidence limits for the flood flow frequencies are shown in tables 5 and 6.

8.0 FLOOD DAMAGES

Major flooding occurred in San Joaquin, Stanislaus, and Merced counties along the lower San Joaquin River in 1983, 1986, 1995 and 1997 [10]. The distribution of flood damages among the three counties has varied considerably depending upon storm paths. However, the highest magnitude of damages occurred to agricultural crops and developments. The 1997 flood event did, however, damage 1,842 residences, mobile homes, and businesses in San Joaquin and Stanislaus counties. Estimated average annual equivalent damages (year 2000) from floods in the Lower San Joaquin River Basin amount to about $20 million based on preliminary HEC-FDA model for the Comprehensive Study. Crop damages ($9 million) account for nearly half of the estimated damages.

Table 11 below entitled “Historical Flooding in the Calaveras River” is provided using data from the 1983 Water Control manual and updated through 2012 with data from CDEC and Corps files.

There is some evidence to suggest that sediment deposition has contributed to reducing channel capacities and contributed to flood problems within the study area. Past farming practices directed sediment-laden agricultural drainage from fields to the river. Current practices are attempting to retain agricultural drainage on site. Upstream diversions on the San Joaquin River and tributaries have reduced the frequency of high flows, thereby reducing the transport of sediment through the river system.

The portion of the study area between Stockton and Tracy has experienced significant development within the past decade. The River Islands master planned community is currently proposed for 5,000 acres of the Stewart Tract between Paradise Cut, the San Joaquin River, and Old River. Applications for Corps and Central Valley Flood Protection Board (CVFPB) permits are currently pending. The proposed project would increase the conveyance capacity of Paradise Cut by setting back approximately 20,000 feet of existing levee and dry excavating approximately 3,000,000 cubic yards of material within the levee setback area. Paradise Cut is a bypass channel connecting to the San Joaquin River and increasing conveyance in the upstream portion of the San Joaquin River.

Flood damages along the San Joaquin River will likely continue to increase due to population growth and urban development. Although new structures will need to comply with land use regulations pursuant to the National Flood Insurance Program (NFIP), there will continue to be increases in flood damages due to residual risks from floods exceeding designed levels of protection, increased flood damages to automobiles and other property outside of regulated structures, and improvements to existing structures in the floodplain that increase the amount of property exposed to potential flood damages.

8.1. Storms and Floods in the Calaveras River Basin including New Hogan Dam

Rain floods can occur anytime during the period from November through April. This type of flood is usually caused by frontal systems from the Pacific Ocean moving against the Sierra Nevada. Rainfall intensities are generally moderate but prolonged over several days. The resulting floods are usually characterized by high peak flows of short duration, but when antecedent rainfall has resulted in saturated ground conditions or when the ground is frozen, the volume of runoff is much greater and flooding is more severe. [11].

Since the Calaveras River Basin is low-lying, snow and snowmelt runoff are negligible in contributing to flooding.

Thunderstorms lasting up to three hours can occur over small areas at higher elevations from late spring through early fall. The resulting runoff is characterized by high peak flows of short duration with low volumes. For small tributaries, peak flows from thunderstorms can approach those which occur during major winter rain floods, but flows on the Calaveras River are barely affected.

Quantitative information on flooding in the study area prior to 1900 is practically non-existent. Streamflow records extend from 1901 to the present for the Calaveras River. Descriptive data on flood events since the turn of the century may also be found in newspaper files; the authorization documents for the flood control projects on the Calaveras River; certain of the design documents for these projects; publications of the U.S. Geological Survey and U.S. Weather Bureau (now National Weather Service); and, since 1950, in unpublished post-flood reports prepared by the Corps of Engineers.

Although quantitative data does not exist for historical floods, descriptions of floods in the last half of the 19th Century indicate their large magnitudes. It is recorded that valley floor area of the Calaveras River was entirely inundated during a number of these floods; during floods that occurred in 1861-62, flooding on the valley floor was deep enough to permit riverboats to reach almost any locality in the inundated area.

The major floods that occurred during the earlier part of the 20th Century (March 1901, January 1909, January-February 1911, and January 1921) were all very similar in their impacts. Flooding was widespread, frequently extending entirely across the area between Mormon Slough and the Calaveras River in the vicinity of Linden, which was entirely flooded a number of times during the period. Subsequent to construction of the Diverting Canal (1910), floodwater ponded on its north side and extended far to the north and east. The area was frequently described as an inland sea. These floods caused extensive damage and great hardship, and repair, restoration, and recovery created major financial burdens on the county government and on the individuals directly affected.

Subsequent to 1936, the original Hogan Dam and Reservoir had a tempering effect on flooding in the study area. Floods that would have reached major proportions were largely averted by that project in February 1938 and February 1963.

The most widespread and destructive flood of any in the recorded history of the Central Valley occurred in December 1955. Floodwater broke out of the Calaveras River to inundate farmlands in the vicinity of Linden. Mormon Slough breached its levees and flooded along both sides from Bellota to the Diverting Canal. An extensive area north and east of the canal was inundated.

During the 1958 flood, Hogan Reservoir filled and spilled for the first time since its completion. About 3,000 acres of farmlands in the vicinity of Linden were flooded by the Calaveras River where two levee breaks occurred. Linden was threatened but not damaged. Levees along Mormon Slough were breached in a number of locations and about 7,000 acres of land flooded in a strip extending from Bellota to the Diverting Canal. A major levee break occurred near the head of the Diverting Canal. Flooding also occurred on 1,500 acres along the north side of the Diverting Canal.

Widespread flooding occurred in northern and central California and western Nevada in December 1964 and January 1965. Severe storms occurred over the watershed but flooding and flood damage was minimal because the levee and channel improvement project was nearly finished at the time and functioned effectively to prevent significant damage to agricultural and suburban residential developments. New Hogan Dam, which became operational just prior to the flood season, stored runoff from a moderately large flood and controlled flows downstream to non-damaging amounts.

8.2. Storms and Floods in the Littlejohn Creek Basin including Farmington Dam

Littlejohn Creek Basin lies on the western, or seaward, slope of the Sierra Nevada. The basin is partially shielded from general storms by the barrier of the Coast Ranges. The peaks rise from 3,000 to 5,000 feet (914 to 1,524 m) in elevation. General rain storms are carried into the basin by moist, unstable Pacific air masses that travel through the San Francisco Bay from the northwest. The Coast Range influences the rate and duration of precipitation that falls on the Littlejohn Creek Basin. General rain floods occur primarily between November and March. Prolonged heavy rainfall produces general rain floods characterized by high peak flows of moderate duration (2-3 days) and relatively shallow depths of 2 to 3 feet (61.0 to 91.4 cm). When antecedent rain has saturated the ground, flooding is more severe. [12].

Comparative flows for observed floods in the Littlejohn watershed since the turn of the century are shown in Table 9 on the next page. It should be noted that damage in the study area during most of the known past floods would have been significantly reduced if the floods had occurred with presently existing flood control facilities completed and in operation.

TABLE 9

HISTORICAL FLOOD FLOWS ON LITTLEJOHN CREEK AT FARMINGTON DAM

DATE	PEAK (cfs)	1-DAY VOL (acre-feet)	3-DAY VOL (acre-feet)
February 1986	23,600	18,952	45,593
April 1958	28,900	14,424	41,136
December 1955	20,000	16,854	34,727
February 1998	24,830	22,865	32,216
January 1983	16,500	12,986	28,128

Source: Water Management Section, Sacramento District, USACE

Other major floods within this century occurred in January-February 1911 and February 1917. Peak flows prior to these project events were 16,000 and 13,600 cfs, respectively. The legendary floods of 1861-1862 are judged to be the largest in peak flow and volume of runoff, but were less damaging than the floods listed due to the area being less populated and developed.

Farmington Reservoir offers flood protection to about 58,000 acres of agricultural land, suburban areas, and industrial properties in the area immediately south of Stockton. Flood damages within the basin are primarily agricultural. Four of the largest floods of record occurred in December 1955, April 1958, February 1986, and February 1998. Maximum storage (53,512 acre-feet) occurred in February 1998. Peak outflow (2,438 cfs) occurred in February 1986. Peak inflow (28,900 cfs) occurred in April 1958, as did the largest flows on Duck and Littlejohn creeks. In April 1958, Duck Creek flows at the Diversion reached a peak of 4,100 cfs, compared with 2,700 cfs in February 1986, 2,600 cfs in December 1955, and 2,100 cfs in February 1998. Similarly, the flow at Farmington peaked at 3,600 cfs in April 1958, compared with 3,000 cfs in February 1986, 2,750 cfs in December 1955, and 2,400 cfs in February 1998. The 1955 and 1958 floods caused much damage.

However, no significant flooding occurred within the Littlejohn Creek basin for the February 1986 event.

In December 1955, flooding in the Littlejohn Creek area affected about 1,800 acres. Farmington Reservoir controlled Littlejohn Creek inflows to a safe channel capacity, but the uncontrolled flow from Duck Creek through the Duck Creek Diversion Channel was more than the lower creek channels could carry. Flood damage was primarily concentrated about South Littlejohn Creek. On the south branch of the creek, the flood damaged barley crops, farm buildings, supplies and equipment. Flood damages on the north branch were primarily to residences and to small business establishments.

In the months preceding the April 1958 storm event, rainfall served to saturate the ground and increase the flood potential in the basin. Rainfall during January and February was about 200 percent of normal, totaling 11 inches (27.9 cm). During the two storm periods in March, there was an additional 6 inches (15.2 cm) of rain. For the period of 30 March through 6 April, a series of short and intense storms produced 6 inches (15.2 cm) of rain. The April floods were due to high flows and the inability of the local rainfall runoff to drain into the main channels. Sections of the natural sloughs and waterways were filled in, and the ground leveled for irrigation, without providing sufficient alternate drainage channels. The result was that about 2,000 acres of farmland were flooded. Depths of flooding varied from a few inches to two feet, with durations ranging from 12 hours to 10 days in ponded areas. Inundated crops included barley, alfalfa, and onions. There was also some damage to land from erosion, as well as to improvements and stored supplies. County roads also sustained fairly extensive damage.

In February 1986, the water level at Farmington Dam reached a high at elevation 155 feet. The flooded area behind the dam was completely drained within 13 days after this record flood event. For the period of 12-21 February, the Flowers Mountain precipitation gage received a total of 7.6 inches. The Stockton WSO Airport precipitation gage received a total of 5.98 inches, while a total of 5.88 inches was recorded for the Knights Ferry 2 ESE gage.

In February 1998, a succession of intense El Niño-driven storms swept over northern and central California for nearly four weeks. These cold storms, originating from the Gulf of Alaska, were accompanied by strong winds. The storms produced low snow levels and widespread showers and thunderstorms. In many areas the ground became nearly saturated due to the cumulative effect of the rains. According to NOAA, California experienced the wettest February on record. The Stockton WSO Airport precipitation gage received a total of 8.01 inches, approximately 360 percent of average. The Flowers Mountain precipitation gage received a rainfall amount totaling about 12.2 inches, approximately 330 percent of average. The Farmington Reservoir pool elevation reached 156.89 feet. This was the first time the pool elevation had exceeded the gross pool level since completion of the project. Farmington Dam and Reservoir were able to prevent an estimated $3.5 million in flood damages.

Table 10. Dams and Lakes in the San Joaquin River Basin

Dams and Lakes in the San Joaquin River Basin

Dam/Lake	Tributary Stream	Storage (Ac-Ft)	Owner / Operator
		Gross Pool	
SAN JOAQUIN RIVER BASIN			
Camanche	Mokelumne River	417,000	EBMUD
New Hogan	Calaveras River	317,100	USACE
Farmington	Little John Creek	52,000	USACE
New Melones	Stanislaus River	2,420,000	USBR
Tulloch	Stanislaus River	67,000	USBR
Don Pedro	Tuolumne River	2,030,000	TID
New Exchequer/ McClure	Merced River	1,024,000	MID
Burns	Bear Creek / Merced Stream Group	6,800	USACE
Bear	Bear Creek / Merced Stream Group	7,700	USACE
Owens	Owens Creek / Merced Stream Group	3,600	USACE
Mariposa	Bear Creek / Merced Stream Group	15,000	USACE
Los Banos	Los Banos Creek	34,600	CA-DWR
Buchanan/Eastman	Chowcilla River	150,000	USACE
Hidden/Hensley	Fresno River	90,000	USACE
Friant/Millerton	San Joaquin River	520,500	USBR
Big Dry Creek	Big Dry Creek, tributary to the San Joaquin River	30,200	FMFCD
TULARE LAKEBED BASIN			
Pine Flat	Kings River	1,000,000	USACE
TOTAL SYSTEM STORAGE		8,185,500	

Key:

CA-DWR	California Department of Water Resources
EBMUD	East Bay Municipal Utilities District
FMFCD	Fresno Metropolitan Flood Control District
MID	Merced Irrigation District
TID	Turlock Irrigation District
USACE	US Army Corps of Engineers
USBR	US Bureau of Rclamation

Table 11. Historical Flooding on the Calaveras River

Historical Flooding in the Calaveras River (1 of 2)			
Flood	**Peak Flow (a) c.f.s.**		
	Recorded Peak Flow at Mormon Slough at Bellota	**Natural Flow at Jenny Lind**	**Calaveras River at Jenny Lind**
March 1907	(b)		34,600
January 1909	(b)		33,000
Jan-Feb 1911	(b)		50,000
January 1916	(b)		22,000
February 1917	(b)		31,300
March 1918	(b)		21,800
January 1921	(b)		37,900
February 1922	(b)		24,500
February 1925	(b)		27,500
February 1936	(b)	(37,000)	10,100
February 1938	(b)	(42,000)	10,600
Nov-Dec 1950	(9000)	(23,000)	7,600
December 1955	(16,000)	(33,000)	14,200
April 1958	15,400	(43,000)	12,100
February 1963	6,700	(25,000)	6,900
Dec 1964-Jan 1965	3,300	(33,000)	2,600
January 1969	10,700	(20,000)	(c)

Note: Neither the Jenny Lind gage nor the Bellota gage were in operation from February 1969 through March 1988.

Table 11. Historical Flooding on the Calaveras River

Historical Flooding in the Calaveras River (2 of 2)			
	Recorded Peak Flow at Mormon Slough at Bellota	Natural Flow at Bellota	Date of Peak at Bellota
April 1988	8,500	(8600)	22-Apr-88
June 1989	1,000	(900)	9-Jun-89
August 1990	1,200	(1200)	3-Mar-90
May 1991	7,900	(7900)	14-May-91
June 1992	4,100	(7000)	15-Feb-92
May 1993	7,600	(7600)	5-May-93
October 1993	1,800	Missing	(d)
May 1996	3,000	(10200)	21-Feb-96
January 1997	7,800	(29600)	2-Jan-97
February 1998	9,600	(40800)	3-Feb-98
February 1999	6,800	(19900)	9-Feb-99
February 2000	4,500	(16000)	25-Jan-00
March 2001	2,200	(5500)	5-Mar-01
January 2002	2,100	(6200)	3-Jan-02
December 2002	700	(4700)	16-Dec-02
February 2004	3,500	(6700)	2-Jan-04
March 2005	4,400	(14500)	23-Mar-05
April 2006	9,500	(32600)	4-Apr-06
February 2007	1,400	(6100)	27-Feb-07
January 2008	1,300	(5700)	28-Jan-08
March 2009	1,000	(10300)	4-Mar-09
January 2010	2,300	(6600)	22-Jan-10
March 2011	8,900	(18200)	20-Mar-11
April 2012	1,700	(6800)	13-Apr-12

(a) Flow values shown in ( ) are estimated. For the Jenny Lind station (1969 and prior), estimated peaks remove the effect of old Hogan dam (1936-1963) or New Hogan dam (1964-present); recorded flows are also shown for comparison. All flows are rounded.

(b) Station not in operation.

(c) Station discontinued.

(d) Station operated by USACE 1988 to 1996 with daily values and from 1996 to present with hourly values. Daily and hourly values from 1998 to present are observed flows affected by regulation of New Hogan dam. Natural peak flows () at Bellota are estimated from 1988 to 1995.

Source: New Hogan Water Control Manual, June 1983, and USACE DSS files.

Table 12. Drainage Area at Selected Locations in the San Joaquin River Basin

Drainage Area of Selected Locations in the San Joaquin River Basin
and Drainage Area Controlled by Upstream Dams
in upstream to downstream order

SAN JOAQUIN RIVER BASIN

USGS Station No.	Location / Dam and Lake	Tributary Stream	Drainage Area	Percent of dA Controlled
11221500	**Pine Flat Lake & Dam**	Kings River	**1545**	100%
11222000	at Piedra	Kings River	1693	91%
11250999	**Friant Dam/Millerton Lake**	San Joaquin River	**1638**	100%
11254001	at Mendota	San Joaquin River	3943	81%
11257999	**Hidden/Hensley**	Fresno River	**236**	100%
11258000	below Hidden dam near Daulton gage	Fresno River	258	91%
11258001	at East Side Bypass (approx)	Fresno River	480	49%
11258999	**Buchanan/Eastman**	Chowcilla River	**235**	100%
11259999	at East Side Bypass (approx)	Chowcilla River	600	39%
11260000	'at El Nido	San Joaquin River	6443	57%
11260288	**Burns**	Bear Creek / Merced Stream Group	**71.9**	100%
11260289	**Bear**	Bear Creek / Merced Stream Group	**72.3**	100%
11260291	**Owens**	Owens Creek / Merced Stream Group	**25.7**	100%
11260292	**Mariposa**	Bear Creek / Merced Stream Group	**108.5**	100%
11261500	at Fremont Ford Bridge	San Joaquin River	7615	52%
11262799	**Los Banos damsite**	Los Banos Creek	**156**	100%
11262800	near Los Banos	Los Banos Creek	159	98%
11273400	above Merced River near Newman	San Joaquin River	7949	51%
11270000	**New Exchequer/ McClure**	Merced River	**1037**	100%
11270610	at McSwain Dam	Merced River	1054	98%
11272500	at Stevinson	Merced River	1273	81%
11273500	at mouth of Merced at River Road Bridge	Merced River	1276	81%
11274000	near Newman	San Joaquin River	9520	54%
11274550	near Crows Landing	San Joaquin River	9694	53%
11274570	at Patterson Bridge near Patterson	San Joaquin River	9749	53%
11288000	**Don Pedro abv LaGrange Dam**	Tuolumne River	**1533**	100%
11290000	at Modesto	Tuolumne River	1884	81%
11290200	at Shiloh Road Bridge nr Grayson	Tuolumne River	1897	81%
11299200	**New Melones**	Stanislaus River	**904**	100%
11302000	below Goodwin Dam near Knights Ferry	Stanislaus River	986	92%
11302500	at Oakdale	Stanislaus River	1032	88%
11303000	at Ripon	Stanislaus River	1075	84%
11303500	***at Vernalis***	***San Joaquin River***	***13536***	***56%***
11308900	**New Hogan**	Calaveras River	**363**	100%
11309500	at Jenny Lind	Calaveras River	393	92%
11309599	Mormon Slough at Bellota	Calaveras River	470	77%
11309601	**Farmington**	Little John Creek	**212**	100%
11309602	at Farmington	Little John Creek	247.9	86%
11323500	**Camanche**	Mokelumne River	**621**	100%
11325500	at Woodbridge	Mokelumne River	661	94%

9.0 DELTA BASE FLOOD ELEVATION, TIDE STAGE FREQUENCY ANALYSIS

A stage frequency analysis was needed for Delta near Stockton. Initially, the analysis was described briefly in the hydrology appendix and focused on two key delta stage gages near Stockton called Rindge Pump gage and Burns Cutoff gage as shown in Figure 9.1. Recently, the Delta stage frequency analysis was moved to the Hydraulics Appendix. Please refer to the Hydraulics Appendix for further details of that analysis.

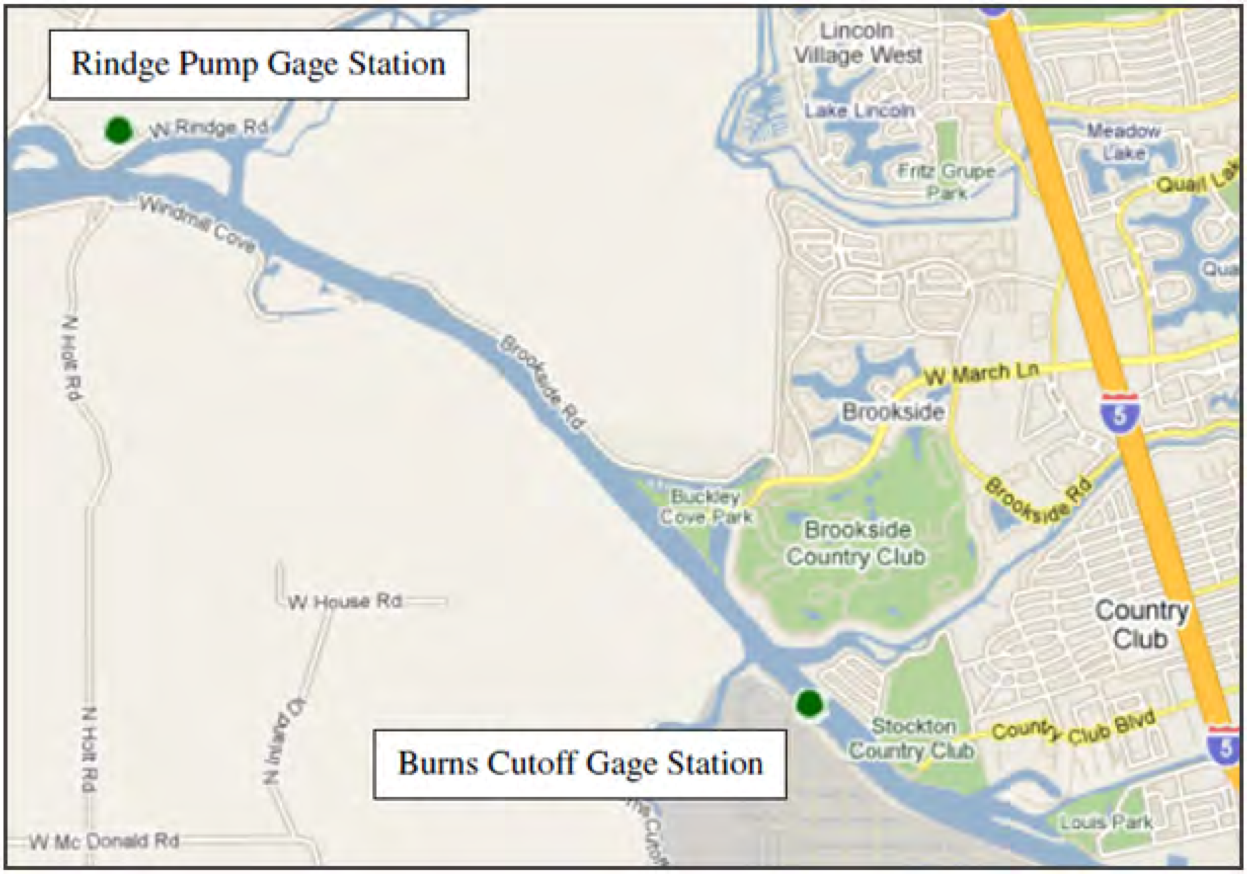


Figure 9.1 – Rindge Pump and Burns Cutoff Gage Station Location Map

10.0 HYDROLOGIC ANALYSIS OF ALTERNATIVES

None of the alternatives presently under consideration will have an effect on the existing or future condition hydrology of the basins and/or river reaches within the study area.

The operation of New Hogan dam was analyzed to determine the level of protection of the dam. The flow-frequency analysis shows that there is a 0.5 (1/200) ACE level of protection in the current operation of the dam and that no changes in operation are required to achieve the state goal of 1/200 year level of protection. The 1958 flood event was the only event in history that produced a spillway event. The New Hogan dam was not constructed until 1963, so the original (smaller) Hogan dam allowed that spillway event and consequential flooding. It was found that the flood control storage capacity of the reservoir lies between the 0.5 (1/200) ACE 3-day inflow volume and the 0.5 (1/200) ACE 4-day inflow volume. However, none of the historic events exceeded to total required storage volume. Therefore, a dam raise was considered infeasible. This analysis was done from a hydrologic perspective only and does not constitute a thorough reservoir re-operation or dam safety investigation as required by regulations. The details of the analysis are further described in a technical memorandum prepared for the LSJR feasibility study by David Ford Consulting Engineers in August of 2011 (Ford, 2011).

The State of California through the FloodSAFE program and the Central Valley Flood Protection Plan (CVFPP) will be studying the potential for re-operation of the flood control projects throughout the central valley. Because the Corps of Engineers has section 7 of the flood control act of 1944 authority over flood control operations, the Corps will engage with the state at an appropriate time. That analysis is not part of this feasibility study and the results will not be known for several years. Further information is available on the DWR website at: http://www.water.ca.gov/system_reop/.

The U.S. Bureau of Reclamation has underway a feasibility study for a new dam upstream of Friant dam and Millerton Lake on the upper San Joaquin river. The Temperance Flat project will provide additional flood protection to the study area, however, construction of the dam is in the future and cannot be considered in the future without project condition of this study. Further information is available online at: http://www.usbr.gov/mp/sccao/storage/docs/phase1_rpt_fnl/.

The U.S. Fish and Wildlife Service is performing a conservation study looking at alternatives for habitat and ecosystem restoration in the upper and lower San Joaquin River corridor. That study may provide additional flood protection benefits to the study area. However, those projects also cannot be considered part of the future without project condition. Further information is available at: http://www.fws.gov/sacramento/Fisheries/San-Joaquin/fisheries_san-joaquin.htm.

11.0 RESULTS AND CONCLUSIONS

A description of the study area, flood history and flood problems, and flood control projects has been presented.

The results of the design storm analysis, the unregulated flow frequency of Bear Creek at Lockeford, Cosgrove Creek at Valley Springs, the Calaveras River at New Hogan and Bellota, and Littlejohn Creek at Farmington Dam and at Farmington, and the San Joaquin River at Vernalis are provided.

In addition existing and future condition without project flows are provided at the damage index points that are shared with the hydraulic analysis, geotechnical analysis, and economic analysis teams.

The following technical memorandums are attached by reference as appendices to this summary hydrologic report:

1) Calaveras River watershed above Bellota hydrologic analysis, by USACE dated April, 2014.
2) Littlejohn Creek above Farmington, Ca hydrologic analysis by USACE, April 2014.
3) USACE Addendum to PBI Report, dated April 2014.
4) The Sacramento – San Joaquin Comprehensive Study, Technical Studies Documentation: Appendices A through D, USACE, 2002.
   On the world-wide-web at: http://130.165.3.37/reports.html
5) New Hogan Dam Water Control Manual, USACE, 1983.
6) Farming Dam Water Control Manual, USACE, 2004.

12.0 REFERENCES:

[1] United States Army Corps of Engineers (USACE), 2002, Sacramento – San Joaquin River Basins Comprehensive Study (Comp Study), California, Interim Report and Appendices.

[2] USACE, 2008, Lower San Joaquin River, California Feasibility Study, Project Management Plan.

[3] USACE, 2004, Lower San Joaquin River, California Section 905(b) Analysis (WRDA 1986).

[4] Peterson-Brustad, Inc (PBI), 2012, Lower San Joaquin River Feasibility Study, F3 Hydrology Appendix.

[5] Peterson-Brustad, Inc (PBI), 2010, San Joaquin River delta Base Flood Elevation Refinement Stage Frequency Analysis.

[6] David Ford Consulting Engineers Inc (Ford), 2011, Lower San Joaquin River Feasibility Study, Calaveras River frequency analysis and hydrographs.

[7] David Ford Consulting Engineers Inc (Ford), 2011, Lower San Joaquin River Feasibility Study, Littlejohn Creek frequency analysis and hydrographs.

[8] National Oceanic and Atmospheric Administration (NOAA), 2011, NOAA Atlas 14, Precipitation-Frequency Atlas of the United States, Volume 6 version 2.0 California.

[9] USACE, 2009, Guadalupe Watershed Hydrologic Assessment, Final Report.

[10] USACE, 1999, Sacramento and San Joaquin River Basins, California, Post - Flood Assessment.

[11] USACE, 1983, New Hogan Dam and Lake, Calaveras River, California, Water Control Manual.

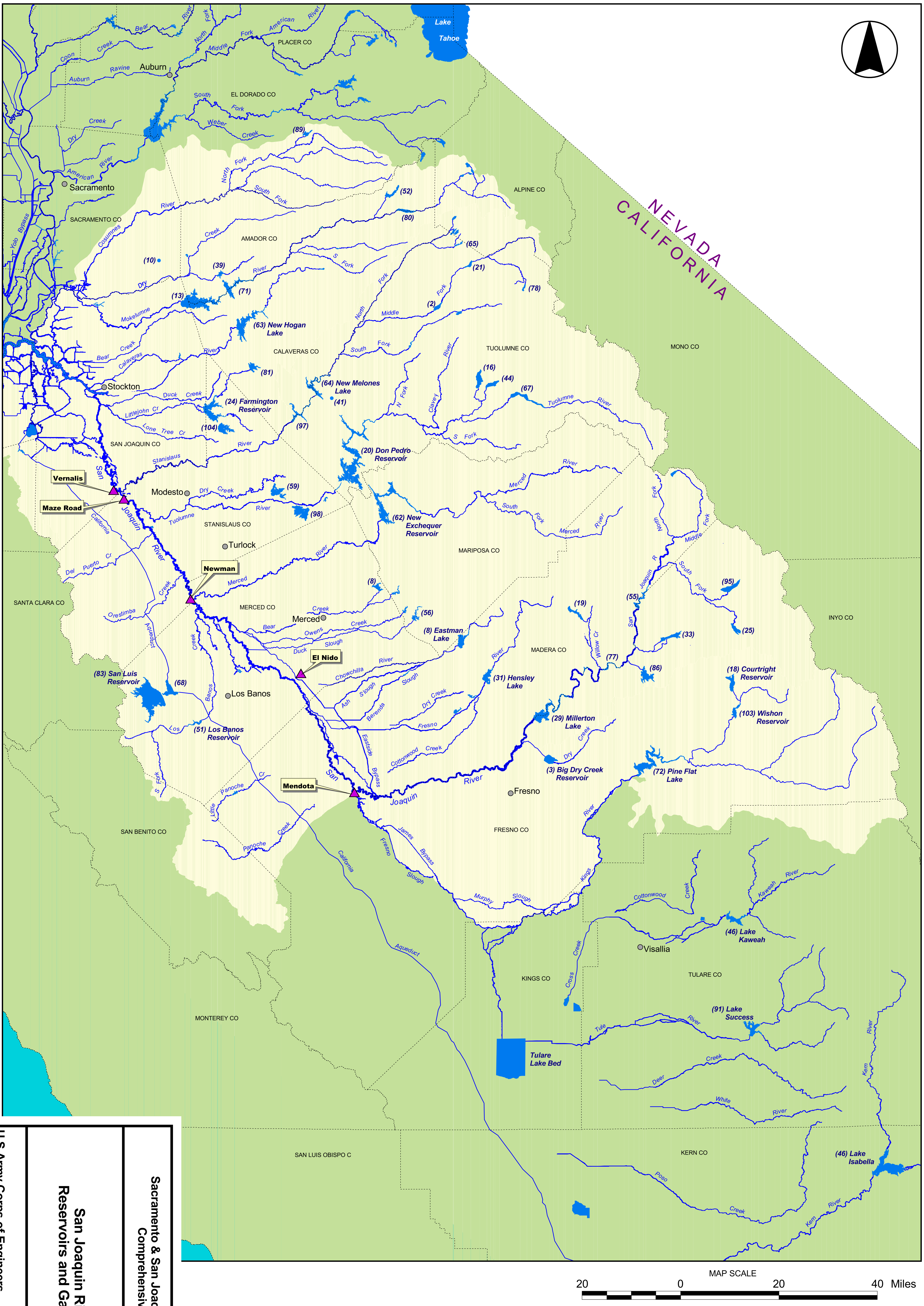
[12] USACE, 2004, Farmington Dam and Reservoir, Littlejohn Creek, California, Water Control Manual.

[13] San Joaquin Area Flood Control Agency (SJAFCA), 1998, Flood Protection Restoration Project, Final Technical Memorandums; No.1 Hydrology, No. 2 Hydraulics, No. 7 Residual Floodplains.

THIS PAGE LEFT BLANK

SEE PLATES BELOW

PLATES



Map Legend:

▭ San Joaquin Basin

🟦 Lake or Reservoir*

River or Stream

County Boundary

▲ Gaging Stations

● City

*Refer to Table II-1 of Appendix C for reservoir inventory number.

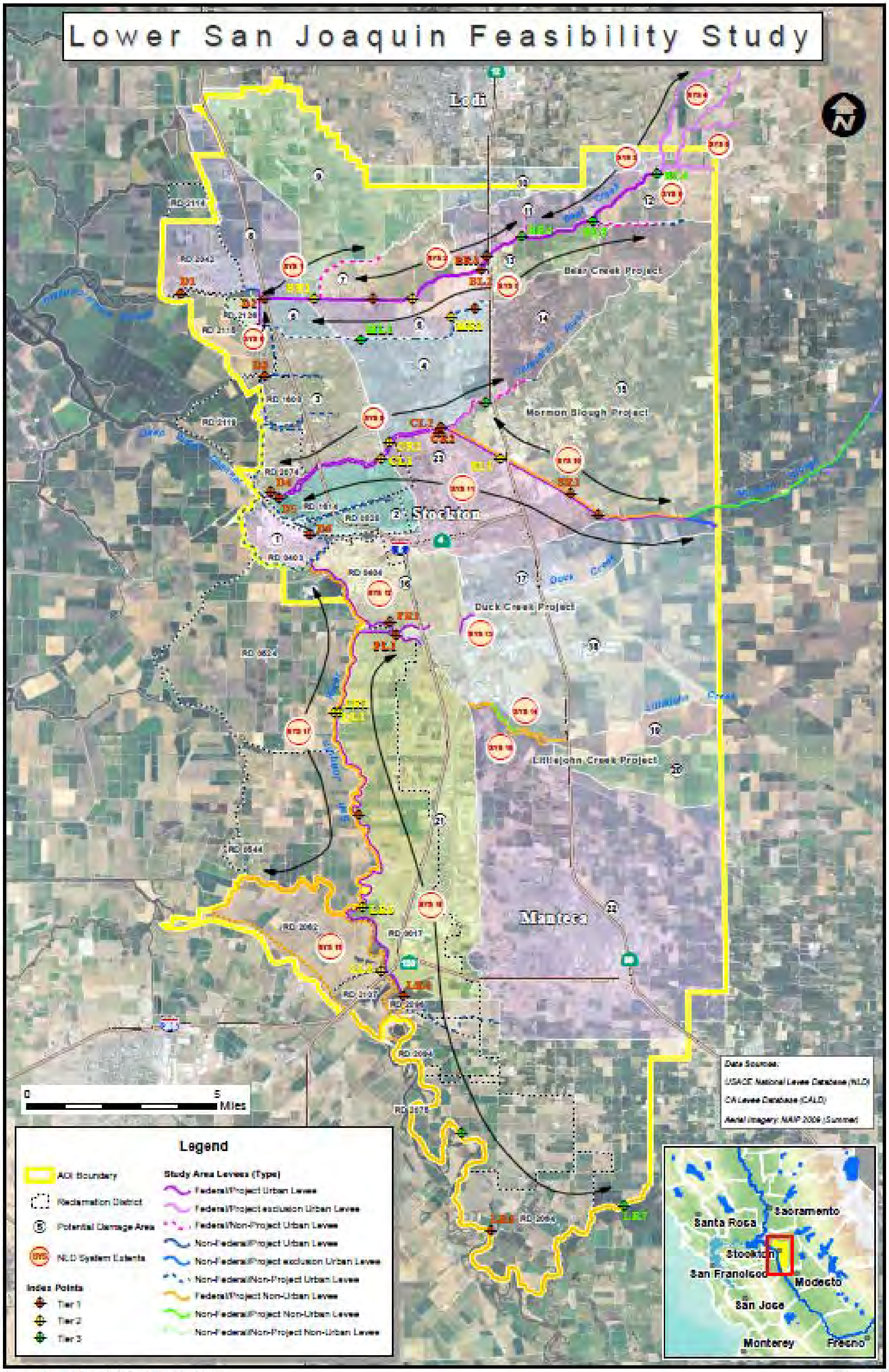
U.S.Army Corps of Engineers
Reclamation Board, State of California
June 2002

San Joaquin River Basin
Reservoirs and Gage Locations

Sacramento & San Joaquin River Basins
Comprehensive Study

PLATE 2

Plate 1. San Joaquin Basin Reservoir and Gage Location, from Comp Study



Prepared by Jim Mars, USACE - Sacramento District 12/14/2011

Plate 2. Lower San Joaquin Feasibility Study Area December 2011

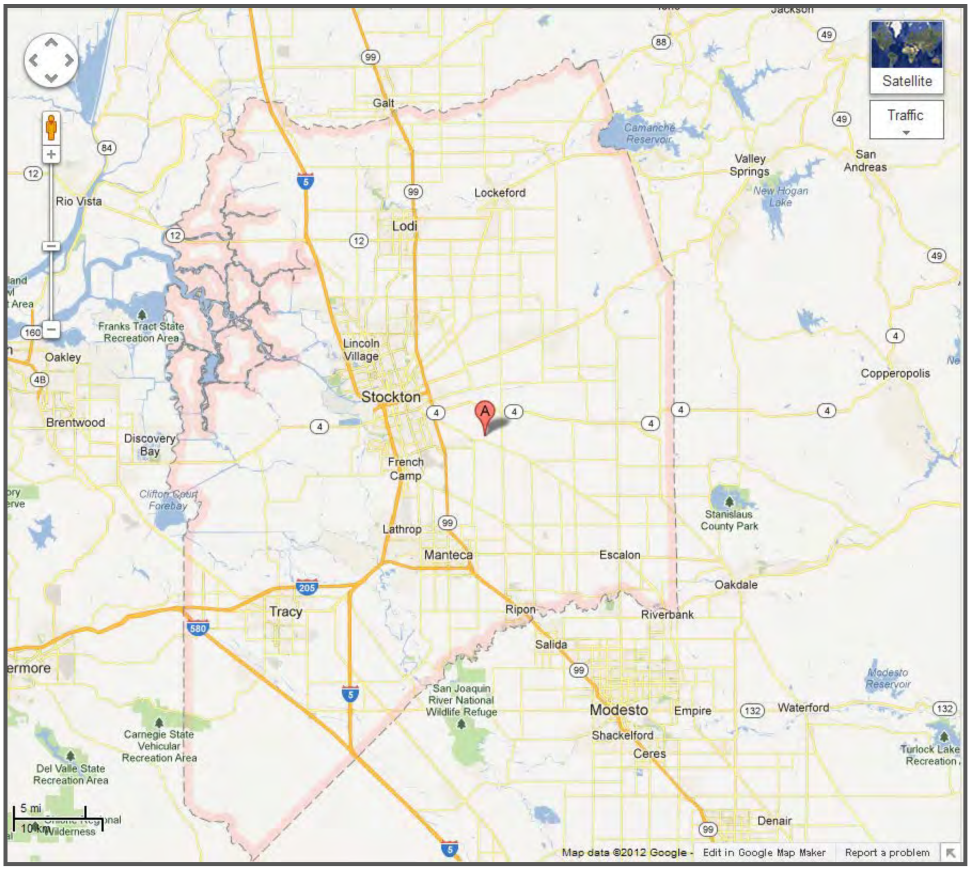


Plate 3. San Joaquin County, California boundary

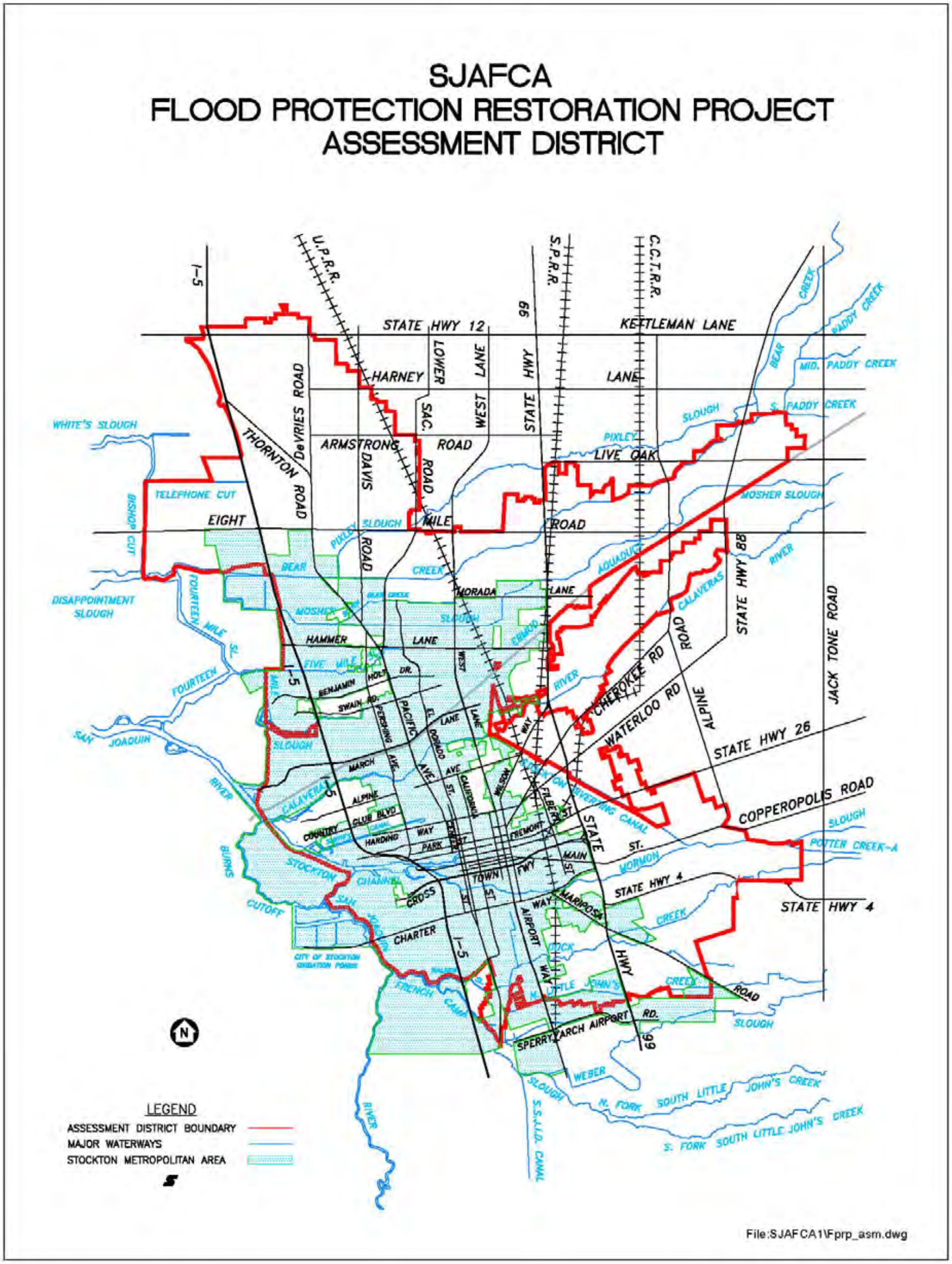


Plate 4. SJAFCA Boundary

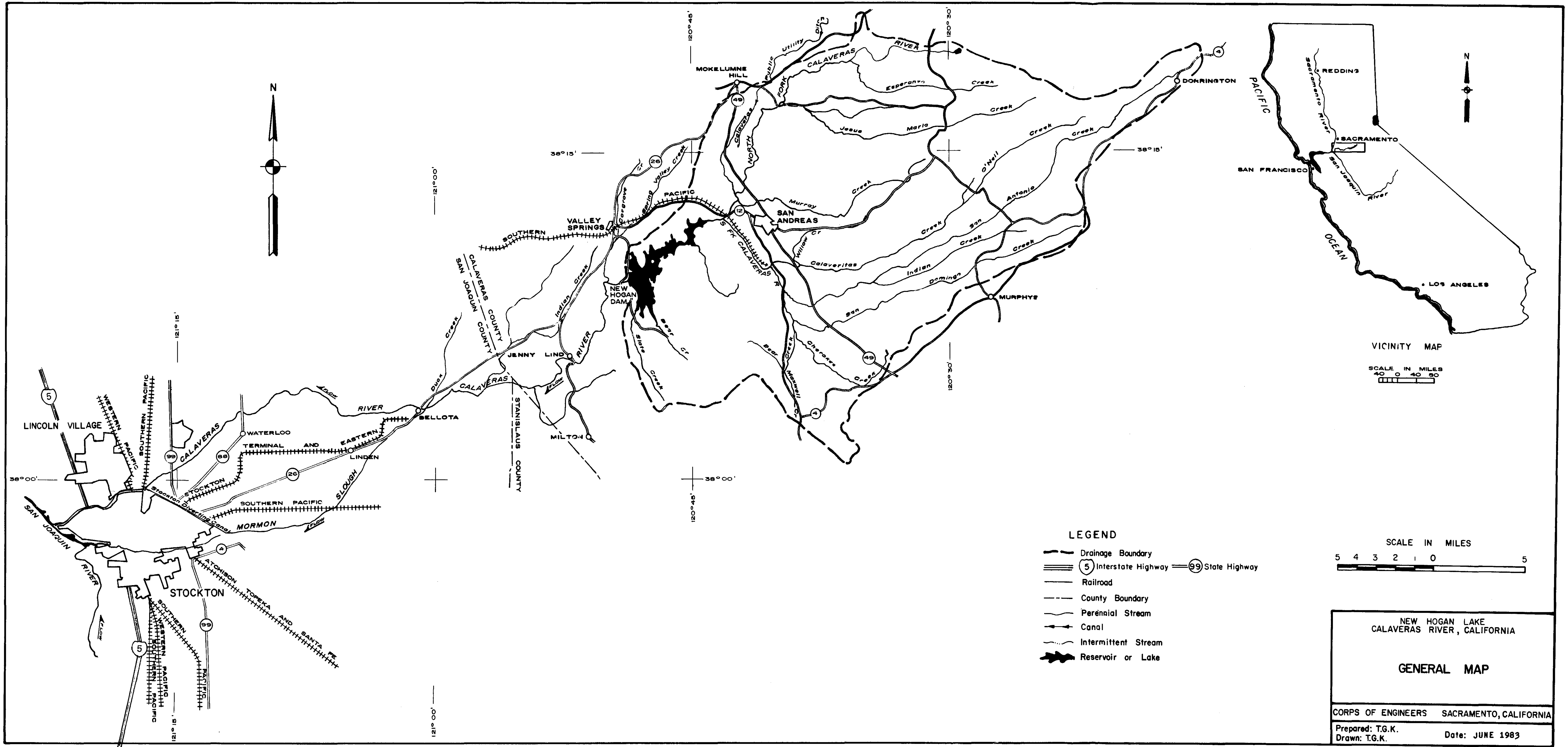


Plate 5. New Hogan Dam General Map

PLATE 2

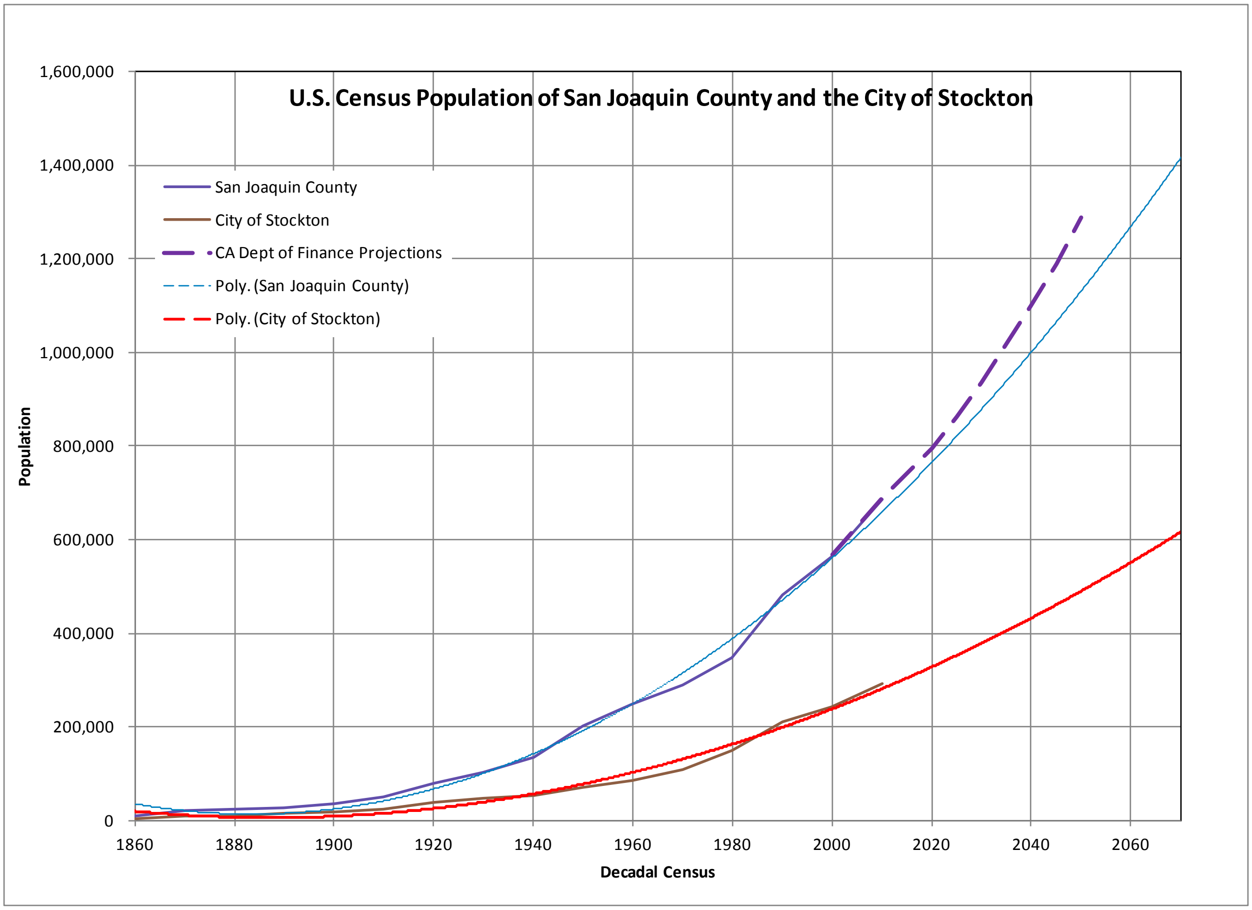
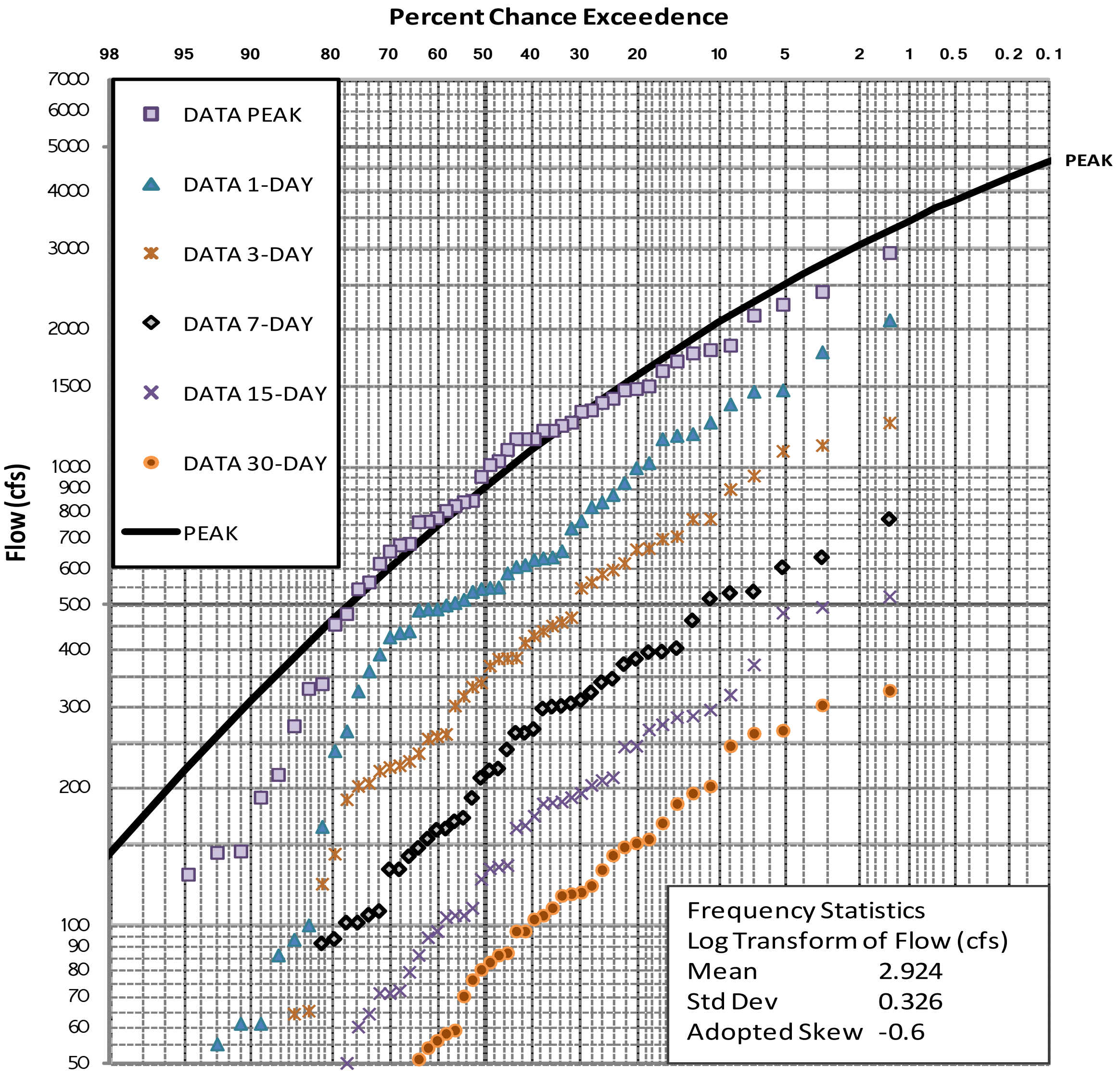


Plate 6. San Joaquin and Stockton Population 1960-2010 and Projection to 2070



NOTES:

1. Median plotting positions.
2. Computed Probability
3. Drainage area: 47.6 sq. mi.
4. WY 1932-1975, 1978-1985

LOWER SAN JOAQUIN FEASIBILITY STUDY
SAN JOAQUIN COUNTY, CALIFORNIA

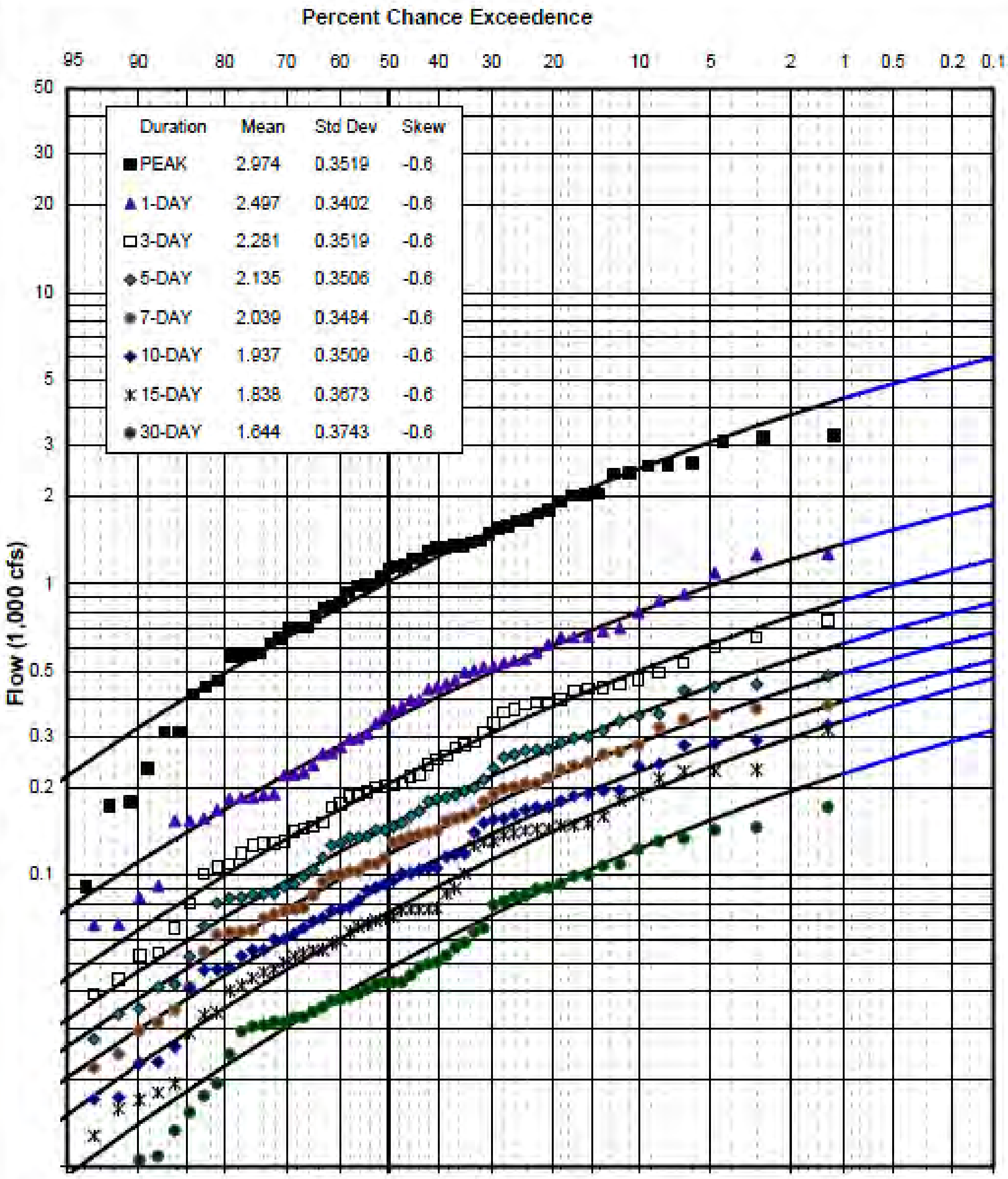
BEAR CREEK AT LOCKEFORD
FLOW FREQUENCY CURVE

UNREGULATED (NATURAL) CONDITION

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

6-Dec-12

Plate 7. Analytical Flow Frequency at Bear Creek at Lockeford



NOTES:
1. Period of record Water Years 1930-2009
2. Peak flow 61 yrs-, daily flows 56-yrs of record.
3. Median plotting positions used to plot flows.
4. Drainage area 21 sq.mi.
5. USGS Station 11309000

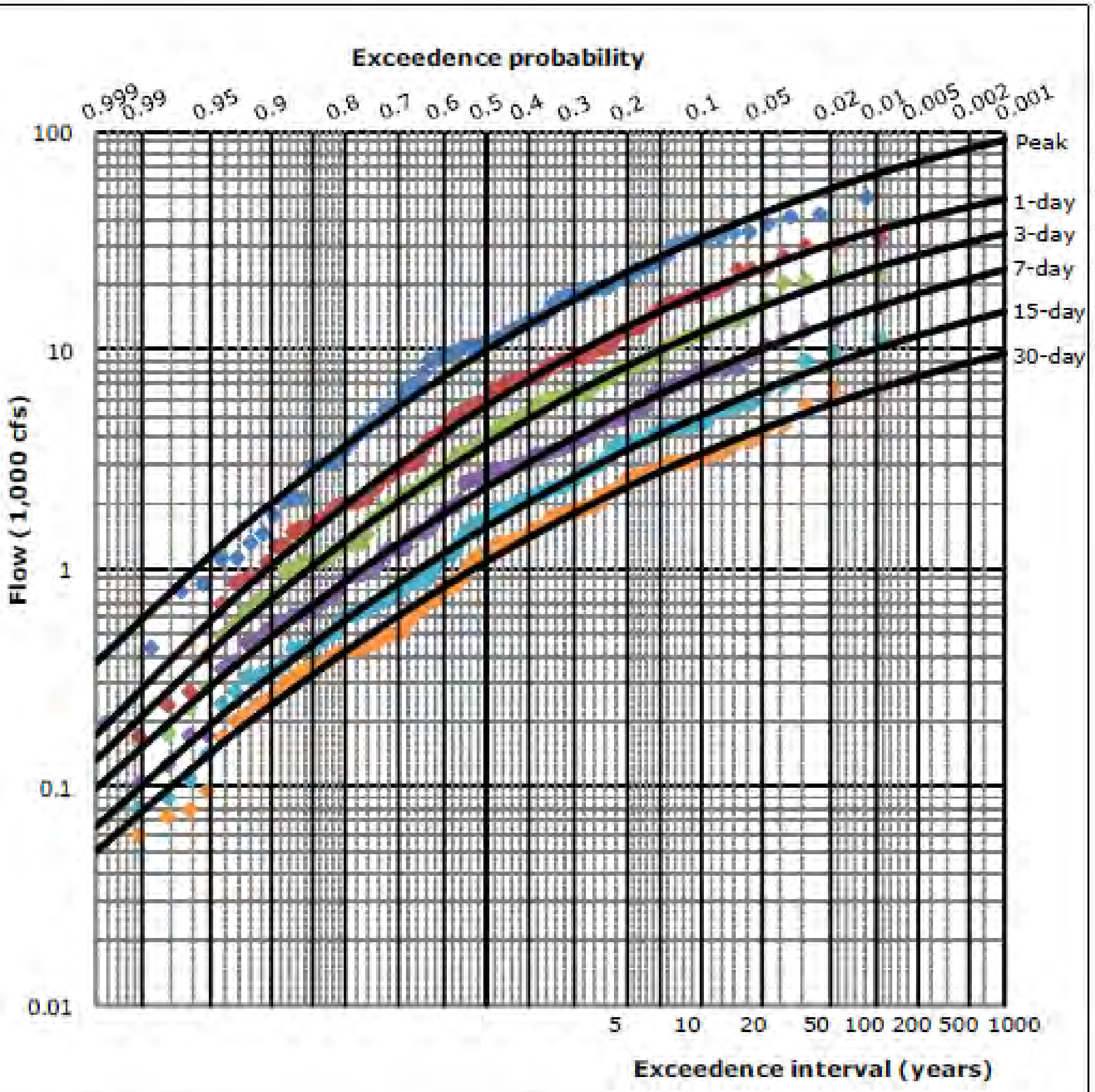
COSGROVE CREEK FEASIBILITY STUDY
CALAVERAS COUNTY, CALIFORNIA

RAIN FLOOD FREQUENCY CURVES

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

DEC 2009 PLATE 11

Plate 8. Analytical Flow Frequency at Cosgrove Creek at Valley Springs



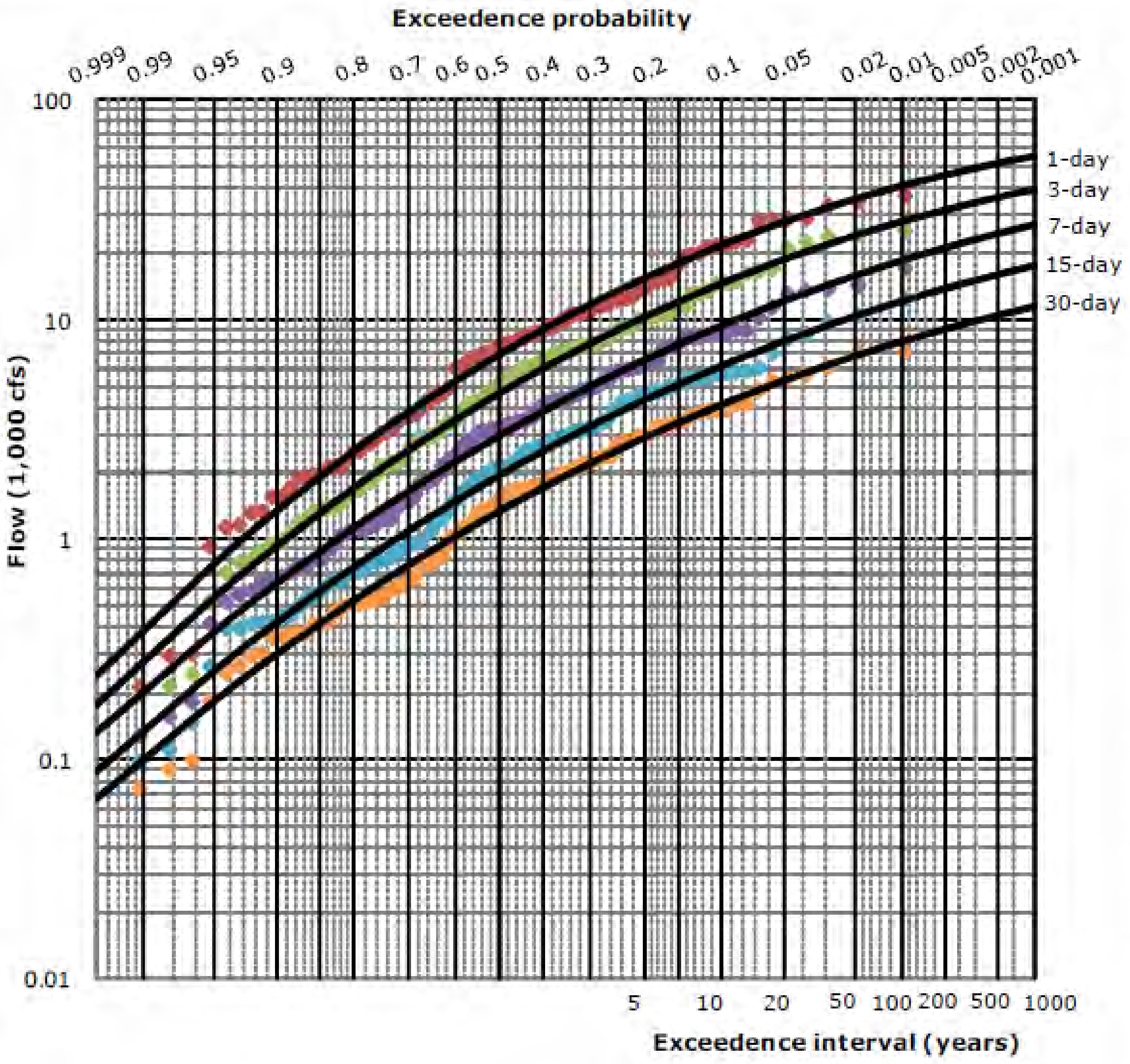
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.946	0.482	-0.727
1-day	3.685	0.501	-0.794
3-day	3.518	0.487	-0.732
7-day	3.324	0.477	-0.651
15-day	3.146	0.473	-0.656
30-day	2.988	0.457	-0.659

Notes:

- Median plotting positions.
- Drainage area: 363 sq. miles.
- Period of systematic record: 1907-2010.
  (Peak flow data intermittent 1930-2010).
- Record lengths
  Peak flows: 86 years.
  Volumes: 104 years.
- Regional skew values developed by USGS.

Plate 9. Analytical Unregulated Flow Frequency at New Hogan Dam



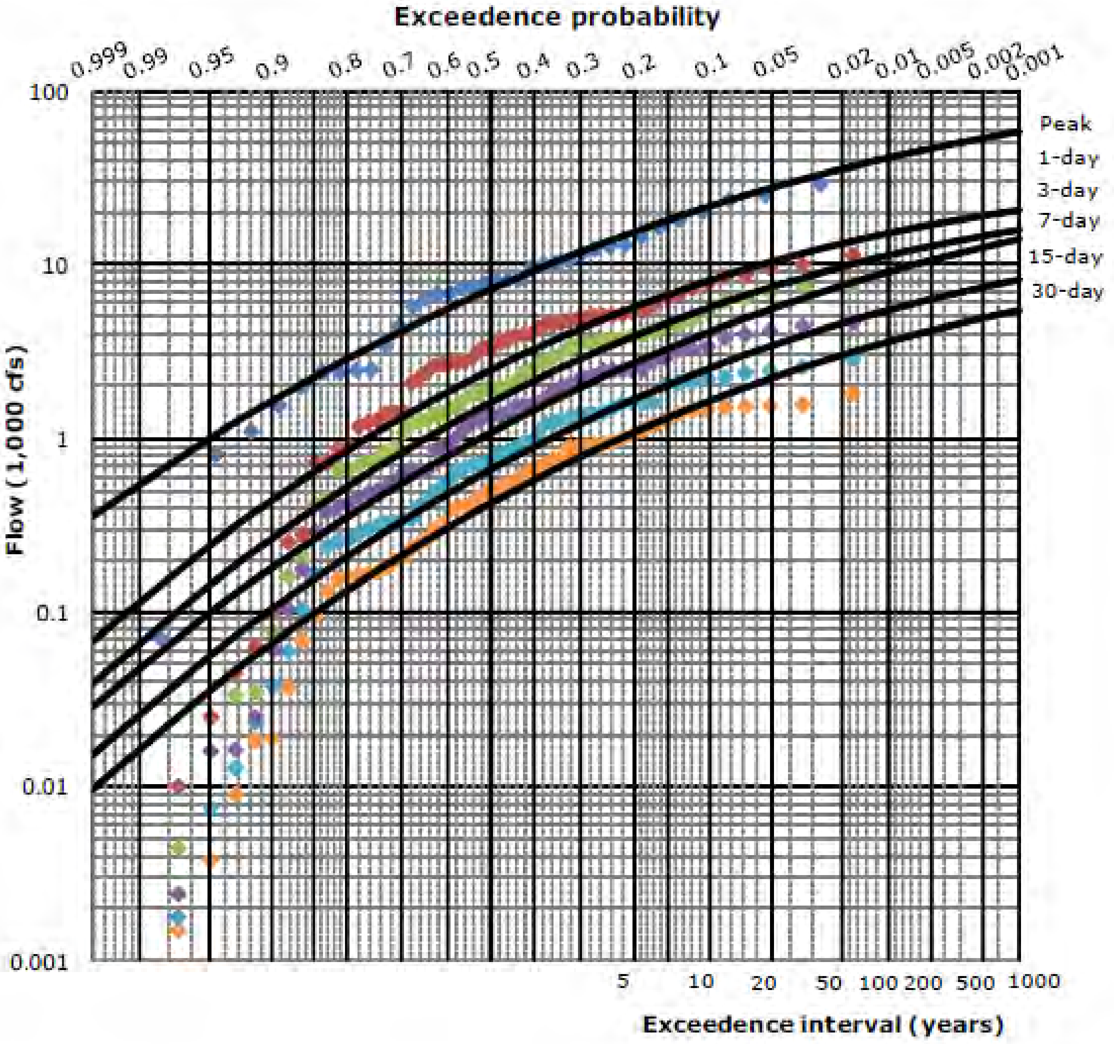
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.775	0.482	-0.810
3-day	3.608	0.475	-0.753
7-day	3.417	0.464	-0.666
15-day	3.240	0.461	-0.671
30-day	3.079	0.448	-0.668

Notes:

- Median plotting positions.
- Drainage area: 473 sq. miles.
- Period of systematic record: 1907-2010.
- Record length: 104 years.
- Regional skew values developed by USGS.

Plate 10. Analytical Unregulated Flow Frequency at Mormon Slough at Bellota



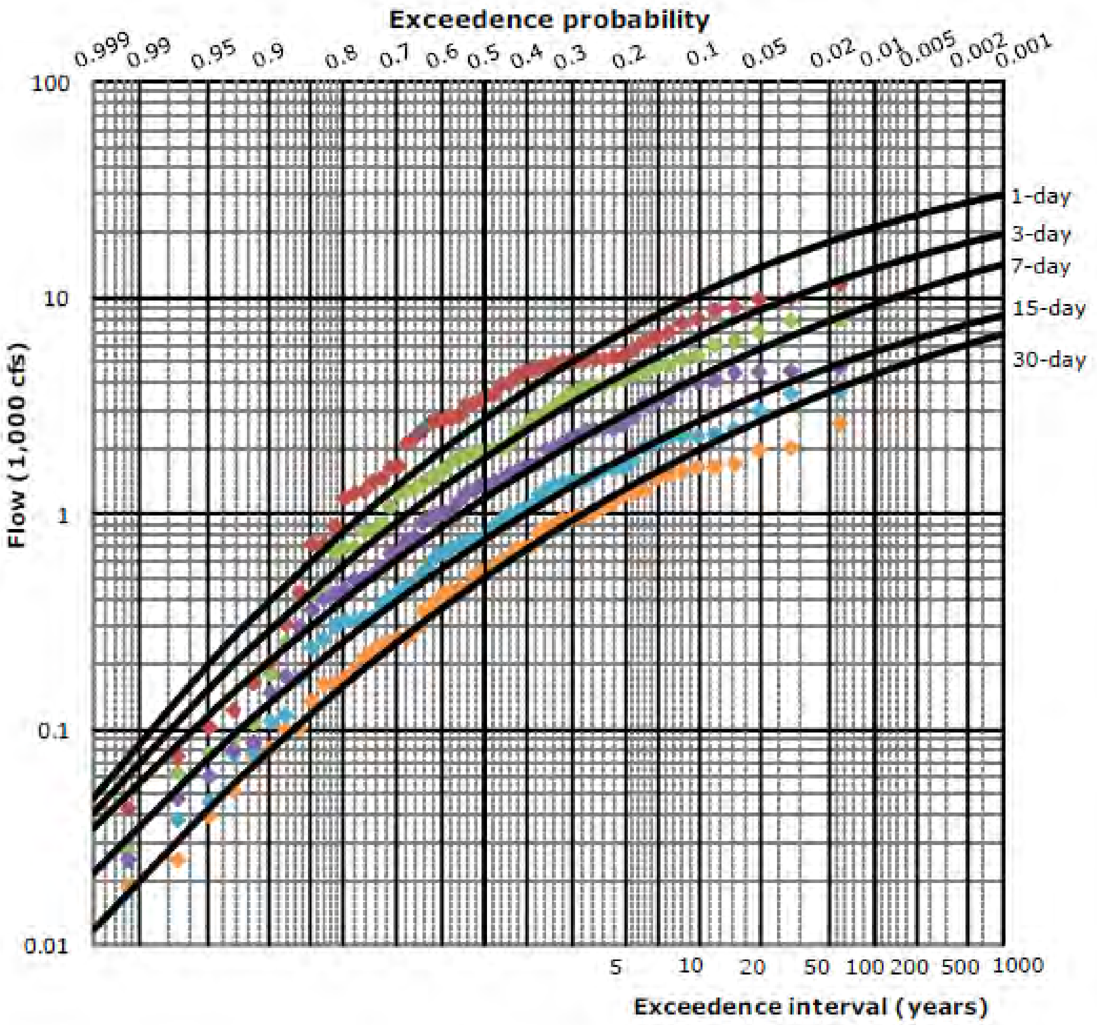
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.811	0.445	-0.692
1-day	3.321	0.507	-0.858
3-day	3.135	0.531	-0.812
7-day	2.970	0.538	-0.675
15-day	2.754	0.553	-0.733
30-day	2.561	0.556	-0.721

Notes:

- Median plotting positions.
- Drainage area: 212 sq. miles.
- Period of systematic record: 1951-2008.
  (Peak flow data intermittent 1952-2004).
- Record lengths
  Peak flows: 53 years.
  Volumes: 58 years.
- Regional skew values developed by USGS.
- Low outliers for volumes: 8 smallest events.

Plate 11. Analytical Unregulated Flow Frequency at Farmington Dam



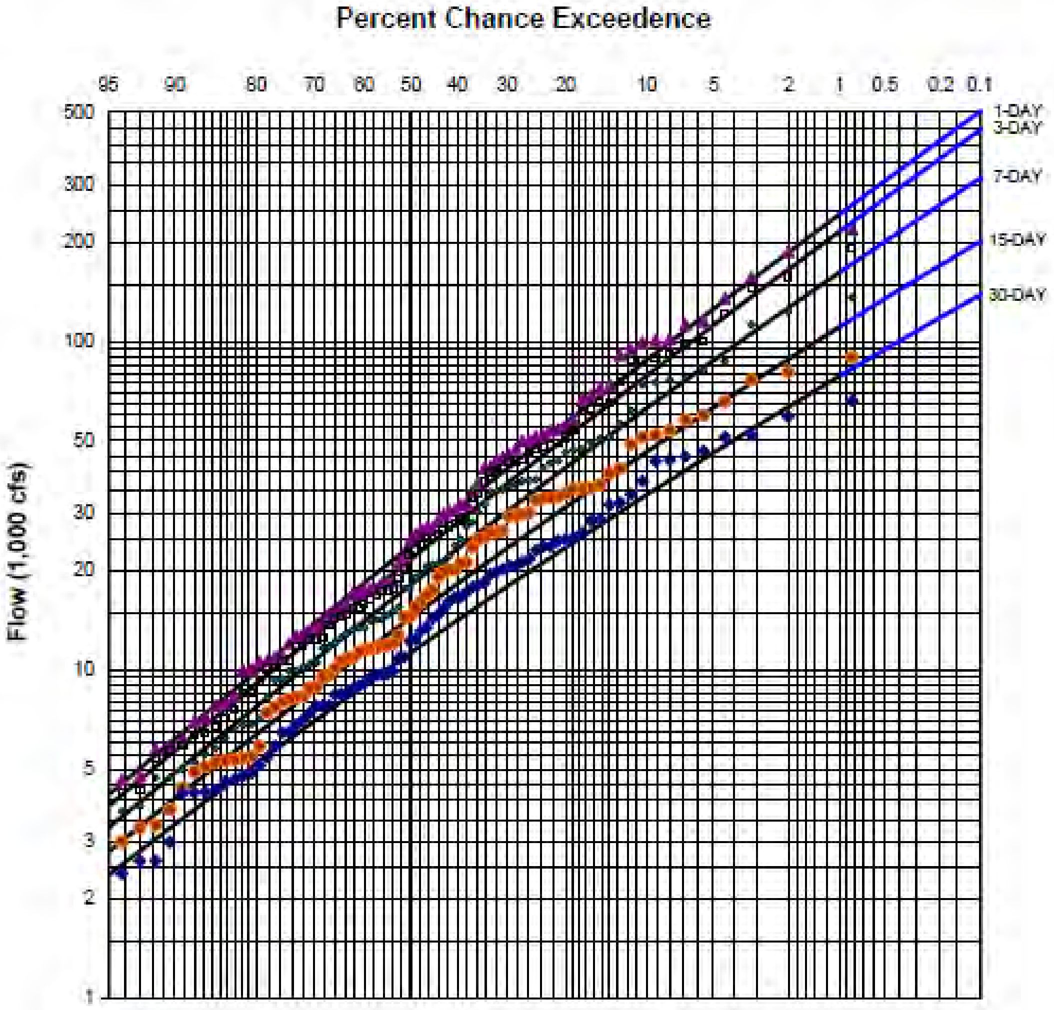
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.356	0.573	-0.849
3-day	3.186	0.545	-0.786
7-day	3.011	0.525	-0.670
15-day	2.815	0.523	-0.722
30-day	2.639	0.556	-0.695

Notes:

- Median plotting positions.
- Drainage area: 219 sq. miles.
- Period of systematic record: 1951-2008.
- Record length: 58 years.
- Regional skew values developed by USGS.
- Low outliers for 1-, 3, 7, and 15-day volumes: 7 smallest events.
- Low outliers for 30-day volumes: 6 smallest events.

Plate 12. Analytical Unregulated Flow Frequency at Littlejohn Creek at Farmington



ADOPTED STATISTICS:

	Mean	Std.Dev.	Skew
1-day	4.375	0.450	-0.1
3-day	4.333	0.445	-0.1
7-day	4.251	0.433	-0.2
15-day	4.148	0.412	-0.2
30-day	4.042	0.392	-0.2

NOTES:

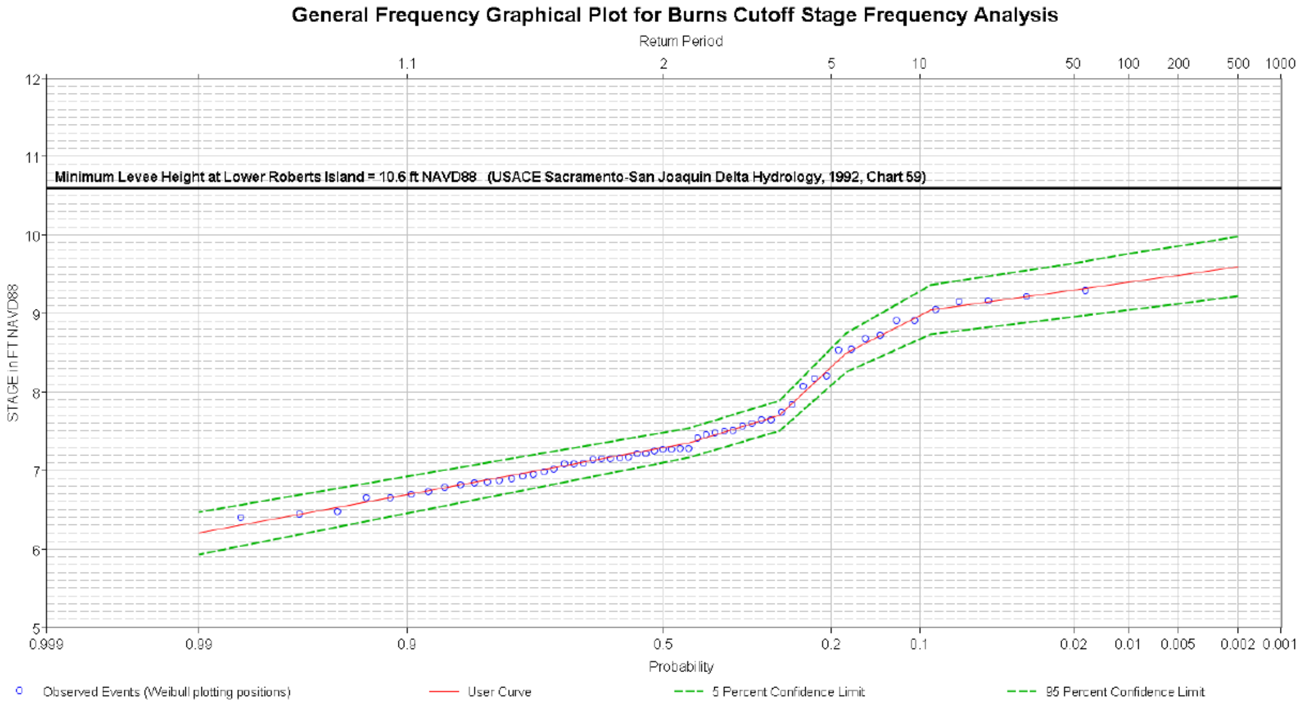
1. No overbank flow.
2. Median plotting positions.
3. Drainage area: 13,536 sq. mi.
4. Period of record: 1917-1998.

SACRAMENTO-SAN JOAQUIN COMPREHENSIVE STUDY
SAN JOAQUIN RIVER BASIN, CALIFORNIA

RAIN FLOOD FREQUENCY CURVES
SAN JOAQUIN RIVER NEAR VERNALIS
UNREGULATED CONDITIONS

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Plate 13. Analytical Unregulated Flow Frequency for the San Joaquin River at Vernalis



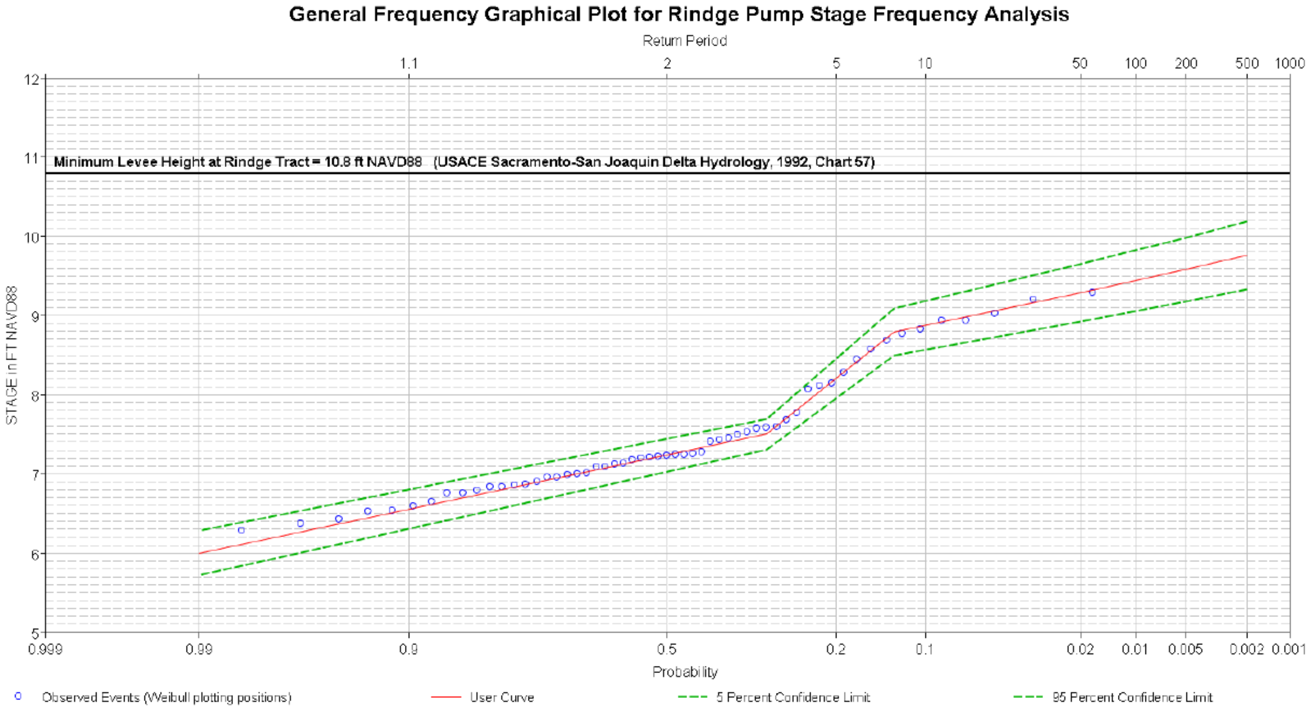


Plate 13b. General Frequency Graphical Plot Stage Frequency Analysis

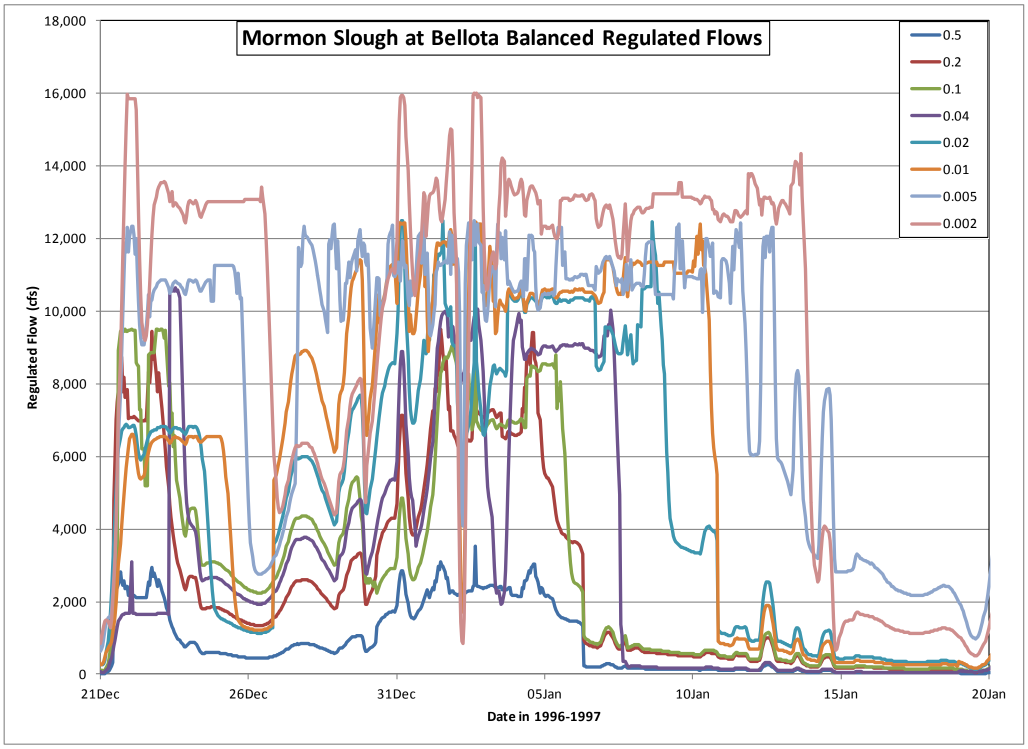


Plate 14. 0.5 to 0.002 AEP Regulated Hydrographs for the Calaveras River at Bellota

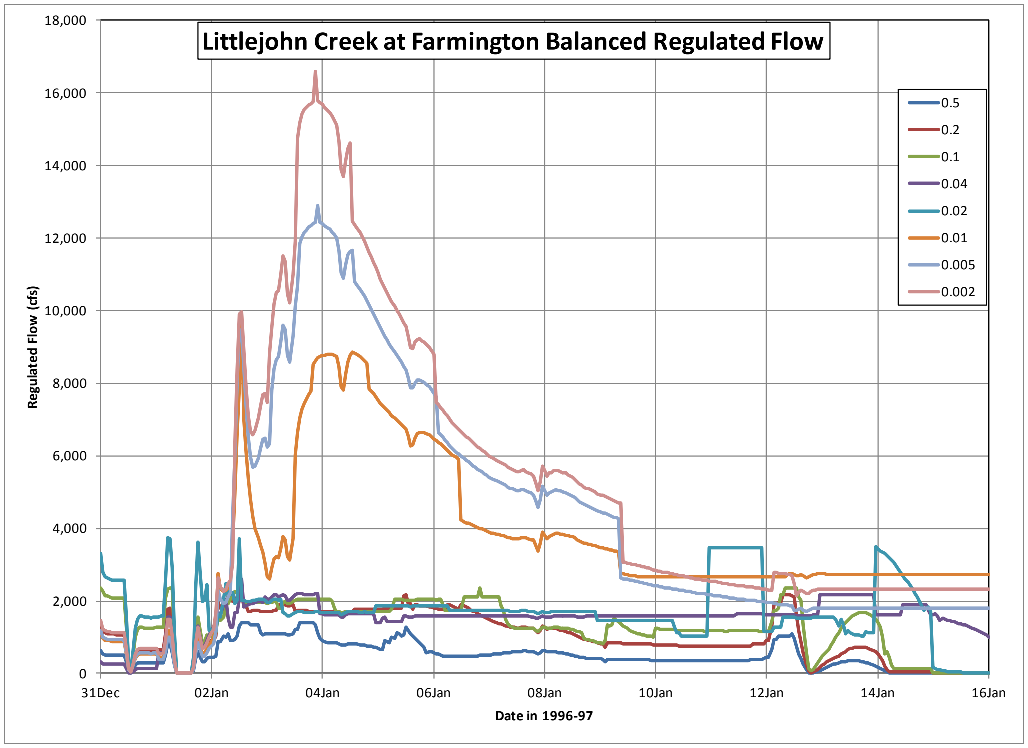


Plate 15. 0.5 to 0.002 AEP Regulated Hydrographs for Littlejohn Creek at Farmington

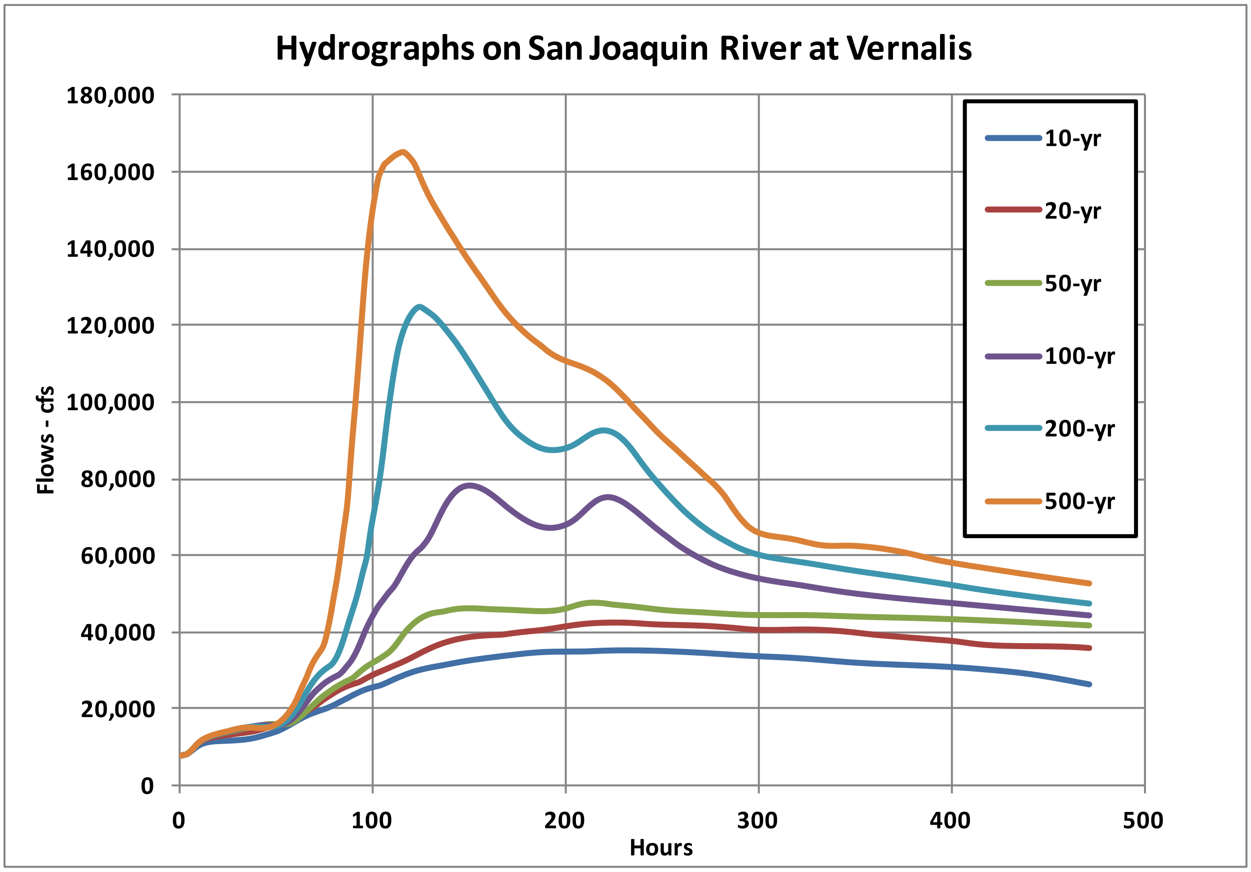
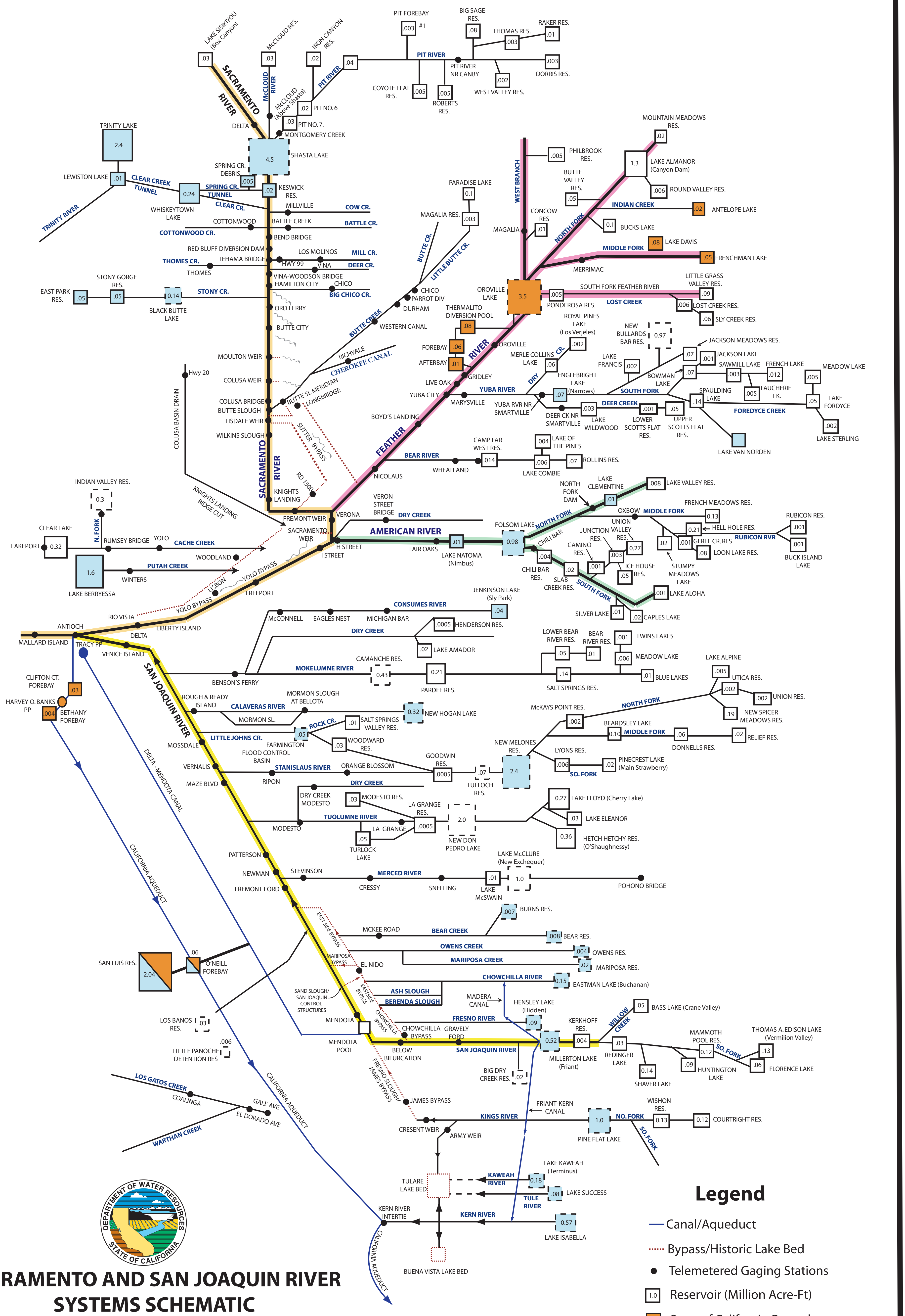


Plate 16. n-year Regulated Hydrographs for the San Joaquin River at Vernalis



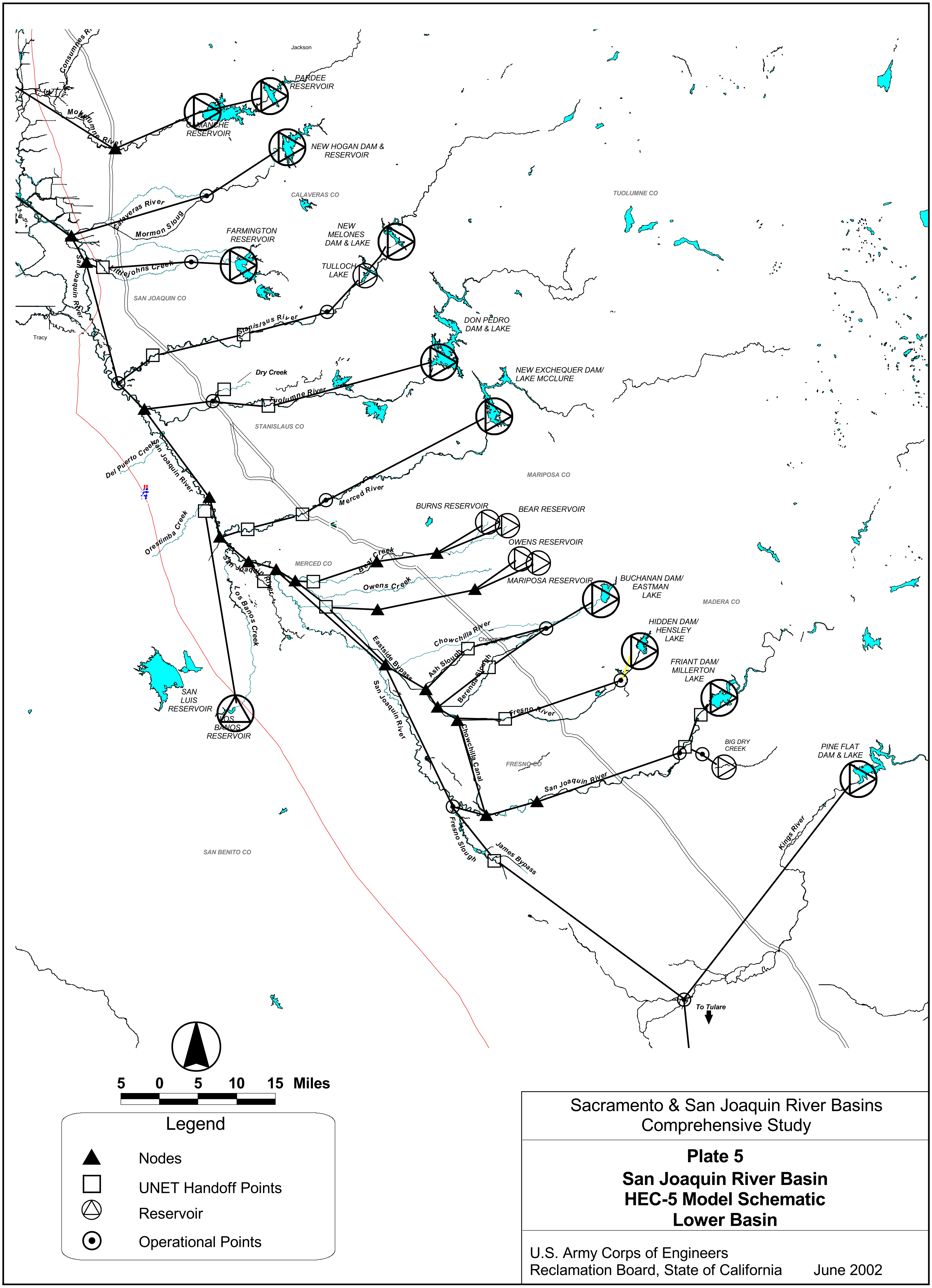
SACRAMENTO AND SAN JOAQUIN RIVER SYSTEMS SCHEMATIC

Department of Water Resources
Division of Flood Management
(April 2012)

Legend

- Canal/Aqueduct
- Bypass/Historic Lake Bed
- ● Telemetered Gaging Stations
- ☐ 1.0 Reservoir (Million Acre-Ft)
- 🟧 State of California Owned
- 🟦 Federally Owned
- ☐ Other Agency Owned
- ⬚ 1.0 Flood Control Reservoir (Million Acre-Ft)
- Floodway

Plate 17. San Joaquin River Basin Systems Schematic

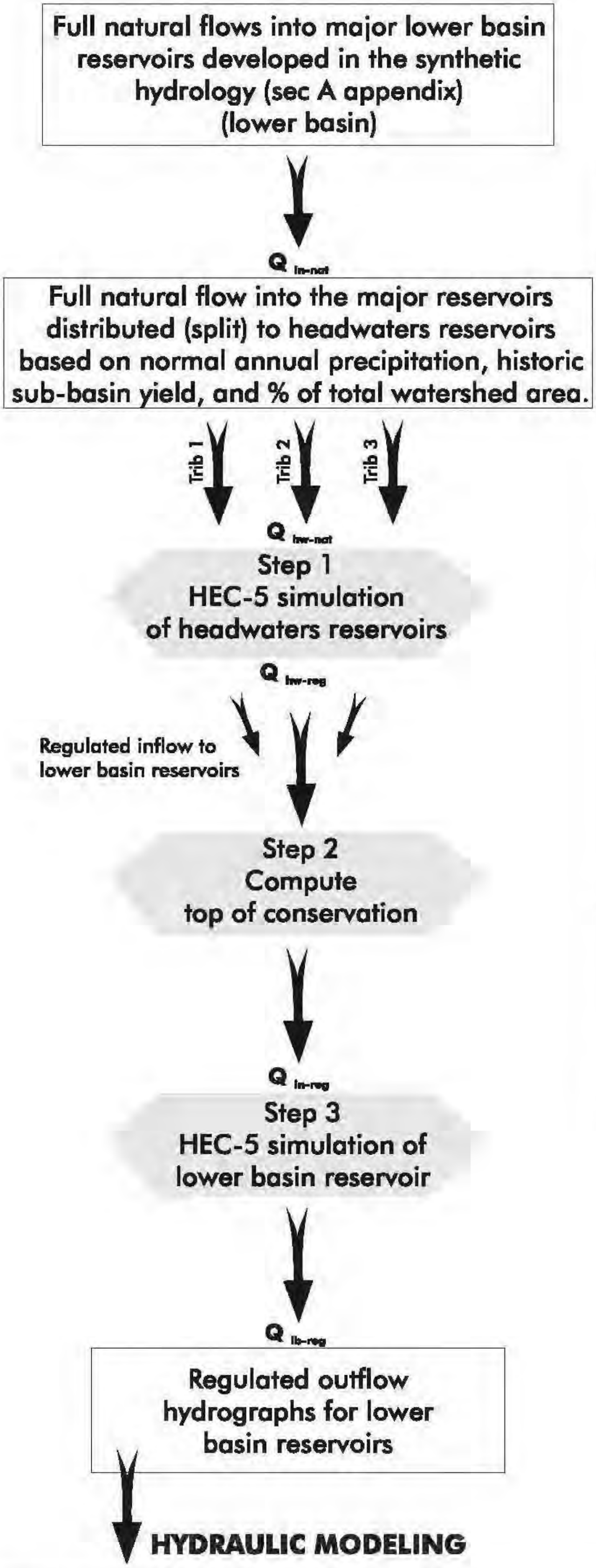


Sacramento & San Joaquin River Basins
Comprehensive Study

Plate 5
San Joaquin River Basin
HEC-5 Model Schematic
Lower Basin

U.S. Army Corps of Engineers
Reclamation Board, State of California June 2002

Plate 18. San Joaquin River Basin HEC-5 Model Schematic Lower Basin



Full natural flows into major lower basin reservoirs developed in the synthetic hydrology (sec A appendix) (lower basin)

$Q_{\text{In-nat}}$

Full natural flow into the major reservoirs distributed (split) to headwaters reservoirs based on normal annual precipitation, historic sub-basin yield, and % of total watershed area.

Trib 1 | Trib 2 | Trib 3

$Q_{\text{hw-nat}}$

Step 1
HEC-5 simulation
of headwaters reservoirs

$Q_{\text{hw-reg}}$

Regulated inflow to lower basin reservoirs

Step 2
Compute
top of conservation

$Q_{\text{In-reg}}$

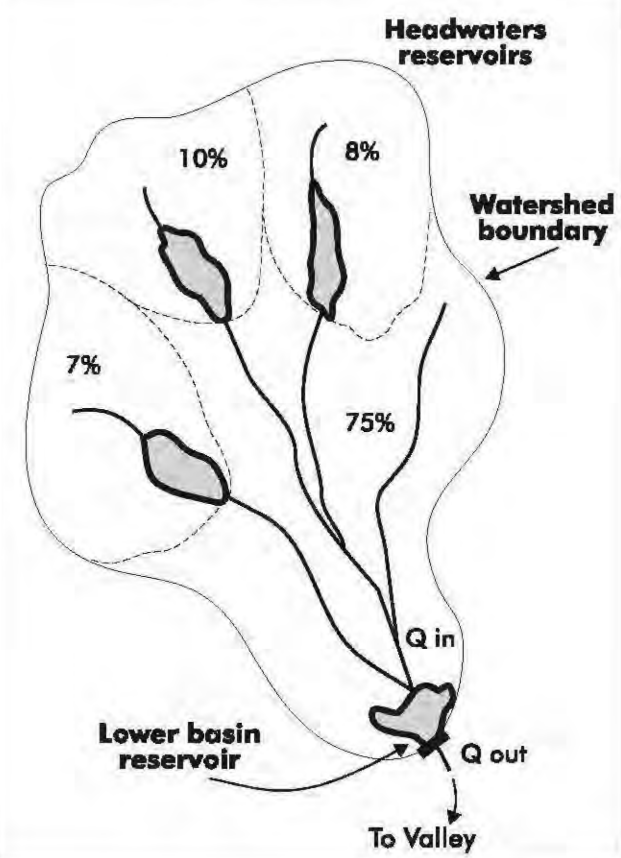
Step 3
HEC-5 simulation of
lower basin reservoir

$Q_{\text{lb-reg}}$

Regulated outflow hydrographs for lower basin reservoirs

HYDRAULIC MODELING

Flowchart Key



Nat – Natural flow not accounting for reservoir operation
Reg – Flow accounting for regulation by reservoir
Q – Flow
lb – Lower basin

Sacramento & San Joaquin River Basins
Comprehensive Study

Plate 6

PROCESS FLOWCHART

U.S. Army Corps of Engineers
Reclamation Board, State of California
June 2002

US_Army_COE-Sac\comprehensive study\F4 conference\appendix figs june 02.cdr

Plate 19. Comp Study Process Flowchart

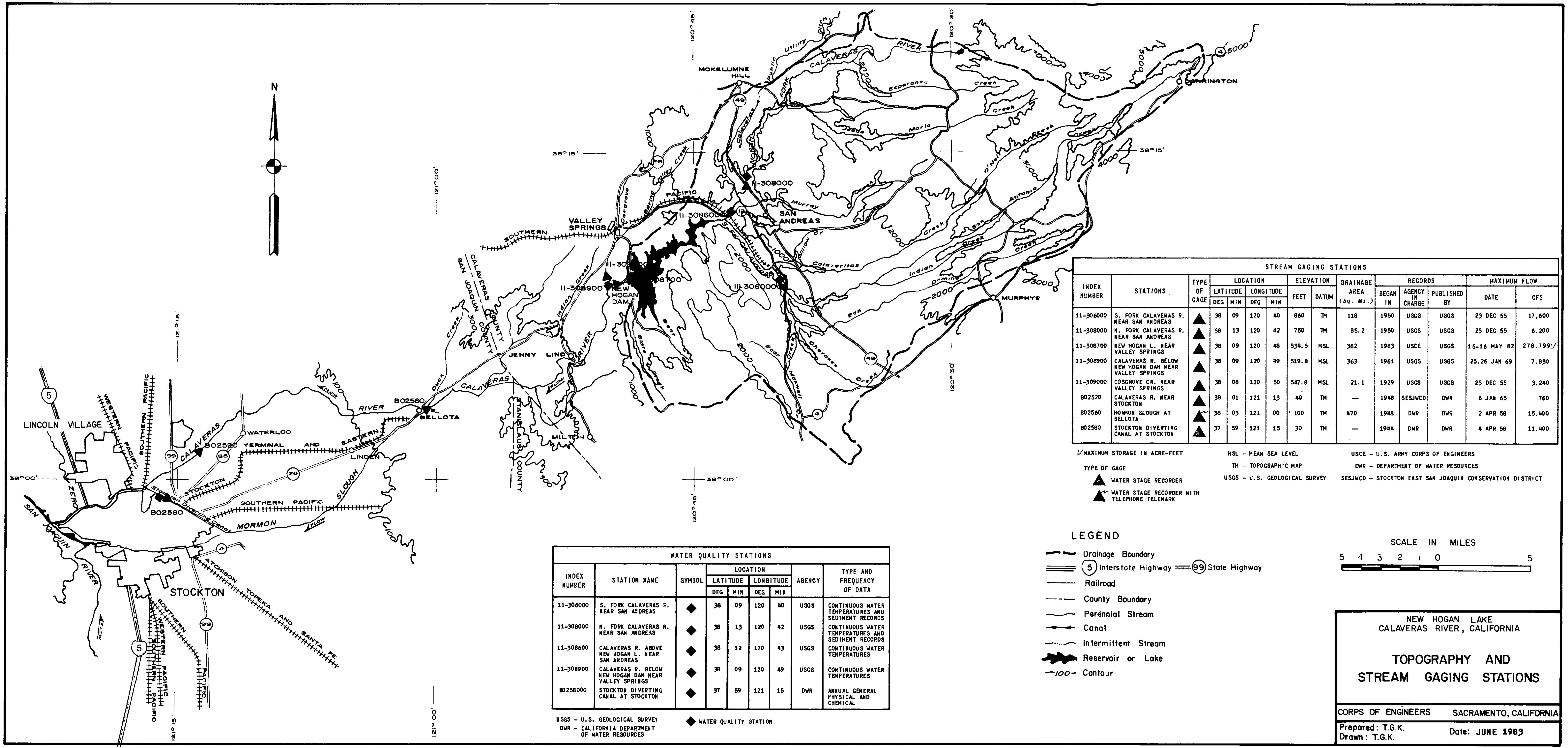
NEW HOGAN DAM AND LAKE CALAVERAS RIVER, CALIFORNIA

WATER CONTROL MANUAL

APPENDIX III TO
MASTER WATER CONTROL MANUAL
SAN JOAQUIN RIVER BASIN, CALIFORNIA

US ARMY CORPS
OF ENGINEERS
Sacramento District

JUNE 1983



STREAM GAGING STATIONS														
INDEX NUMBER	STATIONS	TYPE OF GAGE	LOCATION				ELEVATION		DRAINAGE AREA (Sq. Mi.)	RECORDS			MAXIMUM FLOW	
			LATITUDE		LONGITUDE		FEET	DATUM		BEGAN IN	AGENCY IN CHARGE	PUBLISHED BY	DATE	CFS
			DEG	MIN	DEG	MIN								
11-306000	S. FORK CALAVERAS R. NEAR SAN ANDREAS	▲	38	09	120	40	860	TM	118	1950	USGS	USGS	23 DEC 55	17,600
11-308000	N. FORK CALAVERAS R. NEAR SAN ANDREAS	▲	38	13	120	42	750	TM	85.2	1950	USGS	USGS	23 DEC 55	6,200
11-308700	NEW HOGAN L. NEAR VALLEY SPRINGS	▲	38	09	120	48	534.5	MSL	362	1963	USCE	USGS	15-16 MAY 82	278,799[1]
11-308900	CALAVERAS R. BELOW NEW HOGAN DAM NEAR VALLEY SPRINGS	▲	38	09	120	49	519.8	MSL	363	1961	USGS	USGS	25,26 JAN 69	7,830
11-309000	COSGROVE CR. NEAR VALLEY SPRINGS	▲	38	08	120	50	547.8	MSL	21.1	1929	USGS	USGS	23 DEC 55	3,240
B02520	CALAVERAS R. NEAR STOCKTON	▲	38	01	121	13	40	TM	—	1948	SESJWCD	DWR	6 JAN 65	760
B02560	MORMON SLOUGH AT BELLOTA	▲	38	03	121	00	100	TM	470	1948	DWR	DWR	2 APR 58	15,400
B02580	STOCKTON DIVERTING CANAL AT STOCKTON	▲	37	59	121	15	30	TM	—	1944	DWR	DWR	4 APR 58	11,400

[1] MAXIMUM STORAGE IN ACRE-FEET

TYPE OF GAGE

▲ WATER STAGE RECORDER

▲ WATER STAGE RECORDER WITH TELEPHONE TELEMARK

MSL - MEAN SEA LEVEL
TM - TOPOGRAPHIC MAP
USGS - U.S. GEOLOGICAL SURVEY
USCE - U.S. ARMY CORPS OF ENGINEERS
DWR - DEPARTMENT OF WATER RESOURCES
SESJWCD - STOCKTON EAST SAN JOAQUIN CONSERVATION DISTRICT

WATER QUALITY STATIONS								
INDEX NUMBER	STATION NAME	SYMBOL	LOCATION				AGENCY	TYPE AND FREQUENCY OF DATA
			LATITUDE		LONGITUDE			
			DEG	MIN	DEG	MIN		
11-306000	S. FORK CALAVERAS R. NEAR SAN ANDREAS	◆	38	09	120	40	USGS	CONTINUOUS WATER TEMPERATURES AND SEDIMENT RECORDS
11-308000	N. FORK CALAVERAS R. NEAR SAN ANDREAS	◆	38	13	120	42	USGS	CONTINUOUS WATER TEMPERATURES AND SEDIMENT RECORDS
11-308600	CALAVERAS R. ABOVE NEW HOGAN L. NEAR SAN ANDREAS	◆	38	12	120	43	USGS	CONTINUOUS WATER TEMPERATURES
11-308900	CALAVERAS R. BELOW NEW HOGAN DAM NEAR VALLEY SPRINGS	◆	38	09	120	49	USGS	CONTINUOUS WATER TEMPERATURES
B0258000	STOCKTON DIVERTING CANAL AT STOCKTON	◆	37	59	121	15	DWR	ANNUAL GENERAL PHYSICAL AND CHEMICAL

USGS - U.S. GEOLOGICAL SURVEY
DWR - CALIFORNIA DEPARTMENT OF WATER RESOURCES

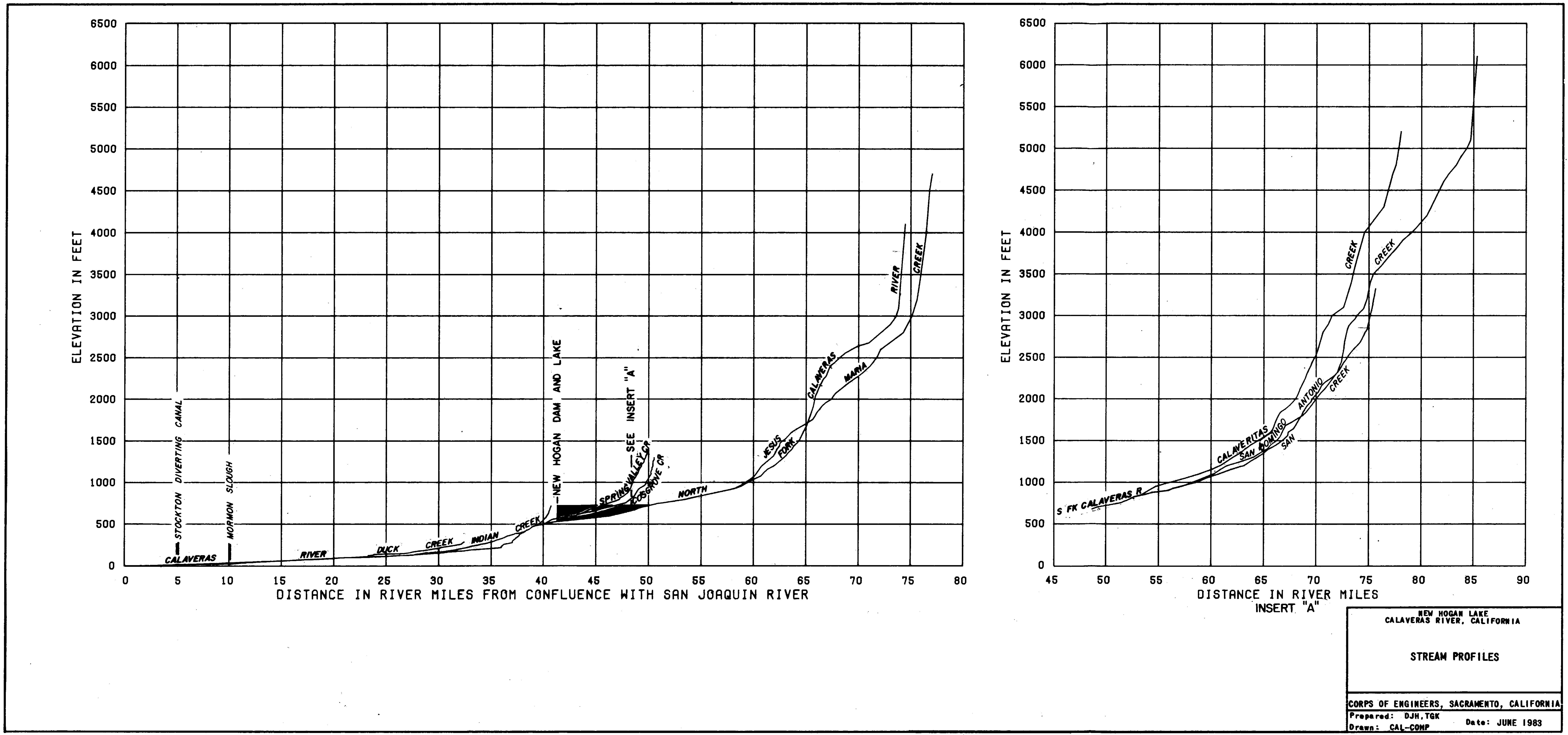
◆ WATER QUALITY STATION

NEW HOGAN LAKE
CALAVERAS RIVER, CALIFORNIA

TOPOGRAPHY AND STREAM GAGING STATIONS

CORPS OF ENGINEERS SACRAMENTO, CALIFORNIA
Prepared: T.G.K.
Drawn: T.G.K.
Date: JUNE 1983

Plate 20. New Hogan Dam Topography and Stream Gage Stations



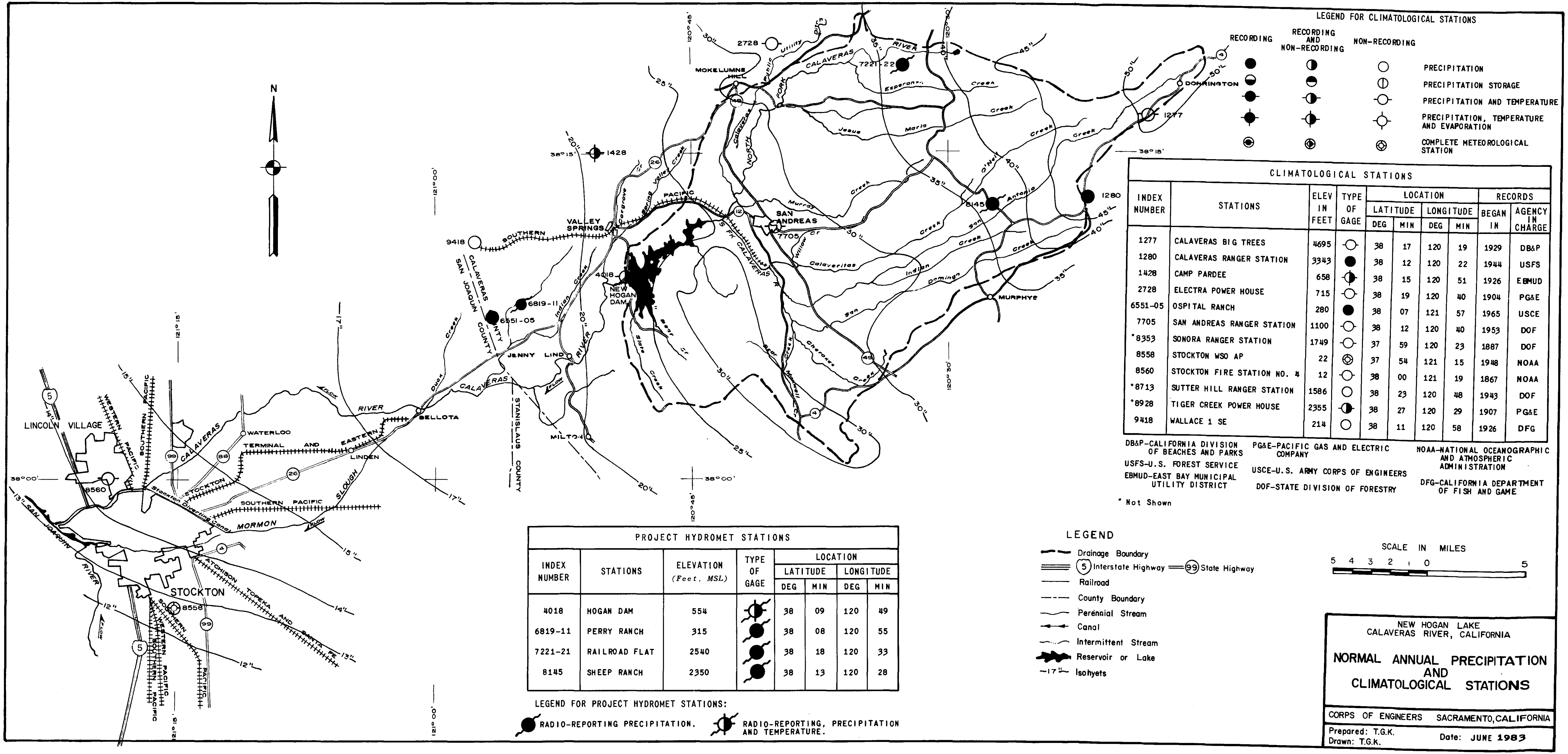
NEW HOGAN LAKE
CALAVERAS RIVER, CALIFORNIA

STREAM PROFILES

CORPS OF ENGINEERS, SACRAMENTO, CALIFORNIA
Prepared: DJH,TGK
Drawn: CAL-COMP
Date: JUNE 1983

Plate 21. New Hogan Dam Stream Profiles

PLATE 11



INDEX NUMBER	STATIONS	ELEV IN FEET	TYPE OF GAGE	LOCATION				RECORDS	
				LATITUDE		LONGITUDE			
				DEG	MIN	DEG	MIN	BEGAN IN	AGENCY IN CHARGE
1277	CALAVERAS BIG TREES	4695	-○-	38	17	120	19	1929	DB&P
1280	CALAVERAS RANGER STATION	3343	●	38	12	120	22	1944	USFS
1428	CAMP PARDEE	658	-◑-	38	15	120	51	1926	EBMUD
2728	ELECTRA POWER HOUSE	715	-○-	38	19	120	40	1904	PG&E
6551-05	OSPITAL RANCH	280	●	38	07	121	57	1965	USCE
7705	SAN ANDREAS RANGER STATION	1100	-○-	38	12	120	40	1953	DOF
*8353	SONORA RANGER STATION	1749	-○-	37	59	120	23	1887	DOF
8558	STOCKTON WSO AP	22	◎	37	54	121	15	1948	NOAA
8560	STOCKTON FIRE STATION NO. 4	12	-○-	38	00	121	19	1867	NOAA
*8713	SUTTER HILL RANGER STATION	1586	○	38	23	120	48	1943	DOF
*8928	TIGER CREEK POWER HOUSE	2355	-◑-	38	27	120	29	1907	PG&E
9418	WALLACE 1 SE	214	○	38	11	120	58	1926	DFG

DB&P-CALIFORNIA DIVISION OF BEACHES AND PARKS

USFS-U.S. FOREST SERVICE

EBMUD-EAST BAY MUNICIPAL UTILITY DISTRICT

PG&E-PACIFIC GAS AND ELECTRIC COMPANY

USCE-U.S. ARMY CORPS OF ENGINEERS

DOF-STATE DIVISION OF FORESTRY

NOAA-NATIONAL OCEANOGRAPHIC AND ATMOSPHERIC ADMINISTRATION

DFG-CALIFORNIA DEPARTMENT OF FISH AND GAME

* Not Shown

INDEX NUMBER	STATIONS	ELEVATION (Feet, MSL)	TYPE OF GAGE	LOCATION			
				LATITUDE		LONGITUDE	
				DEG	MIN	DEG	MIN
4018	HOGAN DAM	554	◑	38	09	120	49
6819-11	PERRY RANCH	315	●	38	08	120	55
7221-21	RAILROAD FLAT	2540	●	38	18	120	33
8145	SHEEP RANCH	2350	●	38	13	120	28

LEGEND FOR PROJECT HYDROMET STATIONS:

● RADIO-REPORTING PRECIPITATION.

◑ RADIO-REPORTING, PRECIPITATION AND TEMPERATURE.

NEW HOGAN LAKE
CALAVERAS RIVER, CALIFORNIA

NORMAL ANNUAL PRECIPITATION AND CLIMATOLOGICAL STATIONS

CORPS OF ENGINEERS SACRAMENTO, CALIFORNIA
Prepared: T.G.K.
Drawn: T.G.K.
Date: JUNE 1983

Plate 22. New Hogan Dam NAP and Climate Stations

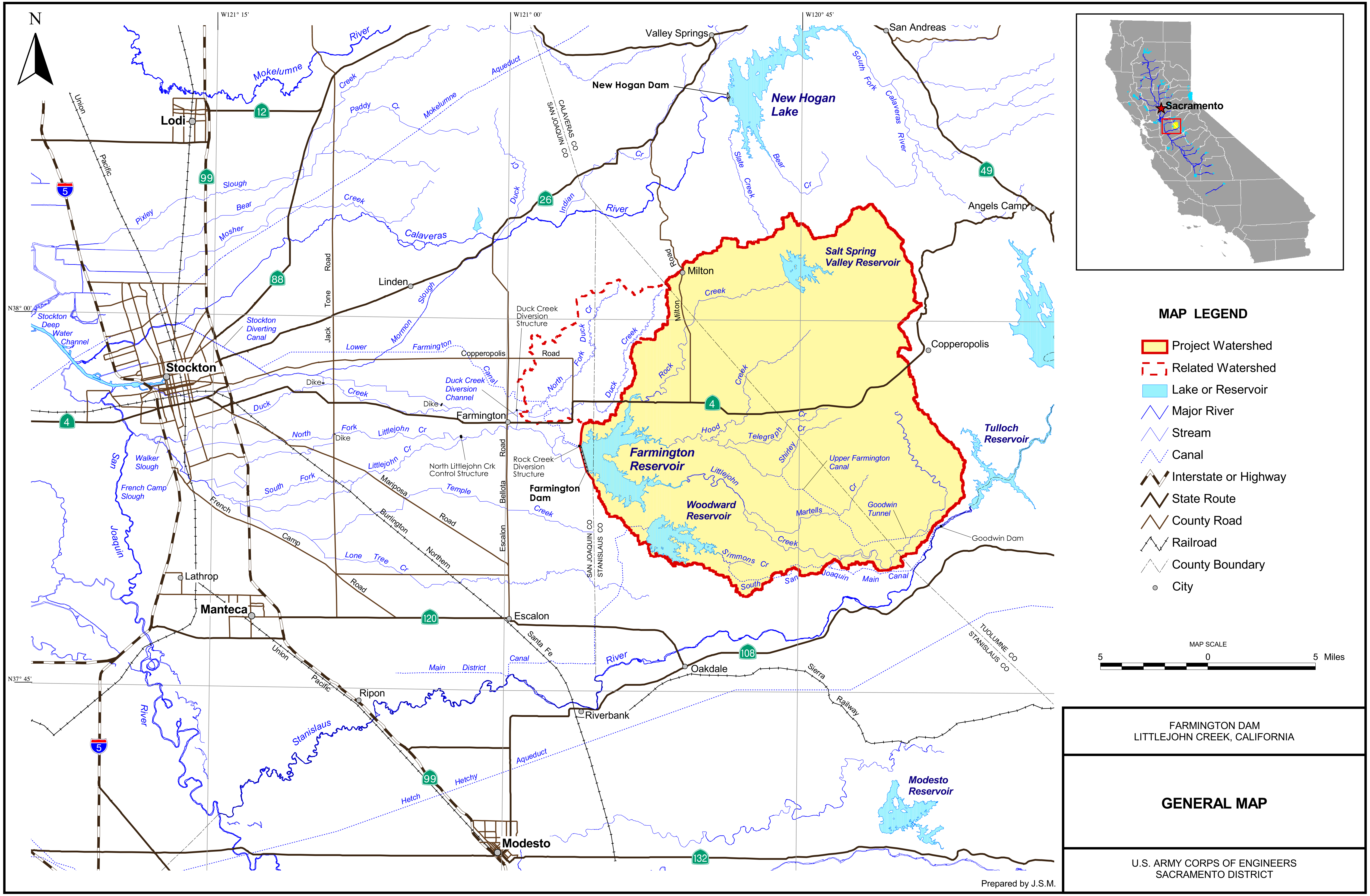
FARMINGTON
DAM AND RESERVOIR
LITTLEJOHN CREEK, CALIFORNIA

WATER CONTROL MANUAL

APPENDIX IV TO
MASTER WATER CONTROL MANUAL
SAN JOAQUIN RIVER BASIN, CALIFORNIA

US ARMY CORPS
OF ENGINEERS
Sacramento District

DECEMBER 1952
REVISED DECEMBER 2004



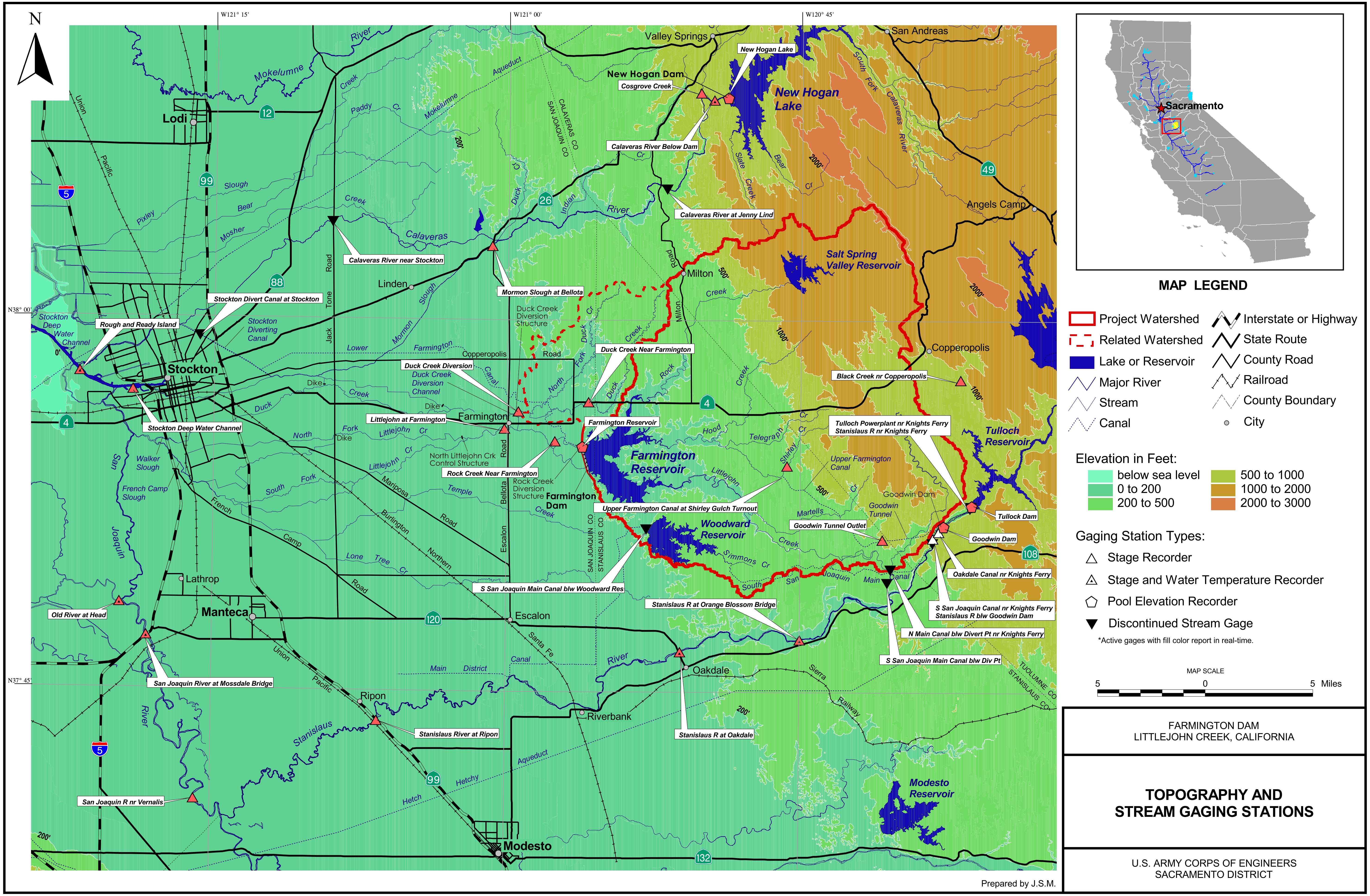
FARMINGTON DAM
LITTLEJOHN CREEK, CALIFORNIA

GENERAL MAP

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Revised Dec 2004 SHEET 1 OF 2 PLATE 2-1

Plate 23. Farmington Dam General Map



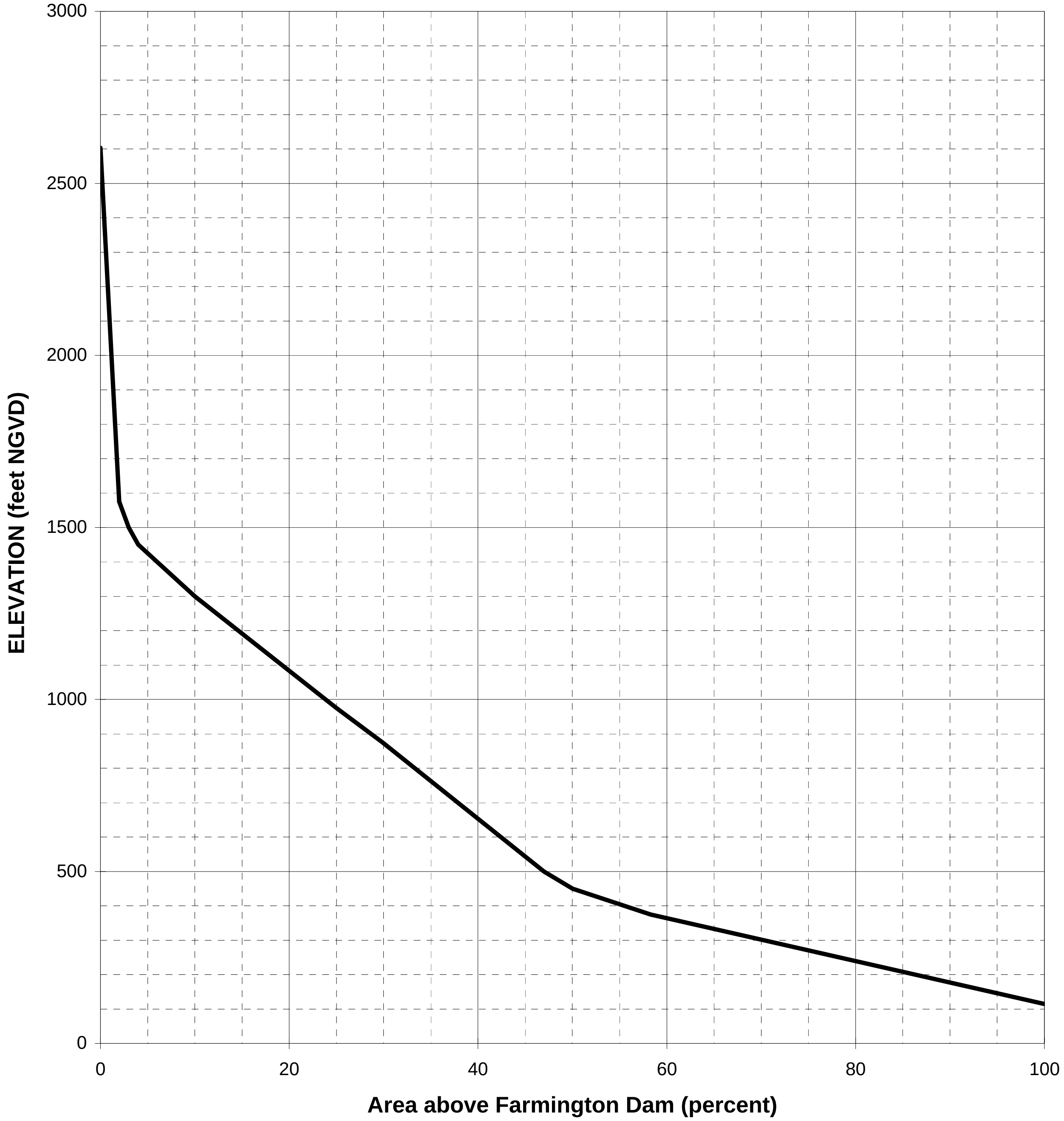
FARMINGTON DAM
LITTLEJOHN CREEK, CALIFORNIA

**TOPOGRAPHY AND
STREAM GAGING STATIONS**

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Revised Dec 2004 SHEET 1 OF 2 PLATE 4-1

Plate 24. Farmington Dam Topography and Stream Gaging Stations



NOTES: 1. Area: 212 square miles

2. Dam site elevation: 115 feet

Prepared by MVB

FARMINGTON DAM
LITTLEJOHN CREEK, CALIFORNIA

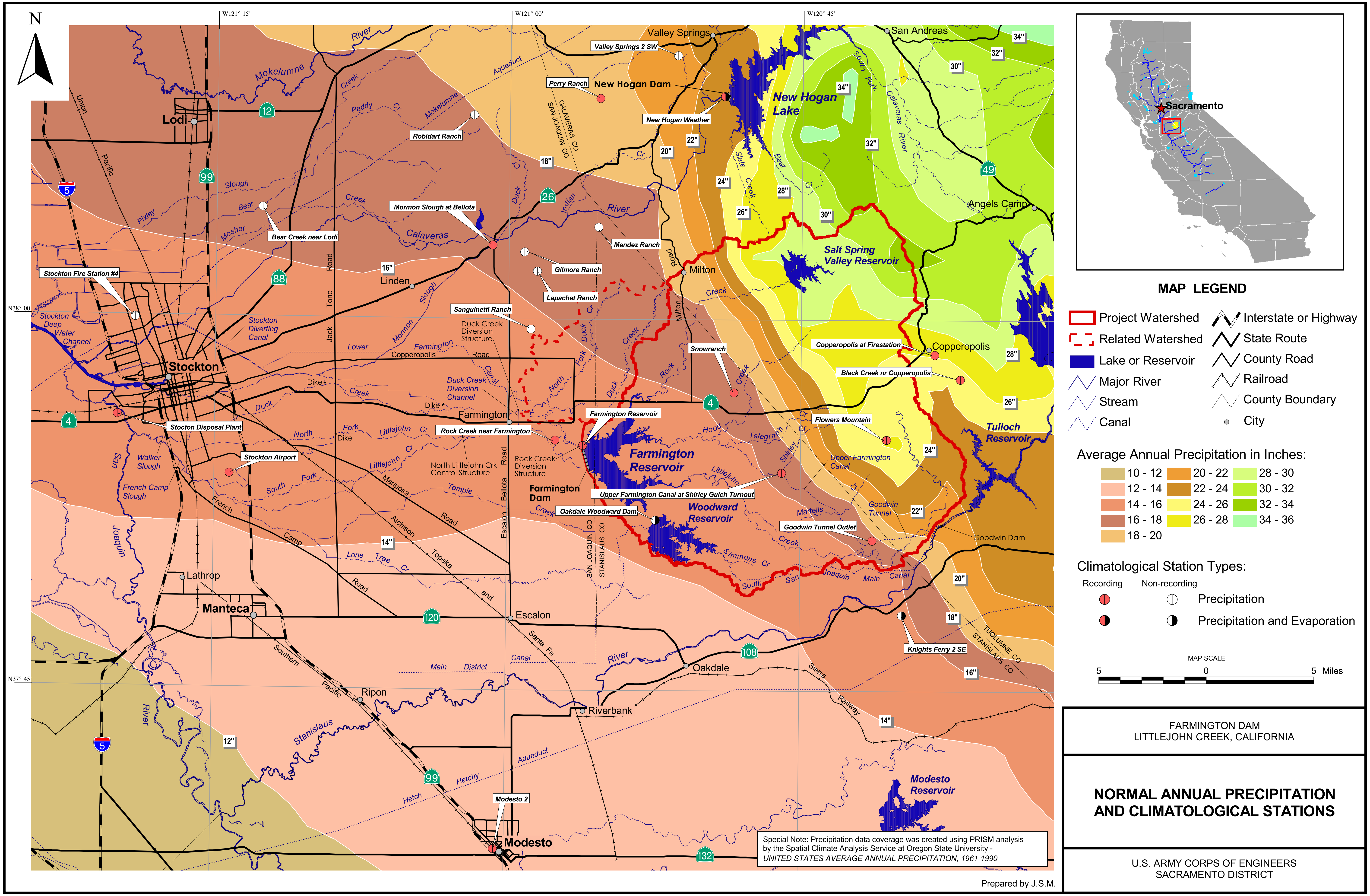
AREA-ELEVATION CURVE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Revised Dec 2004

PLATE 4-3

Plate 25. Farmington Dam Area-Elevation Curve



Special Note: Precipitation data coverage was created using PRISM analysis by the Spatial Climate Analysis Service at Oregon State University - *UNITED STATES AVERAGE ANNUAL PRECIPITATION, 1961-1990*

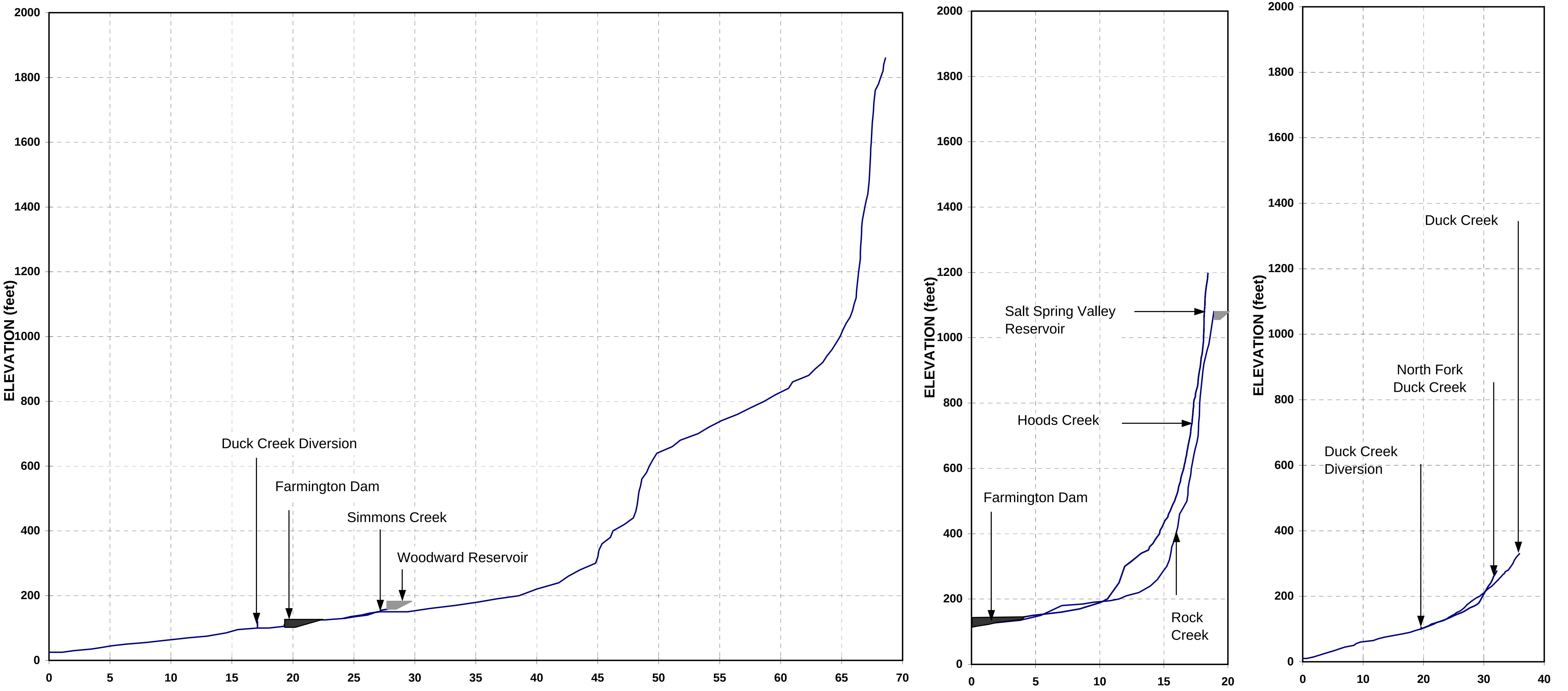
FARMINGTON DAM
LITTLEJOHN CREEK, CALIFORNIA

NORMAL ANNUAL PRECIPITATION AND CLIMATOLOGICAL STATIONS

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Revised Dec 2004 SHEET 1 OF 2 PLATE 4-5

Plate 26. Farmington Dam NAP and Climate Stations



Distance in river miles from confluence with French Camp Slough
LITTLEJOHN CREEK

Distance in river miles from Farmington Dam
ROCK CREEK

Distance in river miles from confluence with Walker Slough
DUCK CREEK

FARMINGTON DAM
LITTLEJOHN CREEK, CALIFORNIA

STREAM PROFILES

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

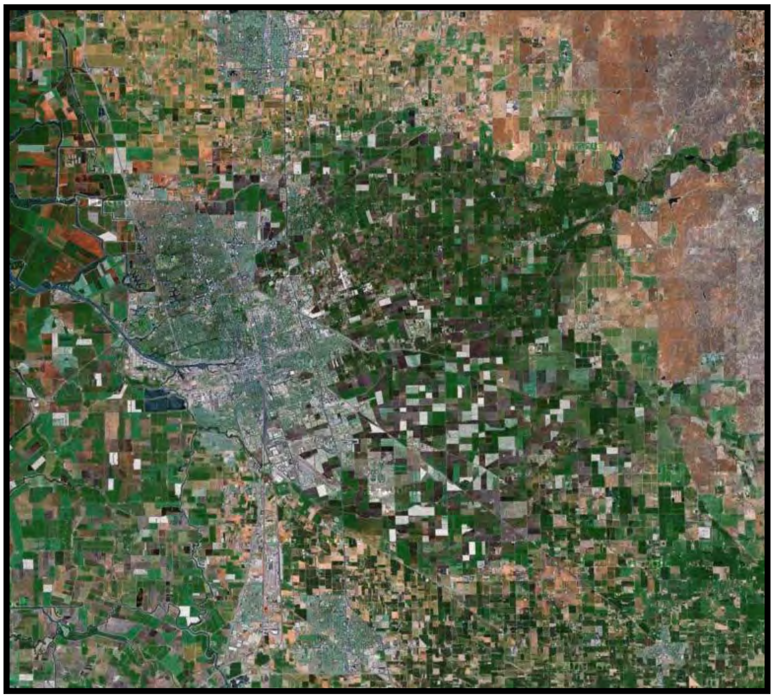
Prepared by MVB

Revised Dec 2004

PLATE 4-2

Plate 27. Farmington Dam Stream Profiles

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY



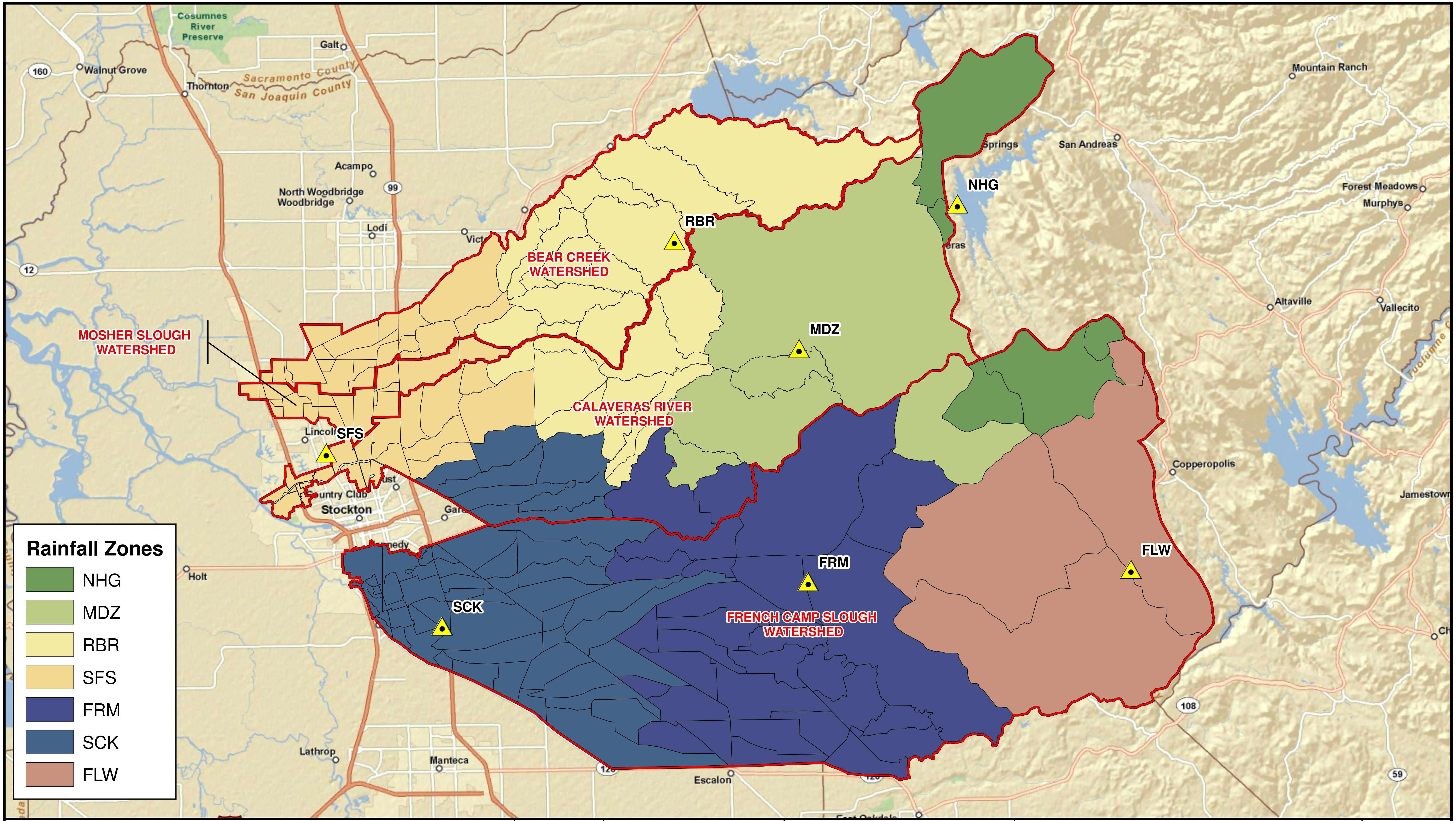
F3 HYDROLOGY APPENDIX

JULY 30, 2012

PREPARED BY:

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260, Folsom, CA 95630
(916)608-2212



□ Watershed Boundary
□ Subbasin Boundary
▲ Precipitation Gage

N

0 5 Miles
1 : 250,000

APRIL 25, 2011

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

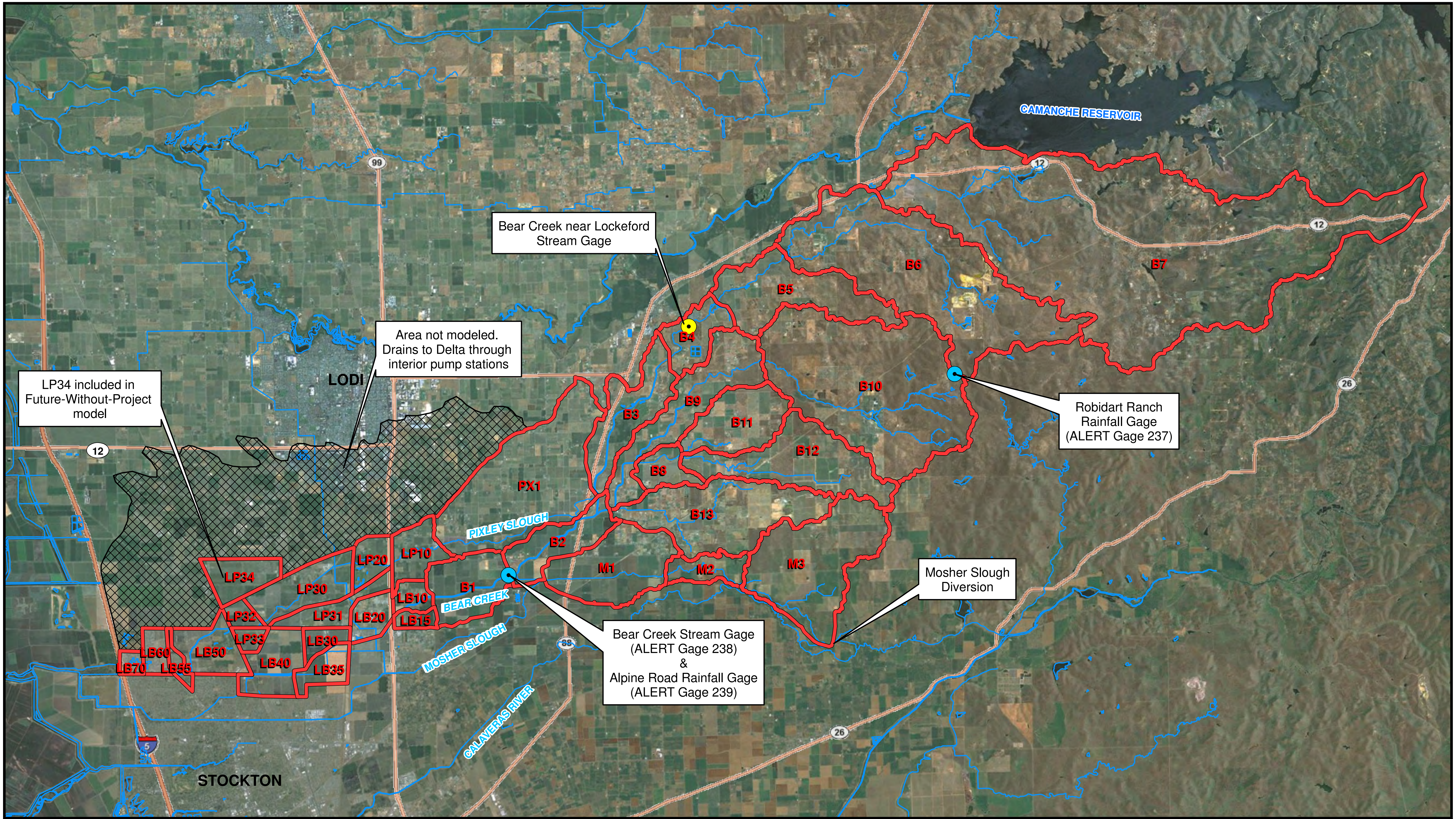
LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

LSJRFS Rainfall Zones

FIGURE

2-1

Plate 28. LSJRFS Rainfall Zones



N

0 0.5 1 2 Miles

1 inch = 2 miles

DECEMBER 8, 2011

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

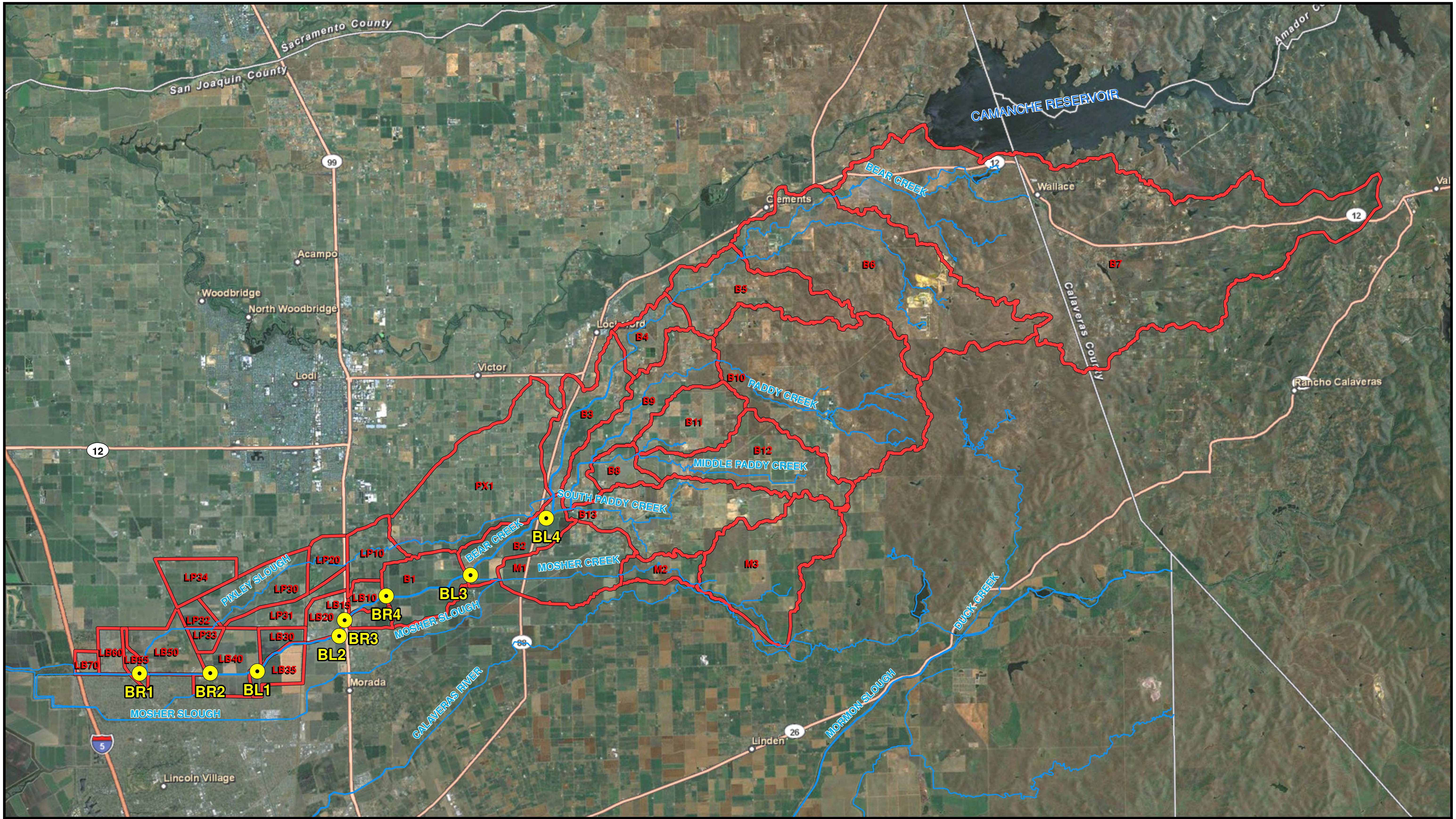
SAN JOAQUIN AREA FLOOD CONTROL AGENCY

BEAR CREEK HEC-HMS SUBBASINS

FIGURE

3-2

Plate 29. Bear Creek HEC-HMS Subbasins



● LSJRFS Index Point

🟥 Subshed Boundary

N

0 0.5 1 2 Miles

1 inch = 2 miles

JUNE 20, 2012

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

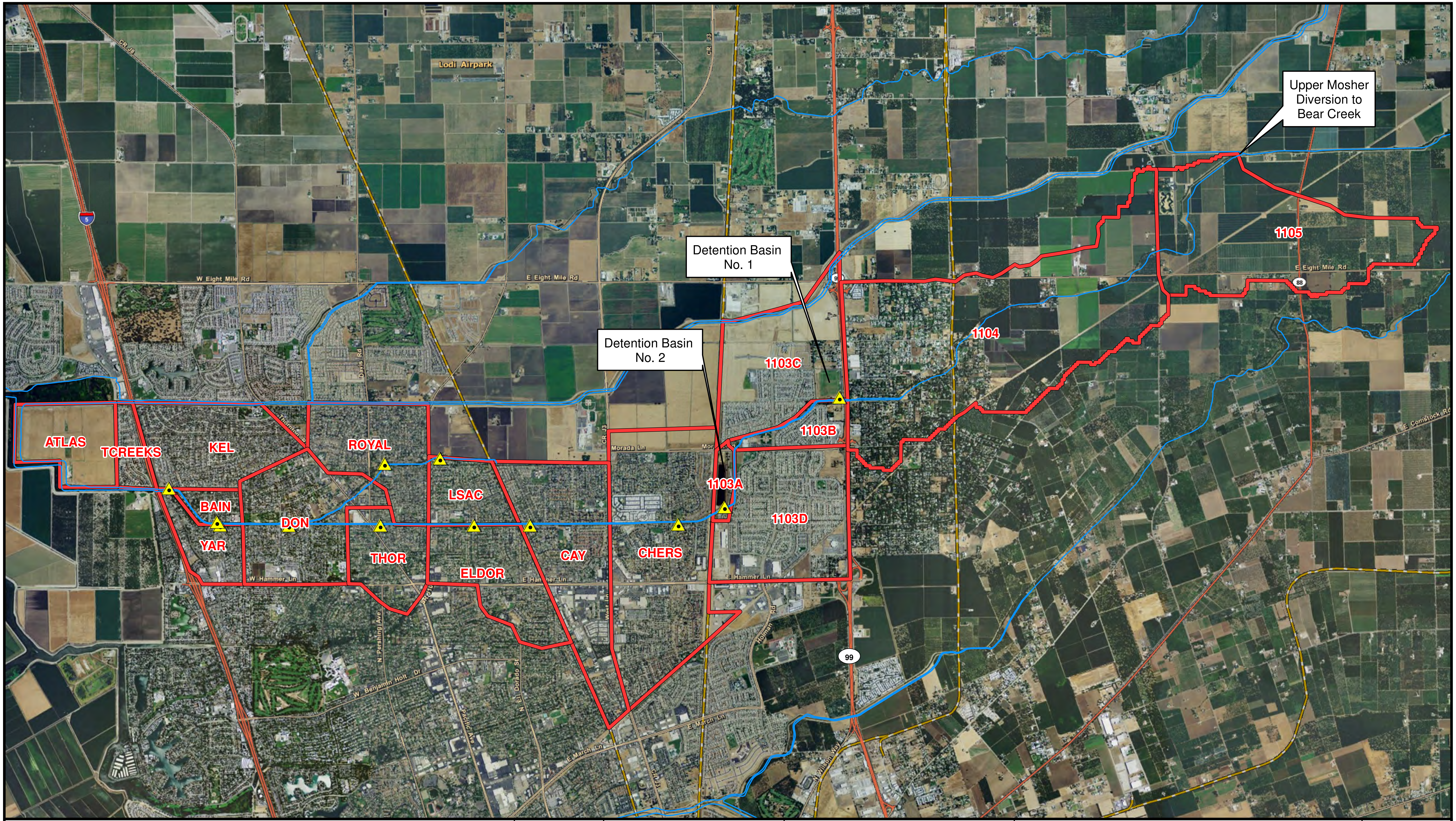
LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

BEAR CREEK WATERSHED
INDEX POINTS

FIGURE

3-12

Plate 30. Bear Creek Watershed Index Points



🟥 Subbasin Boundary

▲ Existing Pump Station

N

0 1,000 2,000 4,000 Feet

1 inch = 4,000 feet

AUGUST 20, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

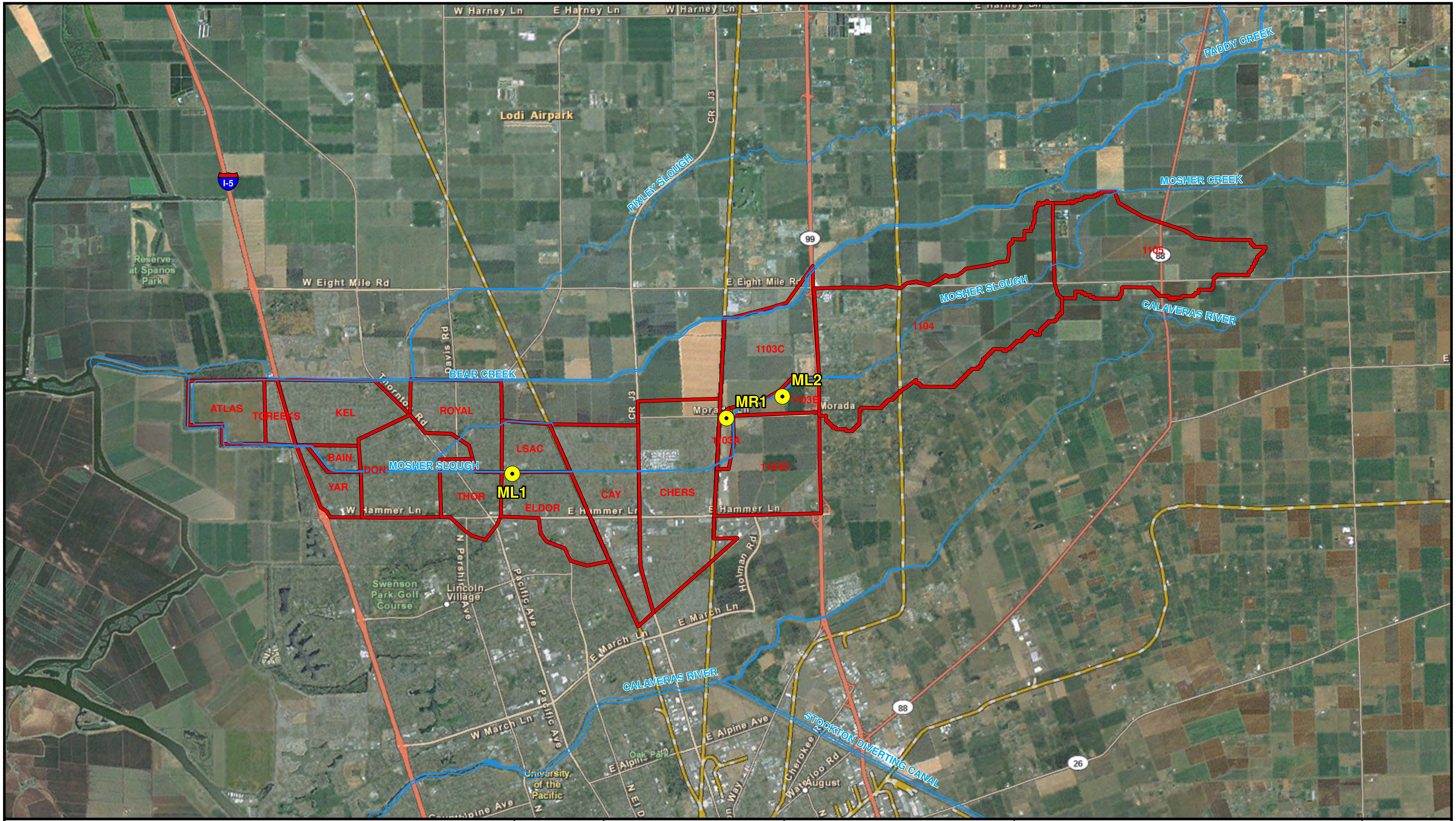
SAN JOAQUIN AREA FLOOD CONTROL AGENCY

MOSHER SLOUGH
HEC-HMS SUBBASINS

FIGURE

4-2

Plate 31. Mosher Slough HEC-HMS Subbasins



⊙ LSJRFS Index Point

□ Subshed Boundary

N

0 0.25 0.5 1 Miles

1 inch = 1 mile

JUNE 20, 2012

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

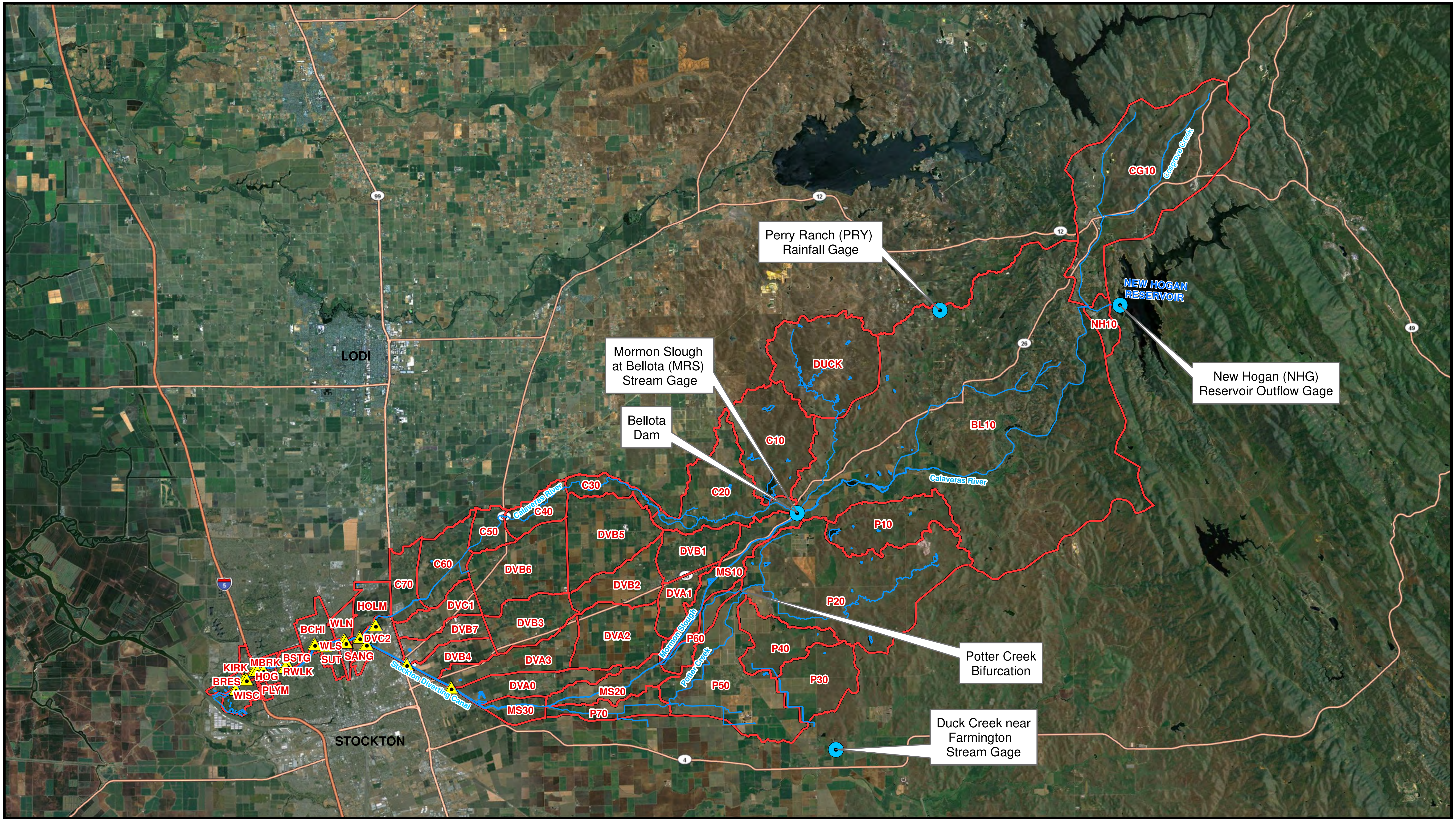
LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

MOSHER SLOUGH WATERSHED INDEX POINTS

FIGURE

4-10

Plate 32. Mosher Slough Watershed Index Points



☐ Subbasin Boundary

▲ Existing Pump Station

N

0 1.5 3 Miles

1 inch = 3 miles

SEPTEMBER 21, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

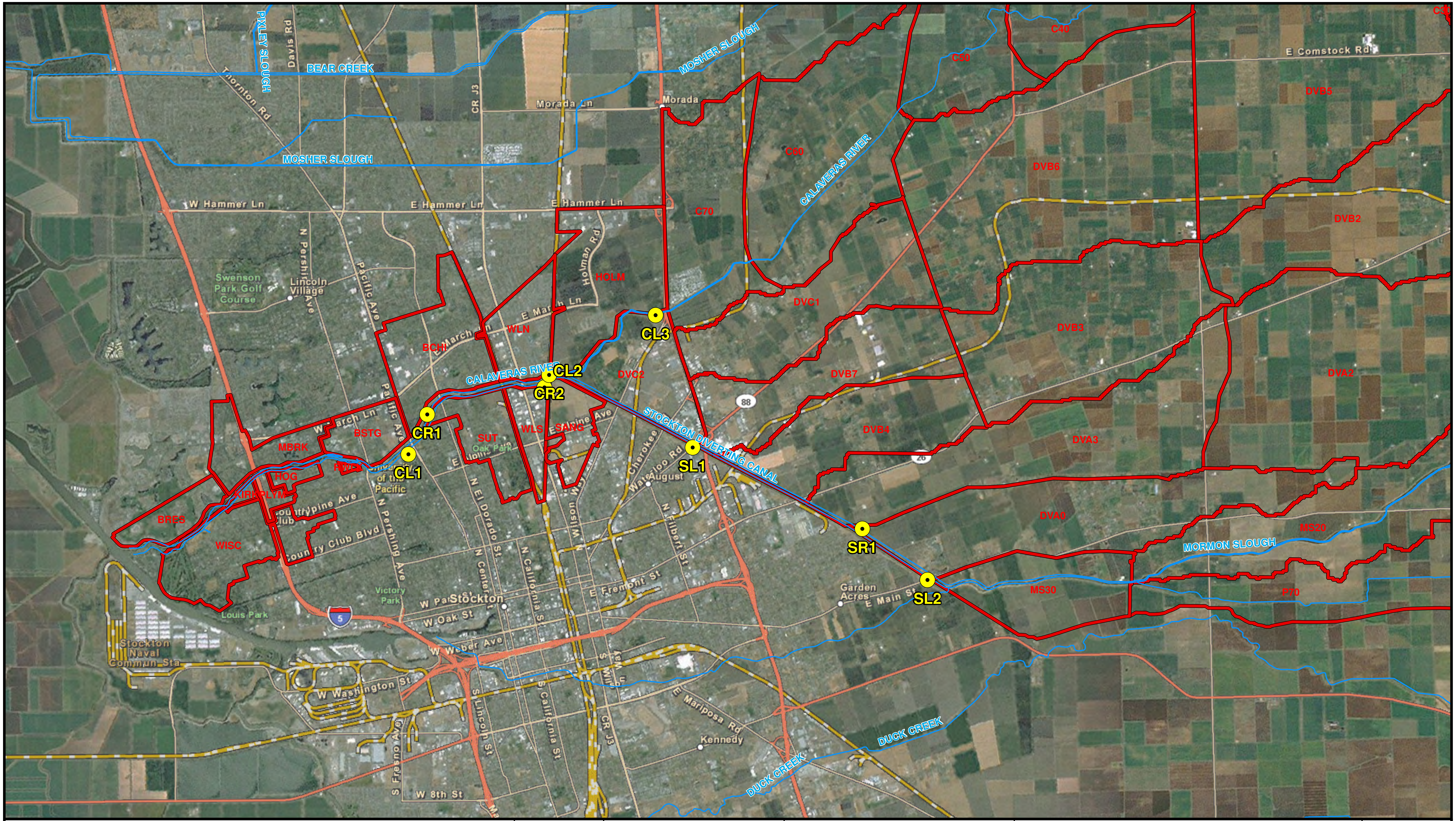
SAN JOAQUIN AREA FLOOD CONTROL AGENCY

CALAVERAS RIVER
HEC-HMS SUBBASINS

FIGURE

5-2

Plate 33. Calaveras River HEC-HMS Subbasins



🟥 Subshed Boundary

● LSJRFS Index Point

N

0 0.25 0.5 1 Miles

1 inch = 1 mile

JUNE 20, 2012

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

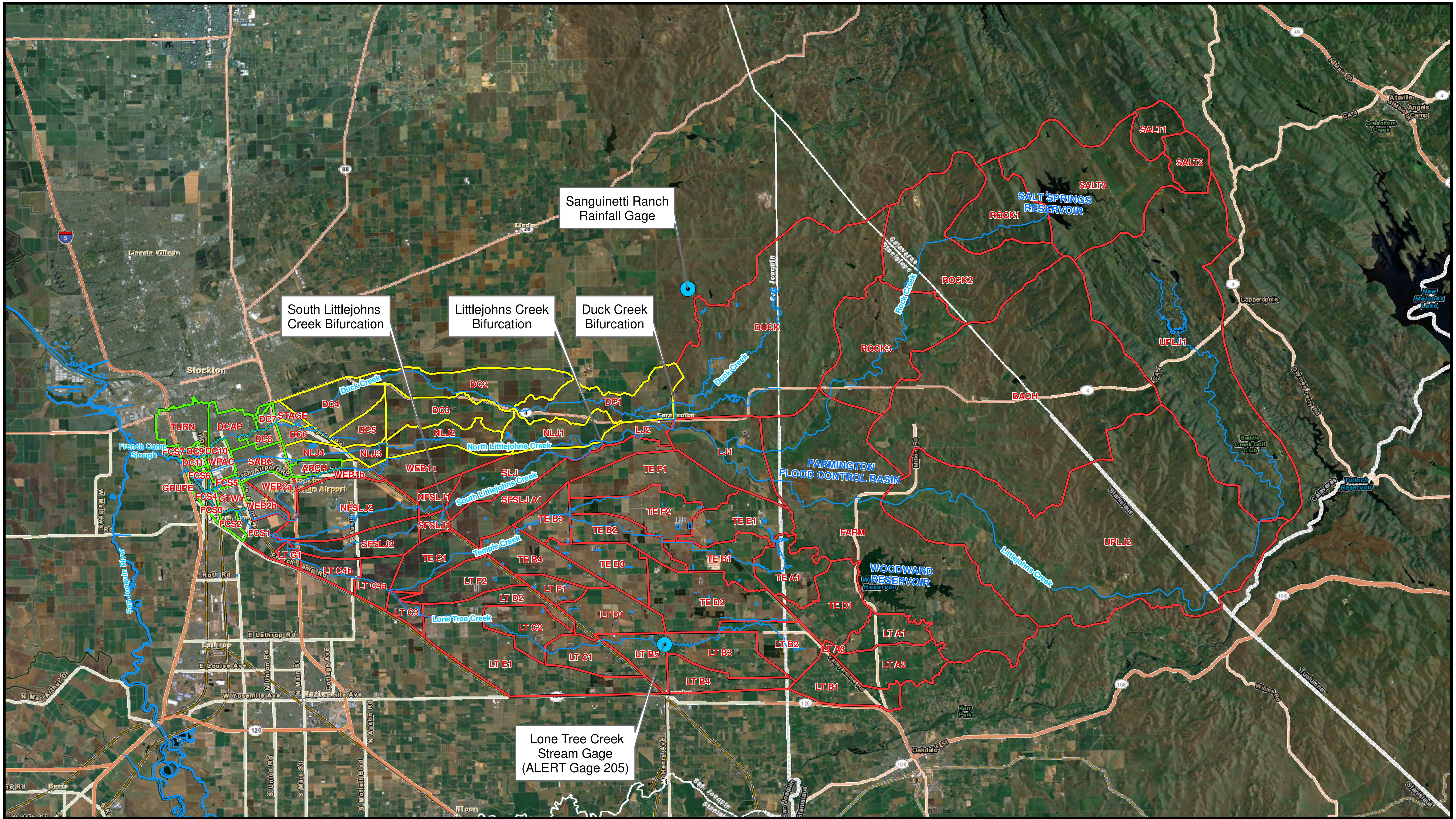
Phone: (916) 608-2212
Fax: (916) 608-2232

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

CALAVERAS RIVER WATERSHED
INDEX POINTS

FIGURE
5-12

Plate 34. Calaveras River Watershed Index Points



Sanguinetti Ranch Rainfall Gage

South Littlejohns Creek Bifurcation

Littlejohns Creek Bifurcation

Duck Creek Bifurcation

Lone Tree Creek Stream Gage (ALERT Gage 205)

🟥 Subbasins from the Tidewater Model

🟨 Subbasins from the Mariposa Lakes Model

🟩 Subbasins Added by PBI

N

0 1.5 3 Miles

1 inch = 3 miles

OCTOBER 19, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

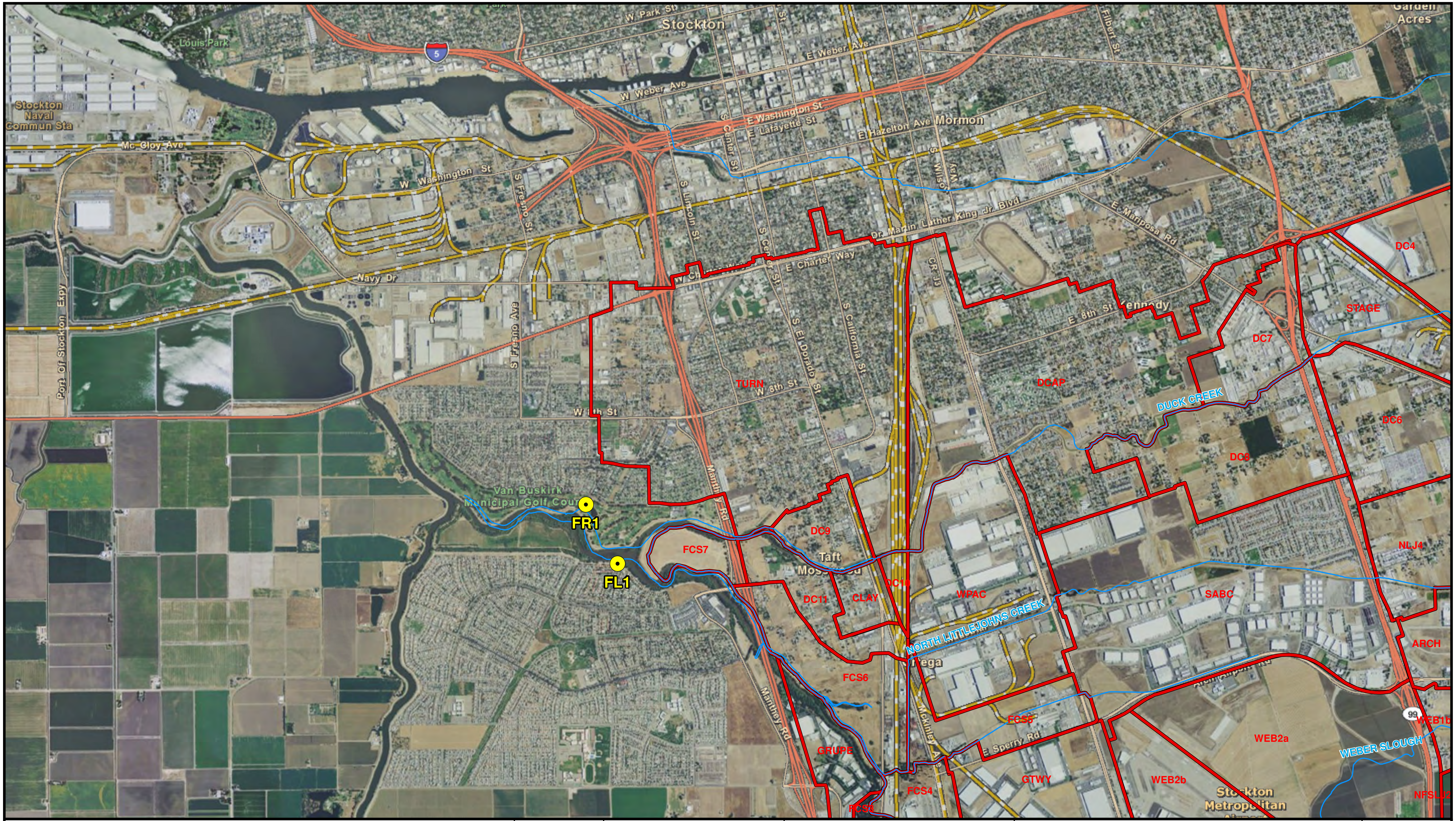
Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

FRENCH CAMP SLOUGH
HEC-HMS SUBBASINS

FIGURE
6-2

Plate 35. French Camp Slough HEC-HMS Subbasins



▭ Subshed Boundary

⊙ LSJRFS Index Point

N

0 0.125 0.25 0.5 Miles

1 inch = 1/2 mile

JUNE 20, 2012

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

FRENCH CAMP SLOUGH WATERSHED INDEX POINTS

FIGURE

6-11

Plate 36. French Camp Slough Watershed Index Points

Appendix 1

Lower San Joaquin River Feasibility Study
Calaveras River above Bellota
Hydrologic Analysis, 20 March 2014

US Army Corps
of Engineers.

Sacramento District

23 June 2014

Table of Contents:

Figures:

Tables:

Appendices:

1.0 Background

The Corps of Engineers, Sacramento District, Hydrology Section (SPK) tasked David Ford Consulting Engineers, Inc (DFC) with the derivation of unregulated and regulated flow-frequency curves at New Hogan Dam and Mormon Slough at Bellota (main control point for New Hogan Dam). Their report is titled: "Lower San Joaquin River feasibility study: Calaveras River frequency analysis and hydrographs" dated June 20, 2011. After DFC performed their analysis, revisions were made by SPK in February of 2012. These include 1) a newer version of HEC-ResSim was utilized for flood routing since the version DFC utilized had difficulty maintaining the objective flow release at Bellota – mainly due to local flow fluctuations 2) SPK reduced to four rather than nineteen the number of pattern floods used for scaling and routing through Res-Sim. As floods equal to or exceeding the 1% ACE event are the primary focus of alternatives in this study, SPK used only patterns that were representative of rare floods. The parts of the DFC analysis that remain valid and are incorporated into SPK's adopted hydrology are 1) unregulated frequency curve analyses including derivation of local flows during historic events 2) analysis of the critical duration and 3) the peak to volume characteristic curves. The parts of the DFC report that are superseded include their adopted unregulated to regulated transform and final regulated frequency curves at each index point. The DFC Report is attached to this Appendix and superseded sections have watermarks labeled "Superseded". The SPK report describes the final adopted hydrology for the feasibility study.

The lower watershed downstream of the Bellota gage was analyzed by Petersen Brustad, Inc (PBI) using a rainfall runoff model. See Chapter titled "Calaveras River Downstream of Bellota" for details on that analysis. The various frequency hydrographs developed at Bellota by SPK (as described in this chapter) became boundary condition input to the HMS model of the Calaveras River produced by PBI. One of the major purposes of the PBI model was to produce concurrent local flow hydrographs for areas downstream of Bellota, during a specific ACE flood event occurring at the Bellota gage. Levees are prevalent on lower Mormon Slough and the lower Calaveras River, which prevents local runoff from getting into the levees except by pumping. As such, a storm centered on the lower watershed will NOT produce the highest runoff within the levee system, needed for alternative analysis. A storm centered somewhere above the Bellota gage is important for modeling the levee system and economic damage areas.

It should be noted that an unregulated flow frequency curve at Bellota was the foundation for derivation of a regulated flow frequency curve at the Bellota gage. As such, the adopted regulated quantile flows are based on many different storm centerings that the gage has encountered during its long period of record.

The study area for the Calaveras River above Bellota is shown in figure 1 below.

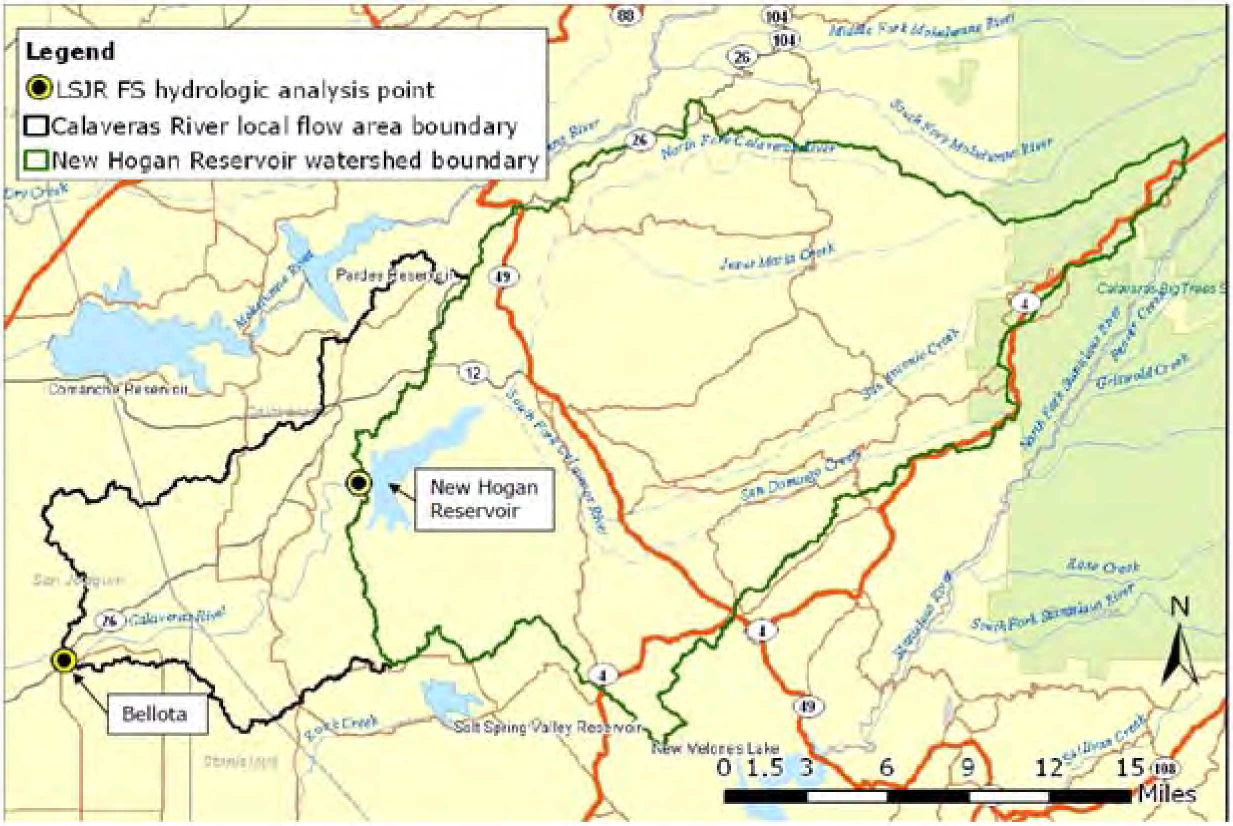


Figure 1. Calaveras River Study area

2.0 Watershed description

The watershed that is the subject of this report—the Calaveras River basin—is part of the lower San Joaquin River basin. It is located in Calaveras, San Joaquin, and Stanislaus counties. Located on Calaveras River approximately 28 miles upstream of Stockton, CA, is New Hogan Reservoir, a multipurpose facility with water supply, recreation, and flood control requirements. The Calaveras River basin encompasses 707 mi2. The north and south forks of the Calaveras River meet just east of New Hogan Reservoir and continue flowing into the reservoir. The basin comprises 3 major areas: The area above New Hogan Reservoir, which includes 363 square miles of relatively low-lying area on the western slopes of the Sierra Nevada. Elevations range from 550 ft at the dam to approximately 6,000 ft at the highest point. The 110 mi^2 area between New Hogan Reservoir and the downstream operation point at Bellota (the bifurcation of the Old Calaveras River and Mormon Slough approximately 18 miles downstream of the reservoir). The elevation at Bellota is approximately 130 feet. The remaining 234 mi^2 area of the Calaveras River and Mormon Slough watershed from Bellota to the confluence with the San Joaquin River. This portion of the watershed is low and flat with little topographic relief. Note: hydrological analysis of this region is completed by Petersen Brustad, Inc and is therefore beyond the scope of the analysis described here. The channel capacity downstream of New Hogan Reservoir is 12,500 cfs and the reservoir operates to limit flow to this value downstream of the dam and at Bellota (USACE 1983). A control structure exists at Bellota to divert the majority of flows into Mormon Slough. Downstream of this structure lies the Old Calaveras River channel, which is overgrown with vegetation. Flow is diverted into the Old Calaveras River when flow at Bellota reaches 13,500 cfs(USACE 1983).

3.0 Procedure for Analysis

The following steps were used to derive hydrographs for Mormon Slough at Bellota.

- Develop unregulated flow time series including New Hogan Dam inflow and local flow (between dam and the Bellota gage). This analysis was performed by DFC
- Develop 1-, 3-, 7-, 15-, and 30-day unregulated volume-frequency curves at New Hogan Reservoir and Mormon Slough at Bellota following the procedures in *Guidelines for determining flood flow frequency, Bulletin 17B* (IACWD 1982), EM 1110-2-1415 (USACE 1993) and using recent USGS regional skew analysis.
- If hourly unregulated flow is not available, convert daily unregulated hydrographs to hourly hydrographs using algorithm which preserves daily volume.
- Input historic and scaled unregulated hourly hydrographs into HEC-ResSim (both reservoir inflow and local flow) to create regulated hourly hydrographs at Bellota.
- Perform critical duration analysis at Bellota to determine volume duration that will be used in unregulated to regulated transform
- Fit at Bellota location, flow transforms to the event maxima datasets identified from the unregulated flow and corresponding simulated regulated time series.
- Developed a regulated flow-frequency curve and associated volumes by applying the flow transforms.
- Developed “expected” outflow hydrographs for Mormon Slough at Bellota for 8 flood frequencies: p=0.5, p=0.2, p=0.10, p=0.05, p=0.02, p=0.01, p=0.005 and p=0.002.   (Here the term expected hydrograph refers to a hydrograph that has a peak corresponding to the regulated flow frequency curve and associated volumes matching those from the family of characteristic curves corresponding to the given regulated peak flow.)

Figure 2 below illustrates the overall process.

The benefit of using multiple pattern floods events is that hydrograph shape, timing of runoff, and storm centering characteristics (spatial distribution of runoff) all result in different peak and volume runoff at index points. Modeling a hypothetical flood event using only one pattern does not account for the true variability of nature. Use of multiple patterns is more in line with USACE risk policies.

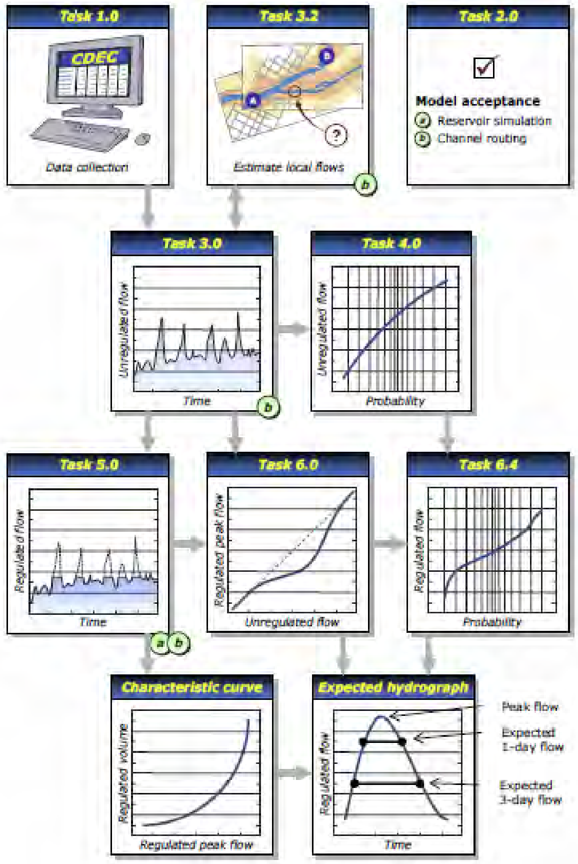


Figure 2: Process Flowchart

4.0 Unregulated flow time series development

SPK's Hydrology Section constructed unregulated flow time series at New Hogan Dam (for the Central Valley Hydrology Study) while DFC produced an unregulated times series at Mormon Slough at Bellota. DFC used the unregulated times series data provided by SPK for New Hogan Dam to construct the Bellota time series. DFC fitted unregulated volume-frequency curves for both of these locations. Thus, for unregulated conditions, the reservoir inflows were needed. For development of the unregulated flow time series downstream of the reservoir, a routing model was required to simulate the translation, attenuation, and combination of the unregulated flow hydrographs through the system. These flow hydrographs included the upstream boundary conditions (derived reservoir inflows) and intermediate area boundary conditions (estimated local flows). The routing yielded unregulated flow time series that served as the basis of: (1) the unregulated frequency analysis and (2) the unregulated-regulated flow transform. For this analysis, we developed an unregulated flow time series on the Calaveras River by: a) calculating daily unregulated reservoir inflow time series b) developing local flow time series for the area between New Hogan Reservoir and the reservoir's control point at Bellota d) completing the unregulated flow time series at the Bellota analysis point.

Obtain daily reservoir inflow. The Corps developed the daily unregulated reservoir inflow time series for New Hogan Reservoir using the continuity equation, in which, for a given time step, the average inflow equals the outflow plus the change in reservoir storage. For the calculation of these inflows, the source of the observed reservoir outflows and observed changes in storage was the Corps's database. By convention in the Central Valley, these calculations were completed on a 1-day time step, thus midnight to midnight values were used. This is consistent with the work completed for the *Sacramento and San Joaquin river basins comprehensive study* (Comp Study) completed in 2002 (USACE 2002).

Estimate local flow For the Calaveras River, local flows needed to be estimated for the area between New Hogan Reservoir and Bellota. Attachment 1 (page 52: Calaveras River local flow development) provides more details on this analysis. The estimation approaches used were:

- Option 1. Direct calculation of local flow using known releases from New Hogan Reservoir and the observed flows at Bellota, routing hourly flows as necessary. In the case of missing streamgage data, local flows values were interpolated as needed.

- Option 2. Estimation of local flows as:

  $QLocal = 3.2(QCosgrove)$

  where *QLocal* is the local flow estimate for a given time, and *QCosgrove* is the observed flow at the Cosgrove Creek near Valley Springs, CA, streamgage. The Corps estimates local flows for the purpose of real-time reservoir operations using this option (John High, personal communication, 11/9/2009).

- Option 3. Estimation local flows as:

  $QLocal = 0.226\ (QNHG)$

  where *QLocal* is the local flow estimate for a given time, and *QNHG* is the unregulated inflow to New Hogan Reservoir. The development of this relationship is show in Attachment 1

In Table 1 we summarize the selected approaches for local flow estimation on the Calaveras River by water year. This flow represents the total local flow contribution at Bellota. Attachment 1 provides details on the development of the local flow times series.

Table 1. Selected local flow estimation approaches for the area on the Calaveras River between New Hogan Reservoir and Bellota

Time period (water year) (1)	**Time step (2)**	**Selected approach[1] (3)**
1907-1929	Daily	Option 3: 0.226 times reservoir inflow.
1930-1969	Daily	Option 2: 3.2 times Cosgrove Creek flow.
1970-1987	Daily	Option 3: 0.226 times reservoir inflow.
1988	Daily	Option 1: directly calculate local flow.
1989	Daily	Option 3: 0.226 times reservoir inflow.
1990-1993	Daily	Option 1: directly calculate local flow.
1994-1995	Daily	Option 3: 0.226 times reservoir inflow.
1996-2009	Hourly	Option 1: directly calculate local flow.
2010	Daily	Option 2: 3.2 times Cosgrove Creek flow.

1. The approach listed is the predominant method for estimating local flows over the time period given. See Attachment 1 for further detail.

Complete unregulated flow time series

For the reservoir's operation point on the Calaveras River at Bellota, DFC combined the daily unregulated inflow time series with the estimated local flows by adding the 2 time series together. DFC did not route the unregulated reservoir inflows because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the reservoir and the operation point is approximately 7 hours, which is less than the 1-day time step of the inflows. In addition, there is little attenuation of flood peaks in this reach because of its length and channel geometry. DFC confirmed this by comparing observed releases from New Hogan Reservoir, observed flows on Cosgrove Creek, and observed flows on the Calaveras River at Bellota. Figure 3 displays the local flow area downstream of New Hogan Dam.

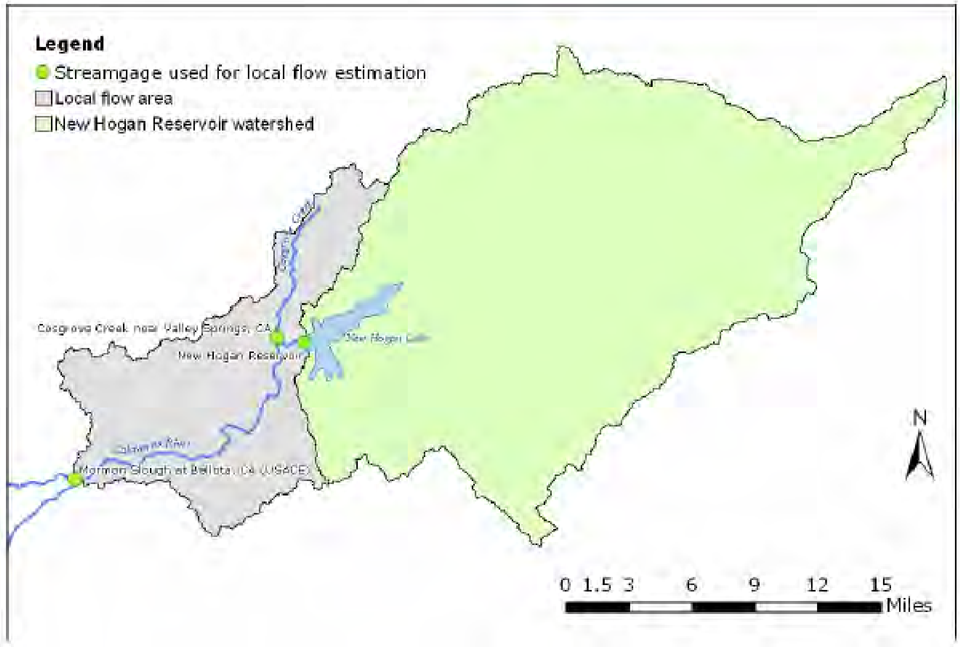


Figure 3: Local Flow Area Below New Hogan Dam

5.0 Unregulated frequency analysis

Accepted procedures to develop unregulated flow-frequency curves are specified in *Bulletin 17B* (IACWD 1982). The current standard-of-practice is to fit a Pearson III (LPIII) distribution to the logarithmic transforms of annual maximum series identified from streamgage data. Additional guidance for fitting frequency curves to volumes for a given duration is provided by EM 1110-2-1415 (USACE 1993). For this analysis, DFC used the unregulated inflows to New Hogan Reservoir to develop such an annual maximum series. However, because DFC only had records of regulated flows on the Calaveras River at Bellota, DFC could not fit a frequency curve directly using this method. Thus, DFC used the synthesized unregulated flow time series at this location and fitted a volume-frequency curve to that series. For this analysis DFC developed unregulated frequency curves that generally follow procedures specified in *Bulletin 17B* (IACWD 1982) with modification from the EMA procedure. This new procedure is being evaluated by the Bulletin 17C Committee for possible adoption for new federal guidelines for flow frequency. HQ USACE has given districts permission to use EMA. The EMA procedure includes different procedures for handling historic floods and a new outlier detection test called Multiple Grubbs-Beck. In some cases, the Multiple Grubbs-Beck test can result in a larger number of low outliers being censored than the Grubbs-Beck test used in Bulletin 17B.

For each analysis location, DFC:

- Identified the annual maximum series.
- (Task 4.1) Calculated regional skew values for each duration of interest using relationships developed by the USGS.
- (Task 4.2) Fitted LPIII distributions to the annual maximum series using the expected moment algorithm (EMA) enabled flow-frequency software PeakfqSA, version 0.937. This was

developed by Tim Cohn of the USGS and is based on the USGS's flow-frequency software PeakFQ (Cohn 2007).

- Reviewed and adopted the curves, checking them for consistency and comparing them to previously accepted values.

Identify annual maximum series

DFC identified the annual maximum series by extracting, from the unregulated flow time series, the volumes associated with the 1-, 3-, 7-, 15-, and 30-day durations. This information is detailed in attachment 1 (see pages 21 and 61). Note DFC developed a peak unregulated flow-frequency curve for New Hogan Reservoir for completeness; however this is not required for this analysis. In addition, DFC did not develop a peak flow-frequency curve for the Calaveras River at Bellota because the temporal resolution of the unregulated flow time series, 1 hour to as long as 1 day, is not an appropriate representation of instantaneous unregulated peak flow values.

Calculate regional skew values

For this analysis, DFC calculated regional skew values for the peak flows and 1-, 3-, 7-, 15-, and 30-day volumes using the relationships developed by the USGS (USGS 2010). In these relationships, the regional skew value is a function of the average basin elevation. The values calculated for each analysis location and duration of interest are shown in attachment 1 (see page 76).

Fit frequency curves

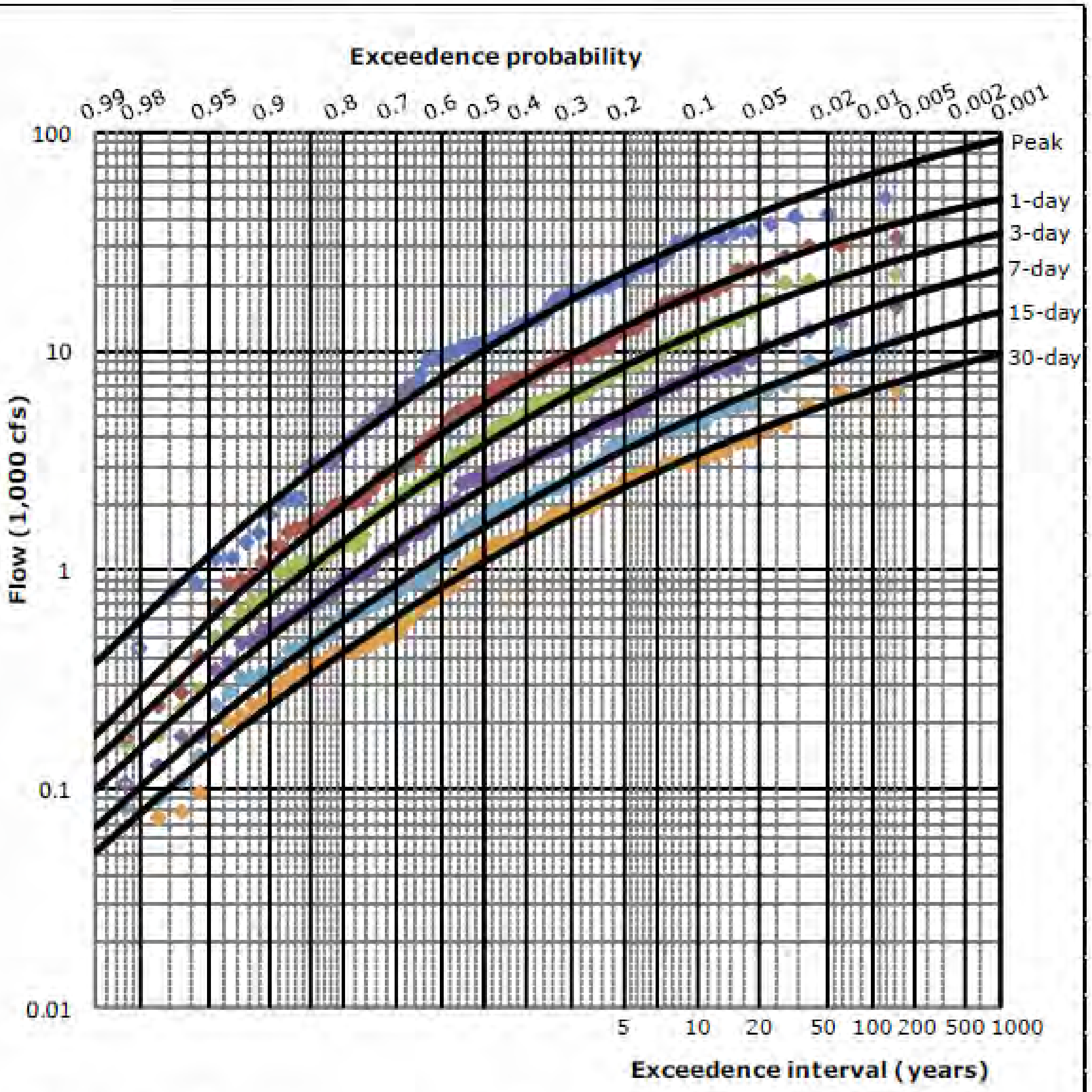
To fit frequency curves to the annual maximum series DFC used: (1) the statistics of the logarithmic transforms of unregulated flow time series (mean, standard deviation, and skew), and (2) the regional skew values for the peak flow, and 1-, 3-, 7-, 15-, and 30-day calculated using relationships developed by the USGS (2010). The "at station" statistics were calculated using the EMA option in PeakfqSA. The weighted skew is automatically calculated by the PeakfqSA software used here.

Review and adopt curves

After fitting, DFC reviewed the frequency curves for consistency and appropriateness. Specifically, DFC:

- Compared the curve of a given duration to the curves associated with the other durations at the same analysis location.
- Compared the curves at a given location to the curves at the other analysis location to check for consistency.
- Compared the curves to those published in the Comp Study. DFC found the frequency curves on the Calaveras River were consistent between durations at each location. The curves do not "cross," and flow quantiles for a given duration at the downstream location are greater than those of the upstream location, as would be expected. As a comparison, DFC considered the volume-frequency curves developed for New Hogan Reservoir in the Comp Study (USACE 2002). The annual maximum series in the Comp Study ended in 1997. DFC also found that the flow quantiles of the curves fitted here and those of the Comp Study differ between the 2 sets of volume-duration curves by only 1% - 13%. The greatest differences (of 8%-13%) are in the 1-day volume quantiles. The 3-day and 7-day volume quantiles differ by only 1% to 5%. Peak

flow-frequency curves varied by as much as 9% because of the increased number of large events included in this analysis as compared to the Comp Study. DFC adopted the unregulated frequency curves for the two analysis locations, New Hogan Reservoir and Bellota, shown in Figure 4 and Figure 5. The detailed parameters used to fit these curves are included in Attachment 1 (see page 76).



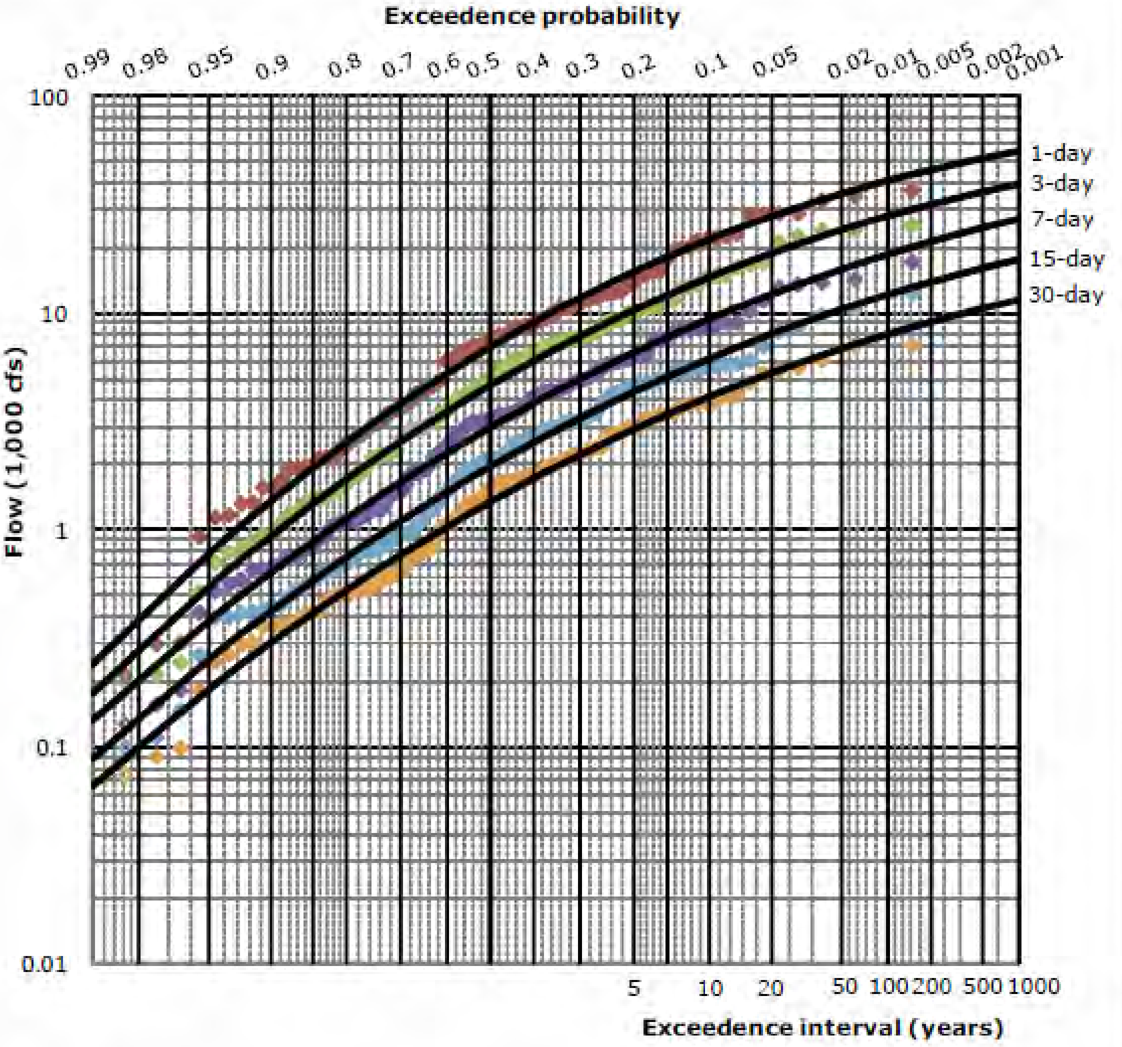
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.946	0.482	-0.727
1-day	3.685	0.501	-0.794
3-day	3.518	0.487	-0.732
7-day	3.324	0.477	-0.651
15-day	3.146	0.473	-0.656
30-day	2.988	0.457	-0.659

Notes:

- Median plotting positions.
- Drainage area: 363 sq. miles.
- Period of systematic record: 1907-2010.
  (Peak flow data intermittent 1930-2010).
- Record lengths
  Peak flows: 86 years.
  Volumes: 104 years.
- Regional skew values developed by USGS.
- Hollow points are censored events.

Figure 4: New Hogan Dam Unregulated Flow Frequency Curves
Note: Multiple Grubbs Beck test censored values shown as hollow points



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.775	0.482	-0.810
3-day	3.608	0.475	-0.753
7-day	3.417	0.464	-0.666
15-day	3.240	0.461	-0.671
30-day	3.079	0.448	-0.668

Notes:

- Median plotting positions.
- Drainage area: 473 sq. miles.
- Period of systematic record: 1907-2010.
- Record length: 104 years.
- Regional skew values developed by USGS.
- Hollow points are censored events.

Figure 5: Mormon Slough at Bellota Unregulated Flow Frequency Curves
Note: Multiple Grubbs Beck test censored values shown as hollow points

Smooth unregulated flow time series The daily unregulated flow time series are appropriate for frequency analysis. However daily upstream and intermediate boundary conditions do not have the temporal resolution required by the CVHS procedures for assessing the effects of regulation, particularly releases as indicated on the emergency spillway release diagram (ESRD). Therefore, the daily reservoir inflows and daily estimated local flows were "smoothed" to hourly time series for input into HEC-ResSim by SPK staff. This smoothing was completed using a mass balance algorithm that interpolates the shape of the hydrograph and estimates peak hourly flows while maintaining daily volumes consistent with the original time series.

6.0 Regulated flow time series development

As mentioned before, SPK developed the adopted regulated times series for this study. To develop regulated flow-frequency curves, the unregulated volume duration- frequency curves are transformed through the unregulated- regulated flow transform. The unregulated-regulated flow transform captures the system's response to large, varied events, and is created using the unregulated and regulated flow time series data. To develop the regulated flow time series, SPK took four selected historical events (1956, 1936, 1938, and 1958) from the unregulated flow time series and simulated those in the regulated system using HEC-ResSim. In addition, SPK downscaled and upscaled the unregulated hourly pattern hydrographs and ran them through HEC-ResSim to represent a full range of different sized events. SPK then compiled the maximum unregulated and regulated flow data pairs for various durations to develop the event maxima datasets. These datasets became the basis for the unregulated to regulated transform development. To create transforms, one must first perform a critical duration analysis at each analysis point for the study.

Determine critical duration

DFC performed a critical duration analysis at two locations. Details on this analysis can be viewed in Attachment 1 (see page 81). In their analysis DFC identified the duration of the unregulated annual maximum series that consistently estimates the largest flow for each probability. In selecting the critical duration, they considered both the "goodness of fit" of each transform and which duration estimates the greater peak regulated flows. From their analysis, they determined that the critical duration at New Hogan Reservoir is 3.5 days, while at Bellota it is 1 day. Thus, the appropriate unregulated-regulated flow transforms used in this analysis were associated with these durations. The critical duration associated with the downstream operation point is shorter than that of the reservoir because of the effects of uncontrolled local flow. Local flow is not insignificant. A PBI rainfall runoff analysis with a calibrated model indicates that a 0.005 ACE storm centered between New Hogan Dam and the Bellota gage is capable of producing a peak flow of 12,500 cfs entirely from the local flow area (drainage area is approx. 100 square miles). 12,500 cfs is the objective flow at the Bellota control point in this watershed.

Table 2. Calaveras River floods-of-record at New Hogan Dam

Water year[1] (1)	**Start date (2)**	**End date (3)**	**1-day max volume (cfs) (4)**	**Selection basis (5)**
1958	3/10/1958	4/30/1958	32,920	*Large inflow event*
1938	1/25/1938	2/28/1938	30,450	*Large inflow event*
1911	1/10/1911	2/28/1911	30,175	Unreliable Local Flow
1936	2/10/1936	3/24/1936	26,987	*Large inflow event*
1907	3/1/1907	4/14/1907	23,641	Unreliable Local Flow
1986	2/10/1986	3/6/1986	23,494	Comp Study storm matrix event
1956	12/15/1955	2/15/1956	20,156	*Reasonable Local Flow Character*
1998	1/1/1998	3/15/1998	16,919	Comp Study storm matrix event
1997	12/1/1996	2/15/1997	16,801	Comp Study storm matrix event
1969[2]	1/5/1969	3/20/1969	14,674	Comp Study storm matrix event
1940	2/11/1940	3/16/1940	13,610	Comp Study storm matrix event
1965	12/18/1964	1/18/1965	12,789	Comp Study storm matrix event
1982	12/28/1981	1/31/1982	12,321	Comp Study storm matrix event
1983	2/25/1983	4/10/1983	10,433	Comp Study storm matrix event
1995	3/1/1995	4/6/1995	10,146	Comp Study storm matrix event
1951	11/12/1950	11/31/1950	9,390	Comp Study storm matrix event
1980	1/1/1980	1/31/1980	8,648	Comp Study storm matrix event
1967	1/20/1967	2/10/1967	6,738	Comp Study storm matrix event
1978	3/1/1978	3/19/1978	5,770	Comp Study storm matrix event

1. Events are in order of increasing 1-day flow volume
2. For the purposes of this analysis, treat the 1969 flood as 1 single event.
3. Pattern flood used for reservoir routing shown in italics font

---

Reservoir Regulation Simulation Criteria

SPK's Hydrology Section performed the final reservoir simulation in HEC-ResSim (version 3.1.8 RC4). This version corrected problems that DFC encountered when running an earlier version that was unable to keep the flow at Bellota to the objective channel flow of 12,500 cfs. At times, the older version of the model produced flows up to 14,000 cfs even though plenty of flood space remained behind the dam.

The HEC-ResSim model was developed as part of the Central Valley Hydrology Study. An Agency Technical Review (ATR) was performed by a retired annuitant working at HEC (Dan Barcellos). The model was setup to follow the rules in the latest approved Water Control Diagram.

Starting storage assumption: Starting storage is assumed to be bottom of flood control as defined in the Water Control Diagram. For each event modeled, 45 days of scaled historic inflow (including pre- and post-waves around the main flood wave) were ran for each simulation. One consistent ratio was applied to all ordinates of the historically based 45 day inflow hydrograph pattern. The purpose of the longer simulation was to partially compensate for the starting storage assumption, i.e. measure the impact of multiple waves of inflow to the dam over time upon its operation. Review of historic floods at New Hogan Dam indicate that starting at bottom of flood control is a reasonable assumption. Figure 6 shows the New Hogan Dam storage at the beginning of the 1997 flood event.

Adjustments for common floods: For the more common events, the antecedent storage condition might have the reservoir below bottom of flood control. In other words, there is water supply space available to absorb the inflow volume during an event. Another factor is that reservoir managers have a history of making releases at less than objective flow rates if forecasts indicate the event will be small. To compensate for these realities, SPK's Hydrology Section produced a graphical peak flow frequency curve at Bellota for the period after the dam was built. The gage record for this period includes both reservoir outflow and local flow. For probabilities of 0.5 to 0.04 ACE, the adopted regulated n-year hydrographs were adjusted to match the graphical peak curve based on historic data. Adjusting the hydrograph to match historic data for common events compensates for our starting storage assumptions, and for the decisions water managers make during these types of events.

Seasonal floods: The scaled events keep their historic time stamp in the dssfile when input into HEC-ResSim. The 1958 flood occurred in early April (maximum 1-day flow occurred April 3rd). The ResSim model has a smaller amount of flood space at this time of year due to the seasonality of the rule curve in the Water Control Diagram. As such, it turned out the 1958 flood pattern was the most difficult for the ResSim model to control. The probability assigned to the scaled 1958 floods came from the 1-day rainflood frequency curve which includes December through March flood events. This is a conservative way of estimating the probability of a specific flood occurring in spring. The true probability of such a flood occurring in April is best evaluated by performing a seasonal flow frequency analysis, which undoubtedly would assign it a more rare frequency than our current method. In hindsight, if SPK conducted this study a second time, it should take this into consideration. Since the median transform was used to define the adopted regulated frequency curve, the current use of the 1958 flood pattern did not adversely impact the outcome of the analysis since the 1958 transform fell on the high side of the four transforms produced.

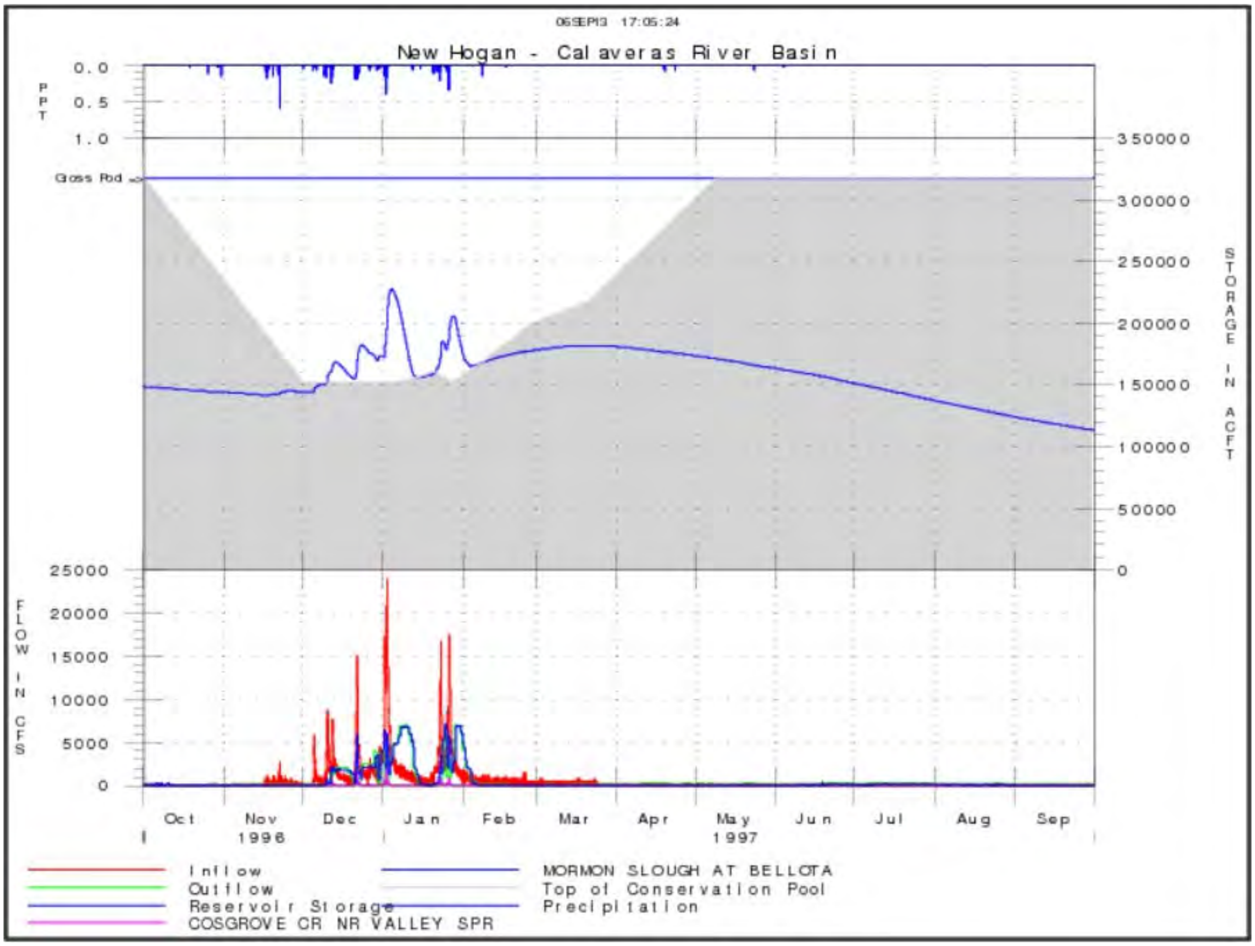


Figure 6: Storage at New Hogan Dam at start of 1997 flood event

Selection of Pattern Floods Used in ResSim Routings. The main focus of this feasibility study is to provide urban areas like Stockton flood protection from rare floods. Many tributaries studied in this feasibility study (such as Calaveras/Mormon Slough) currently have levees that were originally designed to provide protection from the 0.01 ACE event. The sponsors have a keen interest to achieve protection from the 0.005 ACE event. As such, SPK chose to pick some of the rarest historic events as a template for modeling alternatives in this watershed. The rarer flood patterns should also provide a better estimate of the local flow runoff that the reservoir will have to deal with when a really rare events occurs. Within the 104 years of recorded flow, the highest four ranking floods (ranked largest to smallest using the 1-day unregulated volume) are 1911, 1958, 1938, and 1936. 1911 was thrown out; however, because neither the Cosgrove Creek gage nor the Bellota gage were available to estimate local flow and therefore local flow had to be computed as a ratio of reservoir inflow (this method is considered the least accurate method of local flow estimation). The 1911 flood was replaced with the Dec 1955 flood because a) it was one of the most closely monitored/documented floods in the Central Valley and b) its local flow was within the range of variability of the other three large events used in this analysis (1958, 1938, and 1936). Table 3 below shows information about the selected patterns including local flow characteristics.

Event	Ranking by total 1-day unregulated volume	Hourly peak of total flow unregulated hydrograph (cfs)	Hourly peak of local flow hydrograph (cfs)	Percent local flow	Date of 1-Day maximum unregulated flow	Date of 1-day maximum local flow
1958	1	50,300	2190	4%	03 April	01 April
1938	2	46,400	3200	7%	11 Feb	11 Feb
1936	4	41,000	3800	9%	23 Feb	22 Feb
1956	7	30,300	2800	9%	23 Dec	23 Dec

Table 3: Selected Patterns for Res-Sim Routings

The choice of events was guided in part by the confidence in the local flow computations. The method of local flow computation by direct calculation of the difference between the historically observed hourly releases at New Hogan tailwater and the observed flow at Bellota is acceptable. Also acceptable is the method of local flow calculation by ratio of historically observed hourly flow at Bellota and at Cosgrove Creek at Valley Springs. The ratio of local flow at Bellota to the flow at Cosgrove Creek was found to be 3.2 by analysis of historic floods and is used for real-time water control decisions. The analysis was conducted by the District Hydrologist (Robert Collins) some years ago, although the details of the analysis are not currently available. The 1997 flood closely followed this rule as shown in Table 4. The computation of local flow by ratio with reservoir inflow is judged to be the least accurate as this relationship was found to be highly variable. Therefore, events where local flow was computed as a ratio of reservoir inflow were discarded for use in the regulated analyses. A comparison of the ratios of Bellota local flow to reservoir inflow and Cosgrove Creek flow for six historical events are shown in table 4 below.

Ratios of Bellota Local to New Hogan Inflow and Cosgrove Creek to Bellota Local
for six flood events: 1965-1967-1986-1995-1997-1998.
Copied from PORx1.0 simulation.dss by Ford.

Year of Event	Bellota Local	Bellota Frequency	New Hogan Reservoir Inflow	NewHogan Inflow Frequency	Cosgrove Creek	Cosgrove Frequency	Bellota Local / Res Inflow	Bellota Local / Cosgrove Creek
1965	2303.3	0.68	19000.0	0.25	N/A	N/A	12.1%	N/A
1969	1592.4	0.16	21900.0	0.15	N/A	N/A	7.3%	N/A
1986	5849.5	0.11	35500.0	0.04	N/A	N/A	16.5%	N/A
1995	2720.8	0.65	14900.0	0.39	N/A	N/A	18.3%	N/A
1997	6688.3	0.12	25100.0	0.17	2048.0	0.60	26.6%	326.6%
1998	9436.0	0.04	25300.0	0.20	2396.0	0.18	37.3%	393.8%

Average ratio from report; Value * ratio = Bellota Local => 22.6% 320.0%

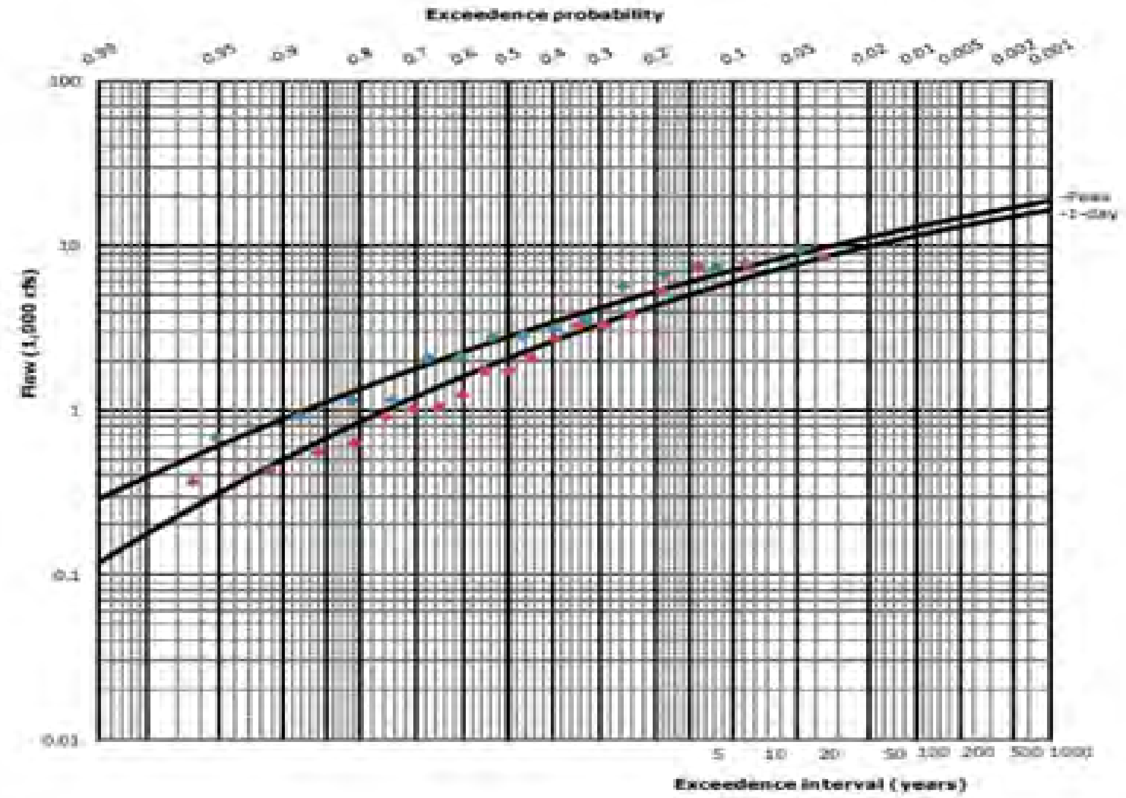
Table 4: Ratios of Bellota Local Flow to New Hogan Dam Inflow or Cosgrove Creek

In summary, since rare floods like the 0.005 ACE event is important for the evaluation of alternatives in this feasibility study, the rarest events were selected as pattern floods to scale and route through HEC-ResSim. The local flow that occurred during these large events is considered the best representation of what might happen in a flood of this magnitude. The 1911 event was thrown out because there is not confidence in the method needed to estimate local flow for this event (Option 3 – ratio of reservoir inflow).

Validating the Transform: USACE guidance indicates that a local flow frequency curve should be developed to determine the lower boundary of a regulated frequency curve developed from an unregulated to regulated transform based on reservoir routings. Theoretically, the transform can exceed the local flow frequency curve but should not fall below it. This is due to the fact that the local flow cannot be controlled and therefore will always impact an analysis point. Local flow does not include reservoir releases. Two estimates of local flow runoff were attempted.

First attempt: DFC derived a "Limited Use Frequency Curve" for peak and 1-day durations using 14 and 19 years of record, respectively. This was the number of water years in which the Option 1 method of local flow calculation was available. Figure 7 below displays the curves. DFC termed it as "Limited Use" because a) it does not include reservoir releases and 2) it was based on a limited number of years of data. The DFC "Limited Use Curve" is provided in this report for interest only and was not utilized in this study, other than to help verify the transform at Bellota was reasonable. The maximums derived for these two curves do not necessarily represent annual maximums, although typically maximum local flow does occur approximately the same time (within a few days) whenever New Hogan Dam has the largest inflow of the water year. Instead, the data used represents the peak local flow runoff that occurred within the 45 day window of the selected flood event that DFC analyzed for each water year where local flow could be calculated using Method 1. Table 5 displays the various quantiles computed from this curve. The adopted transform at Bellota *does not* fall below the Limited Use Frequency Curve for all frequencies (except the 0.005 ACE event). Since a flow frequency curve based on 14 years of data is highly suspect at the upper end due to the small sample size, the curve was not really used for the study. As mentioned later in this report, the 0.50 to 0.04 ACE event hydrographs were modified to match a family of graphical flow frequency curve at Bellota (these curves include both local flow and reservoir releases). For rarer floods, SPK decided to use the PBI calibrated rainfall runoff model with a storm centering above Bellota to estimate local runoff potential for floods equal to or rarer than the 0.02 ACE event. Again, DFC's Limited Use Frequency Curve is presented here for interest only but the study results did not depend on it.

DFC performed a coincidence analysis to determine the relationship of New Hogan Dam inflow and local flow at Bellota (page 25 of attached DFC Report). This was done out of concern that scaling dam inflow and local flow by the same factors may result in local flow that becomes too rare. Figure 19 of DFC Report shows the probability of local versus New Hogan inflow for selected flood patterns and scalings. Ufortunately, the frequency of local flow is appraised with DFC's "Limited Use" flow frequency (14 years of data) which is not very trustworthy. As such, the results are ***inconclusive.***The plot appears to show that 1) local flow is highly variable depending upon the flood event and 2) scaling local flow hydrographs (see values for same color pattern) might not significantly change relationship between reservoir inflow and local flow.



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.421	0.355	-0.492
1-day	3.270	0.427	-0.644

Notes:

- Median plotting positions.
- Drainage area: 110 sq. miles.
- Record lengths:
  Peak flows: 14 years.
  1-day volumes: 19 years.
- Regional skew values developed by USGS.

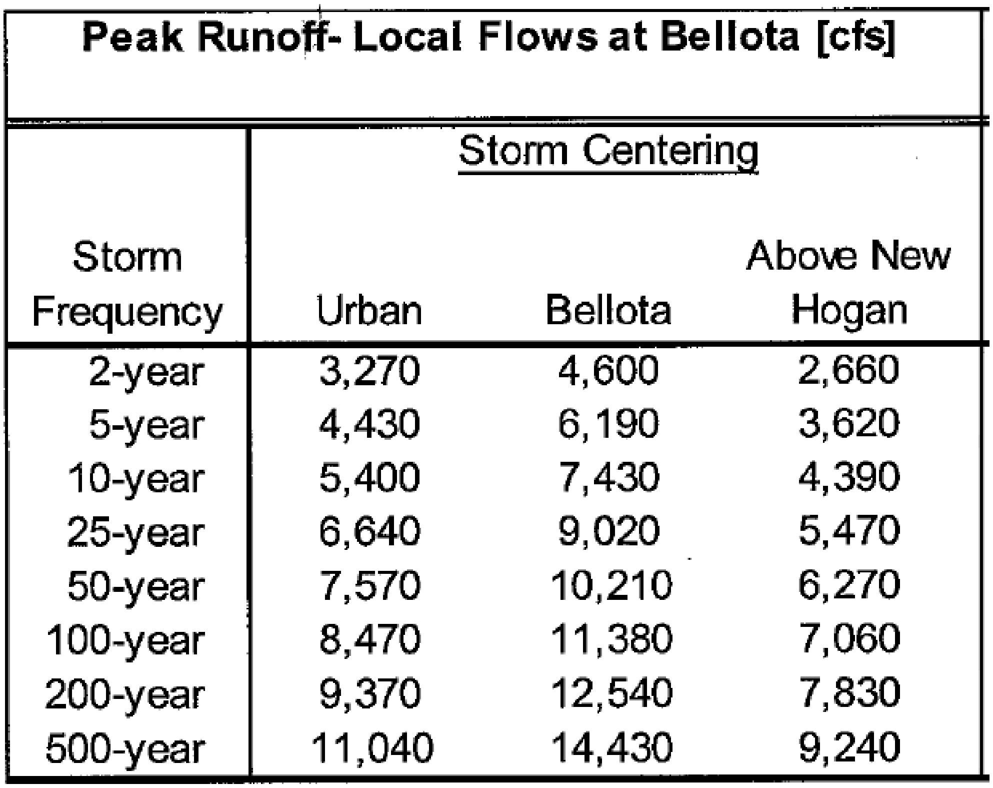
Figure 7: Limited Use “Local Flow” frequency curve (not used in study).

Annual exceedence probability (1)	1/annual exceedence probability (2)	Peak flow (cfs) (3)	1-day volume (cfs) (4)
0.500	2	2,817	2,067
0.200	5	5,310	4,324
0.100	10	7,134	6,015
0.050	20	8,942	7,688
0.020	50	11,318	9,855
0.010	100	13,103	11,449
0.005	200	14,874	12,995
0.002	500	17,188	14,957

Table 5: *Limited Use “Local Flow” Frequency Curve for Mormon Slough at Bellota

*Note: This curve was not used in this study. Presented for interest only. Does not include New Hogan Dam releases. Based on 14 and 19 yrs of data for the peak and 1-day durations.

2nd Attempt: For the overall study, PBI developed a calibrated rainfall runoff model for the lower watershed below New Hogan Dam. The study results of their analysis are discussed in Chapter D (Calaveras River Downstream of Bellota). The model was calibrated to the Bellota gage for a historic storm. After building a calibrated model, an attempt was made to estimate the local flow runoff potential including for the 0.005% ACE event. PBI input two different 0.005 ACE design storms into their calibrated model that were centered between the dam and the Bellota gage. One design storm was the hypothetical, pyramid shaped, storm within HMS that was fully balanced to multiple-duration depths found in NOAA14 and using TP40 areal reduction factors (these factors are built into HMS). The other storm used a 72-hour, 1997 hyetograph pattern that was balanced to only the 72-hour, 0.005 ACE NOAA14 depth and using the HMR 59 areal reduction factor for this duration. In both cases, the resulting peak flow at Bellota in their model was 12,500 cfs. PBI also input various frequency storms centered between Bellota and the dam to get a handle on local flow frequency. The results of those runs is shown in Table 6 below. Except for the 0.50 (2-year event), the transform at Bellota (transform based on the reservoir modeling of both reservoir outflow and local flow combined) did not fall below the local flow runoff peak predicted by PBI's model. Since peak flow frequency at this location was adopted from the graphical regulated frequency curve at Bellota based on 23 years of data, the transform was not used for any events more common than the 0.02 ACE. The PBI analysis results helped validate SPK's transform was reasonable for events more rare than the 0.04 ACE event. This is further explained below.



Peak Runoff- Local Flows at Bellota [cfs]

Storm Frequency	Storm Centering Urban	Bellota	Above New Hogan
2-year	3,270	4,600	2,660
5-year	4,430	6,190	3,620
10-year	5,400	7,430	4,390
25-year	6,640	9,020	5,470
50-year	7,570	10,210	6,270
100-year	8,470	11,380	7,060
200-year	9,370	12,540	7,830
500-year	11,040	14,430	9,240

Table 6: Bellota local flow peaks for storm centerings by PBI.

Note: The storm centered between New Hogan Dam and Bellota (3rd column labeled "Bellota") produced the highest local flow runoff.

0.005 ACE Event: The results of the ResSim modeling (specifically the adopted regulated flow frequency curve) indicate the 0.005 ACE runoff for the Mormon Slough at Bellota analysis point is 12,500 cfs. This may seem to contradict the fact that the local flow runoff is also estimated to be 12,500 cfs for the same frequency event based on rainfall runoff modeling. The discrepancy can be explained as follows:

> a) As Table 4 above indicates, the relationship between New Hogan Dam inflow and local flow runoff is highly variable and not well correlated. The possibility of a 0.005 ACE release from New Hogan Dam and a 0.005 ACE local flow runoff during the same flood event is considered highly unlikely based on Table 4. In fact, for the three largest floods in which local flow can be reasonably calculated (1958, 1936, and 1938), the local flow peak never exceeded 4,000 cfs. 4,000 cfs is approximately a 0.20 ACE (5-year return period) flood based on the DFC Limited Use frequency curve, which implies that the two watershed areas (above and below the dam) are not highly correlated during extreme storms. Another factor is that the maximum local flow runoff sometimes occurs earlier than the peak of the reservoir inflow hydrograph. See the last column of Table 3.
>
> b) The New Hogan Dam Water Control Manual specifically requires the dam to keep releases to no more than 12,500 cfs at Bellota. The rules force the dam to cut back on releases if local flow is high. A separate analysis by DFC at New Hogan Dam indicated the reservoir could keep its releases to about 12,500 cfs (just downstream of the dam) during a 0.5% ACE inflow event if the dam does not have to adjust for downstream local flow. See Attachment 2. Historically, the local flow runoff tends to peak about the same time or earlier than the peak of the reservoir inflow hydrograph. Since the reservoir can delay its maximum releases beyond the time of its maximum inflow, the local flow has a chance to pass downstream before large releases from the dam are necessary (in other words timing comes into play). The above stated facts help explain why the flow at Bellota can be maintained at 12,500 cfs during this size event for some patterns in SPK's ResSim model.

8.0 Create Mormon Slough at Bellota Hydrographs for Specific Frequencies

The following steps were performed to extract an outflow hydrograph for each “n-year” event corresponding to the regulated flow-frequency curve for Mormon Slough at Bellota.

1. Simulate the 1936, 1938, 1956, and 1958 events with HEC-ResSim version 3.1.8 RC4. This version corrects defects in the downstream rule logic. These simulations correspond to the development of regulated flow time series in the DFC report. These simulations develop regulated flow time series for scale factors from 1.0 to 3.0 of reservoir inflow and local flow, which are input to the simulation model. The four events were chosen out of a list of the highest floods of record.

2. Extract the 1-day unregulated flow volume and regulated peak flow at Bellota from the DSS files output from simulations in step 1. The 1-day unregulated flow volume was identified as the “critical duration” by DFC in Attachment 1 (see page 81) for the .02 to 0.005 ACE events. So, the independent variable (x-axis) of the flow-flow transform is the 1-day unregulated flow, with the peak regulated flow being the dependent (y-axis) value. Then use a spreadsheet to input the 1-day unregulated flow and peak regulated flow data pairs to compute the transform for each pattern. SPK’s Hydrology Section decided to adopt the median transform to develop a regulated peak flow frequency curve. To compute the median curve, an average regulated peak flow value (y-axis) is computed for each x value from the two innermost transforms (note: we developed four transforms). Figure 8 displays the four individual event based transforms plus the average and median transforms for the Bellota gage location. Table 7 displays individual values from the average and median transforms. The median transform was adopted for the study.

3. The regulated hydrographs for the 0.5 to 0.04 ACE flows at Mormon Slough at Bellota were *revised to fit observed conditions at the Bellota gage* via a family of graphical curves using 23 years of historic data (water years 1988 to 2010). It is noted that using this approach may limit the ability of the District to evaluate alternatives involving reservoir reoperation or reconfiguration. This is because it is not possible to generate equivalent graphical frequency curves for with-project conditions. Currently, reservoir reoperation is not one of the alternatives being moved forward in the analysis. The methodology described above uses the HEC-ResSim program, with unimpaired inflow data input to the reservoir and local flow areas, with operational rules documented in the Water Control Manuals. This provides a consistent reservoir operation that follows the Congressionally authorized plan of operation. In actual operation as shown by the historically observed flows, the reservoir was operated differently. That is, for smaller, frequent events, the reservoir was not drawn down as quickly as the water control plan suggests, but holds runoff in storage longer while making smaller, lower, releases. For example, during the 1997 flood event, the peak of the simulated release from the dam using HEC-ResSim was 12,500 cfs while the historic release was only 7,500 cfs. Figure 9 shows the actual operation for the January 1997 flood, while Figure 10 shows the hypothetical operations (note: the inflow hydrograph for the hypothetical simulation is derived from daily inflow values smoothed into hourly values using an algorithm which preserves the historic daily volume). Besides modifying the peak of the hydrograph for these frequency events, the volume was also modified to match a frequency analysis of historically observed flows. The runoff volume was found by computing the 1, 3, 7, and 15-day flow

volumes from historic daily regulated flow time series at Bellota, and then extracting annual maximums and computing the plotting positions of the resulting annual maximums, then interpolating the 0.5 to 0.04 ACE flow magnitudes. The derived values are shown in Table 8 below. The following steps were taken to produce hydrographs for these frequencies:

   a. For the target frequency, select a 1997 pattern hydrograph with the scale factor that provides the proper unregulated volume based on critical duration (1-day for Bellota) unregulated frequency curve.
   b. Based on the scale factor chosen in (a) above, obtain the corresponding Res-Sim output hydrograph at Bellota.
   c. For the target frequency, find the appropriate peak flow and volumes from the graphical regulated frequency curves (Table 8).
   d. Input the regulated hydrograph found in step b and the peak and volumes found in step c into HyBART in order to balance/adjust the hydrograph.

4. For the 0.02 to 0.002 ACE events, regulated peak flows were derived by the unregulated to regulated transform method in Figure 8. The procedure to derive final regulated hydrographs is described below.

   a. For the target frequency, select a 1997 pattern hydrograph with the scale factor that provides the proper unregulated volume based on critical duration (1-day for Bellota) unregulated frequency curve.
   b. Based on the scale factor chosen in (a) above, obtain the corresponding Res-Sim output hydrograph at Bellota.
   c. For the target frequency, find the appropriate peak flow (from the transform in Figure 8) and the concurrent volumes based on the DFC peak to volume regression analyses. DFC analyzed regulated peak flow to volume relationships from a regression analysis using multiple pattern events. The analysis was based on routing scaled historic flood patterns through Res-Sim and analyzing the resulting regulated flow hydrographs to obtain matching peak and volume data pairs. The data pairs were then used in a regression analyses, with peak being the known value x and volume being the prediction value y. Relationships were derived by DFC for regulated peak to regulated 1-, 3-, 7-, 15-, and 30-day volumes. The DFC analysis can be viewed in attachment 1 (see page 89).
   d. Input the regulated hydrograph found in step b and the peak and volumes found in step c into HyBART in order to balance/adjust the hydrograph.
   e. Create plot similar to the one shown in Figure 11 based on all hydrographs produced in HyBART including the 0.5 to 0.04 ACE events. Perform additional smoothing on the hydrograph volumes in HyBART for the 0.02 and 0.01 ACE frequency hydrographs to facilitate consistency between all frequencies so that the lines do not cross each other. The final adopted peak and volumes are plotted in Figure 11. Note: The 0.5 to 0.04 frequency hydrographs remain consistent with the family of graphical curves base on 23 years of data while the 0.005 and 0.002 ACE event hydrographs generally follow the DFC peak to volume relationships. Smoothing was performed on the 0.02 and 0.01 ACE hydrographs to achieve consistency in the plot in Figure 11.

In summary, Table 9 displays the final adopted regulated peak and volumes for each frequency event. Table 9 values were input to the program HyBART, a hydrograph balancing routine, along with pattern hydrographs from Res-Sim simulations of the 1997 flood. Simulated patterns were used rather than the actual observed pattern as the simulated and observed patterns are significantly different. The program HyBART creates balanced hydrographs that match the regulated peak flows in table 9 and follow the pattern of the 1997 flood event. HyBART creates a balanced hydrograph using all input peak flows and volumes. The Res-Sim model output hydrograph most closely associated with a specific frequency was selected as the input hydrograph for HyBART prior to balancing. For interest, the 1997 flood event pattern hydrographs for scale factors of the observed flood from 1.0 to 2.6 are shown in figure 12.

The resulting regulated flow hydrographs for the 0.5 annual chance exceedance probability (ACE) to 0.002 ACE events are consolidated in the spreadsheet: MSB-RegFlowFreq-1997SimPattern-Hydrographs.xlsx. A plot of the balanced regulated flows are shown below in figure 13. The hydrographs in figure 13 were eventually provided to PBI to route through their HEC-HMS model to compute additional hydrographs for index points downstream of Bellota in the Calaveras River watershed. The PBI model used a 1997 pattern storm to compute concurrent local runoff from sub-basins located downstream of the Bellota gage.

The DFC Limited Use flow-frequency curve was developed as a best fit analytical frequency curve of a 14 year period of historic data developed by subtracting lagged reservoir releases from observed flows at Bellota (reflective of local flow frequency only); whereas the flow-frequency for the 0.5 to 0.04 ACE events in table 8 were adopted from a graphical frequency curve based on a 23 year period of observed regulated flow (including local flow and reservoir releases at Bellota) after New Hogan dam was built. As only 23 years of record are available, the graphical curve is only useful for predicting peak and volumes for events *no rarer than* the 0.04 ACE (25-year return period). Although this is an apple to orange comparison, the values between the two frequency curves are substantially different only at the 0.5 ACE (2-year) frequency.

The 1997 event was chosen as the one event for producing specific frequency floods for the following reasons: a) It was a recent event in which hourly **hyetograph patterns** were available b) The various frequency hydrographs produced in this analysis became input to the HMS model produced by PBI, wherein the PBI rainfall runoff model produced concurrent runoff for areas downstream of the Bellota gage. c) In order to synchronize the two efforts, the same flood event (1997 flood) needed to be modeled in order for the timing of the total watershed runoff to be consistent with a real event.

9.0 Risk Parameter for the FDA Program

USACE policy is to use risk analysis as part of its planning and design processes. SPK’s Hydrology Section is assigned the task of providing hydrologic risk parameters for use in the Flood Damage Analysis (FDA) program. The assignment of a period of record for the flow frequency curve input into FDA for each study index point is important as it defines the confidence limits about the curve. Here are some guiding thoughts on that parameter for the lower Calaveras River watershed. The assigned period of record for Mormon Slough at Bellota and index points downstream (Mormon Slough and Calaveras River) is 52 years. The critical duration for Mormon Slough at Bellota was determined to be 1-day. As the runoff at Bellota is a combination of both reservoir releases (driven by volume of inflow into the dam) and local flow, using a volume duration curve (as opposed to a peak curve) is acceptable. The 1-day unregulated flow frequency curve at Bellota has a 104 year period of record. Factors for this decision are as follows:

The HEC-ResSim model ResSim version 3.1.8 RC4 used in this hydrologic analysis is quite adept at figuring out how to adjust reservoir releases to maintain downstream channel capacity while accounting for the rise and fall of the local flow hydrograph at the Bellota gage. This is due to 1) the reservoir release logic imbedded in HEC-ResSim is quite complex and iterative 2) the model is given perfect foresight into the future to see the local flow hydrograph. For these reasons, the model may be too efficient in using the full downstream channel capacity; whereas a human operator would be more cautious without the perfect foresight. Currently, the Water Management Section of SPK uses the real-time gage on Cosgrove Creek to predict local flow (Cosgrove Creek x 3.2 = total local flow at Bellota). This relationship was determined by the District Hydrologist working at SPK and was based on evaluation of historic data. Prior to real-time data being available at Cosgrove Creek, the regulated flow at Bellota did exceed 12,500 by more than a thousand cfs when the New Hogan Dam water managers miscalculated the local flow runoff during the 1986 flood. The Cosgrove Creek daily recording device was re-established in 1991 after a long period of being unavailable. While the availability of real-time Cosgrove Creek flow measurements aids in the local flow estimation, a human operator may still be reticent to assume that the “Cosgrove Creek measured flow times 3.2 = total local flow at Bellota” rule is infallible. As such a human operator would probably release less than the reservoir model, which would have the impact of filling up the reservoir storage faster. Under these circumstances, the reservoir would provide a lower level of protection from extremely rare floods since the downstream channel is being used less efficiently.

Another factor in this discussion is the method in which both reservoir inflow and local flow are scaled by the same factor for routing through the HEC-ResSim model. From experience with the Central Valley Hydrology Study, SPK has learned that scaling reservoir inflow and local flow by the same factor can sometimes result in a conservative estimate of local flow. The standard deviation and skew of reservoir inflow frequency curve and the local flow frequency curve are often quite different. Typically, the local flow frequency curve flattens out at the upper end while the reservoir inflow frequency curve keeps rising (higher standard deviation). This is because the upper watershed above the reservoir has higher rainfall depths in the mountains due to orographic effects, which results in a higher standard deviation (steeper slope of the curve). Scaling the local flow hydrograph and the reservoir inflow hydrograph by the same factor can result in local flow becoming increasingly rare in relation the reservoir inflow frequency. For example, scaling a specific flood by a factor (that originally had 0.04 ACE reservoir inflow frequency and 0.10 ACE local flow frequency) might result in a reservoir inflow and coincident local flow that are both equivalent to a 0.01 ACE event. This can change the dynamics of simulated floods as opposed to what might really happen in nature. Depending upon the watershed, SPK feels its current method could result in conservative estimates of local flow runoff.

The two issues above may have a cancelling effect upon one another, the first being less conservative and the last one being too conservative. Further sensitivity analyses or refinement of the hydrology could be done in PED phase to assess the above concerns. For the feasibility study, it is currently recommended that the period of record assigned to the Mormon Slough at Bellota gage in the FDA program be 52 years (which is half the unregulated frequency curve period of record of 104 years at this location). This 52 year period of record is also applicable to points downstream of the Bellota gage because 1) much of the downstream watershed has levees 2) there are only a few locations where additional local flow enters 3) the bulk of the water in the levees comes from upstream of Bellota.

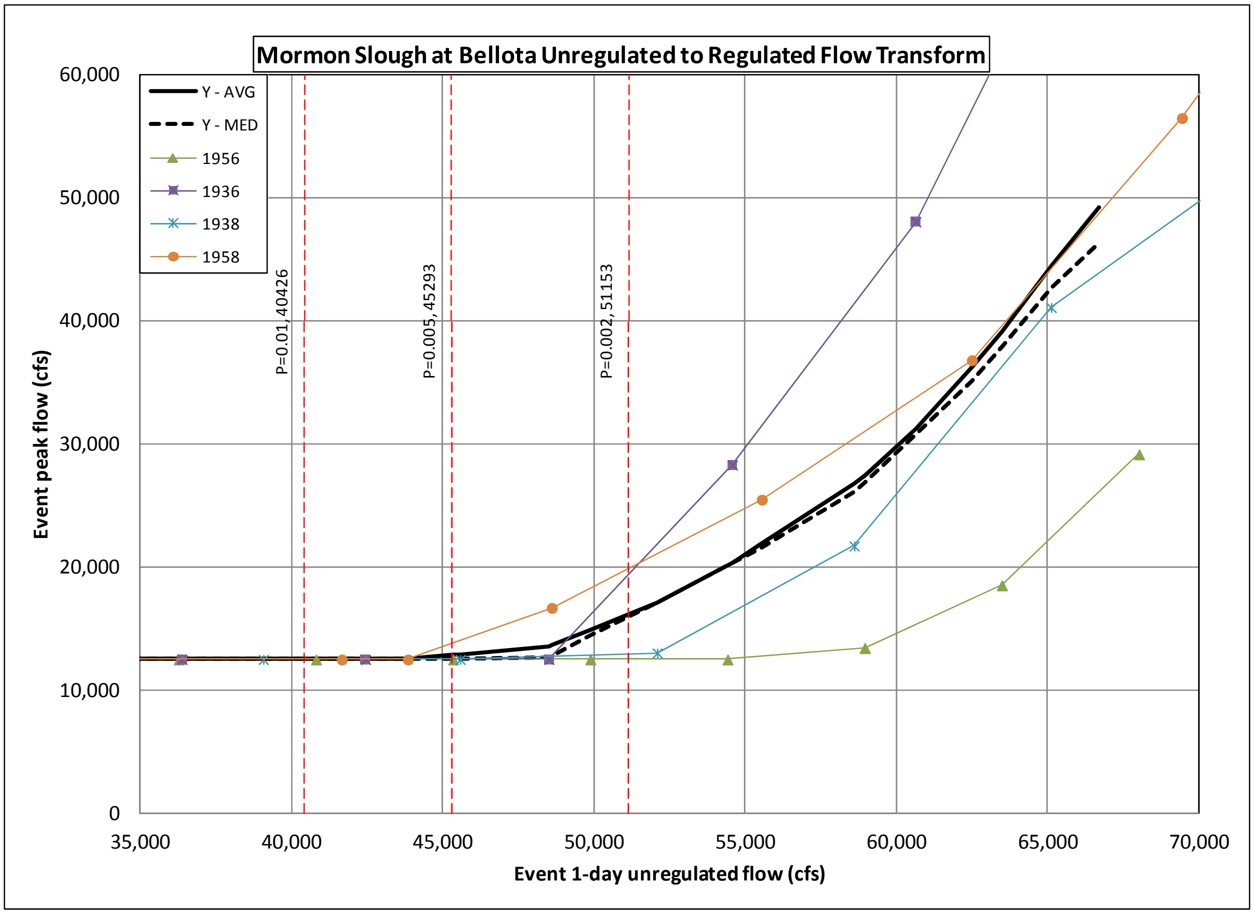


Figure 8. Unregulated 1-Day Flow to Regulated Peak Flow Transform at Bellota.

N-probability Events				
1/AEP	**AEP**	**Unregulated cfs**	**AVG transform**	**MEDIAN transform**
50	0.02	35,185	12,500	12,500
100	0.01	40,426	12,500	12,500
200	0.005	45,293	12,818	12,500
500	0.002	51,153	16,188	15,961

Table 7: 1-day Unregulated Flow and Regulated Peak Flow Comparison at Bellota.

Note: The median transform was adopted for Bellota as it appears to better fit the scaled event traces.

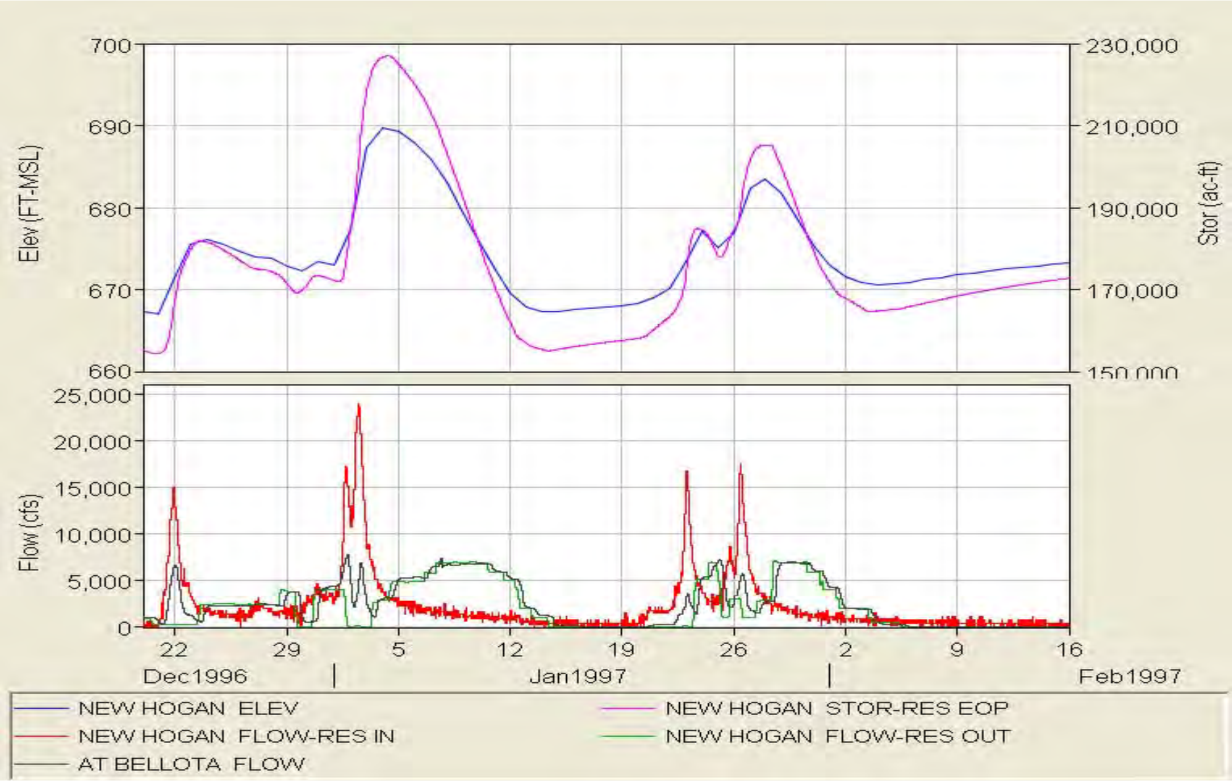


Figure 9. Actual operation of New Hogan dam during the 1997 flood event.

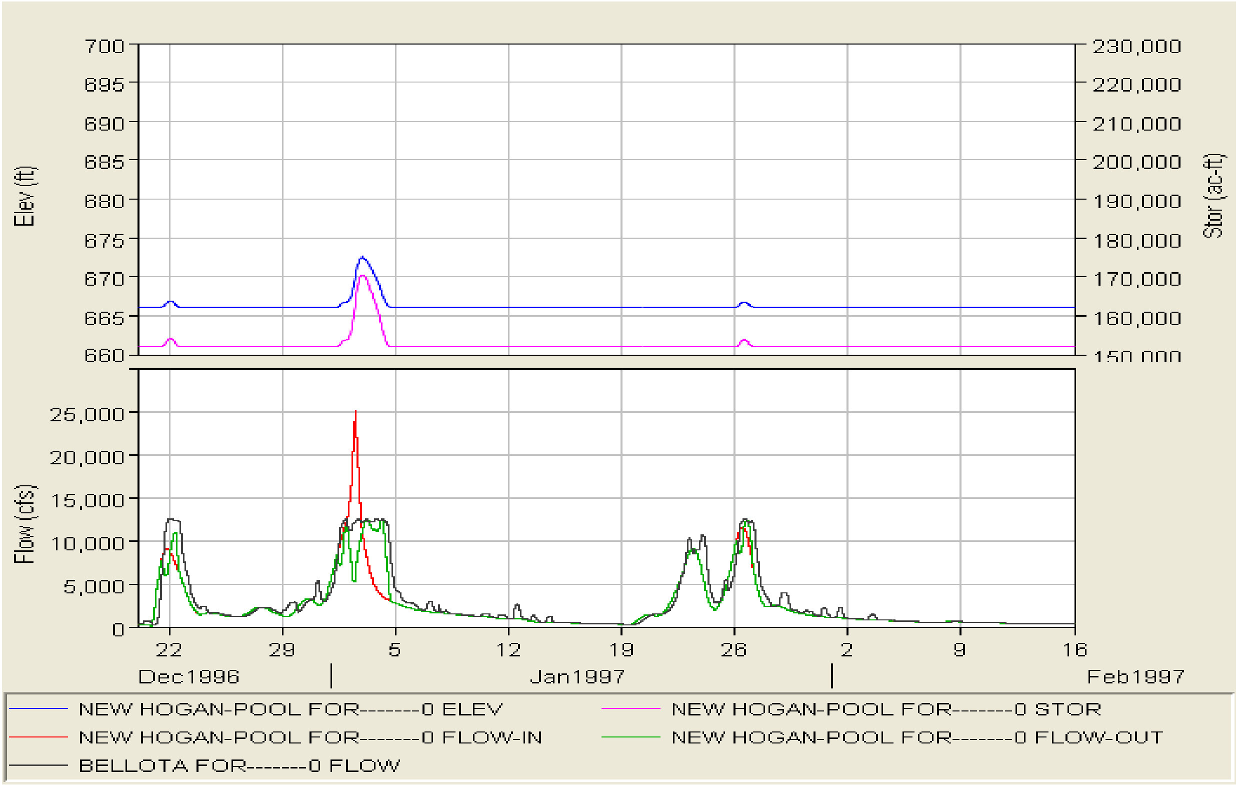


Figure 10. Simulated operation of New Hogan dam for the 1997 flood event.

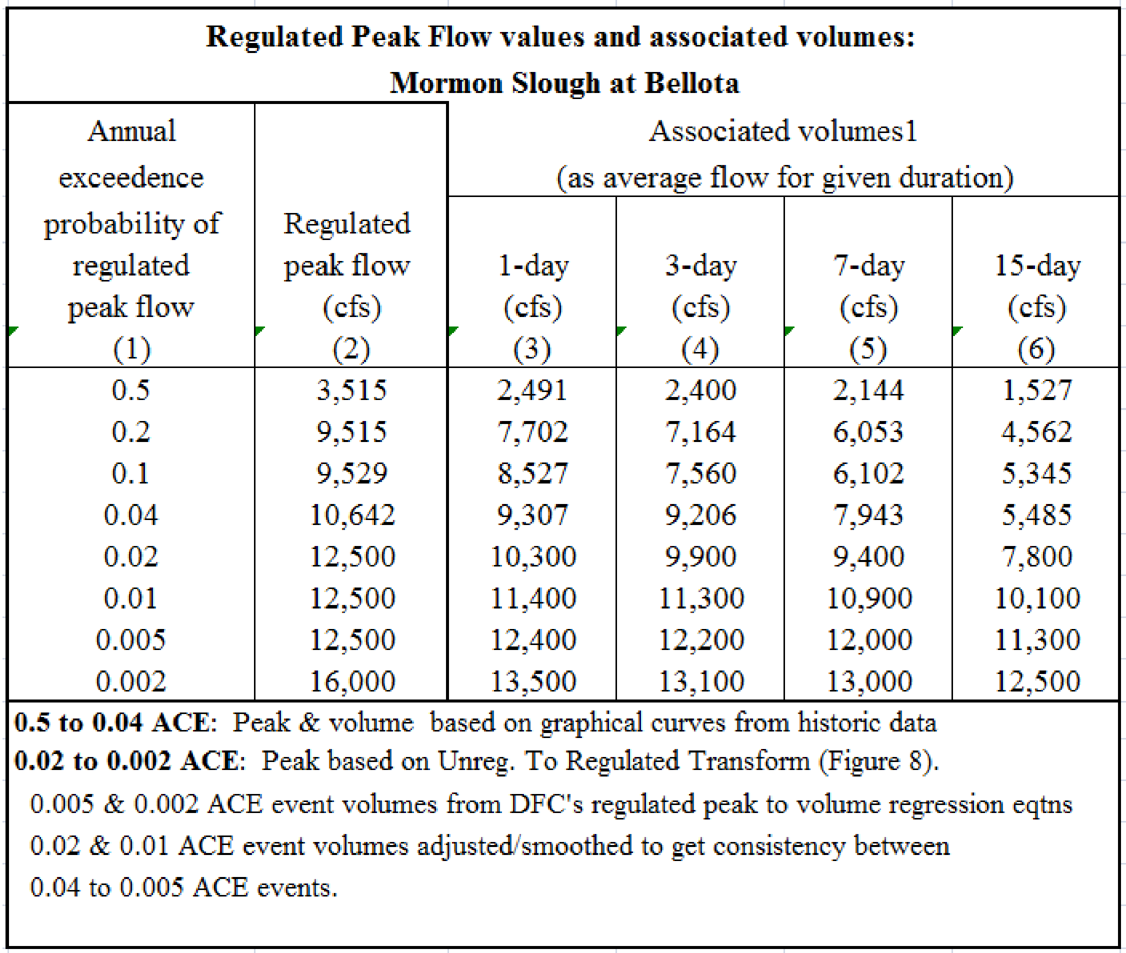
Bellota n-Day Max Flows		Peak	1day	3day	7day	15day	1/Prob
No.	Prob	Y-Axis	Y-Axis	Y-Axis	Y-Axis	Y-Axis	
1	0.9583	738	105	91	64	62	1.04
2	0.9167	959	506	299	187	108	1.09
3	0.8750	1,284	617	387	216	173	1.14
4	0.8333	1,297	692	502	315	175	1.20
5	0.7917	1,404	1043	586	319	202	1.26
6	0.7500	1,463	1131	734	385	219	1.33
7	0.7083	2,144	1176	760	422	234	1.41
8	0.6667	2,186	1239	776	423	267	1.50
9	0.6250	2,228	1259	804	433	279	1.60
10	0.5833	2,343	1791	891	604	348	1.71
11	0.5417	3,016	1832	1,120	639	361	1.85
12	0.5000	3,515	2491	2,400	2,144	1,527	2.00
13	0.4583	4,439	3309	3,055	2,530	1,575	2.18
14	0.4167	4,501	3895	3,579	2,691	2,396	2.40
15	0.3750	5,111	3978	3,701	3,168	2,481	2.67
16	0.3333	6,820	4108	3,793	3,449	2,923	3.00
17	0.2917	7,833	6915	6,740	4,916	3,260	3.43
18	0.2500	9,499	7635	6,977	5,160	3,350	4.00
19	0.2083	9,514	7647	7,138	6,050	4,509	4.80
20	0.1667	9,519	7938	7,277	6,067	4,786	6.00
21	0.1250	9,635	8071	7,996	6,104	4,991	8.00
22	0.0833	9,876	8522	8,021	6,919	5,288	12.00
23	0.0417	10,602	9266	9,145	7,891	5,475	24.00

Interpolated Values							**1/Prob**
No.	**AEP**	**Peak**	**1day**	**3day**	**7day**	**15day**	
12	0.500	3515	2491	2400	2144	1527	2
19-20	0.200	9515	7702	7164	6053	4562	5
20-21	0.100	9529	8527	7560	6102	5345	10
22-23	0.040	10642	9307	9206	7943	5485	25
24	0.020	12,500	10,300	10,300	9,400	7,800	50
25	0.010	12,500	11,400	11,300	10,900	10,100	100
26	0.005	12,500	12,400	12,400	12,400	12,400	200
27	0.002	16,000	13,500	13,100	13,000	12,500	500

Values in Yellow are from Transform Curve and Table

Table 8. Peak, 1-, 3-, 7-, and 15-day Flows for Mormon Slough at Bellota from historic graphical curve.

Note: 0.50 to 0.04 ACE values derived from graphical curve of 1988 to 2010 water year data. 0.02 to 0.002 ACE highlighted in yellow are derived from reservoir simulations of scaled events



Regulated Peak Flow values and associated volumes:
Mormon Slough at Bellota

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes1 (as average flow for given duration)			
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)
0.5	3,515	2,491	2,400	2,144	1,527
0.2	9,515	7,702	7,164	6,053	4,562
0.1	9,529	8,527	7,560	6,102	5,345
0.04	10,642	9,307	9,206	7,943	5,485
0.02	12,500	10,300	9,900	9,400	7,800
0.01	12,500	11,400	11,300	10,900	10,100
0.005	12,500	12,400	12,200	12,000	11,300
0.002	16,000	13,500	13,100	13,000	12,500

0.5 to 0.04 ACE: Peak & volume based on graphical curves from historic data

0.02 to 0.002 ACE: Peak based on Unreg. To Regulated Transform (Figure 8).

0.005 & 0.002 ACE event volumes from DFC's regulated peak to volume regression eqtns
0.02 & 0.01 ACE event volumes adjusted/smoothed to get consistency between
0.04 to 0.005 ACE events.

Table 9. Regulated Peak Flows and Associated Volumes for Mormon Slough at Bellota.

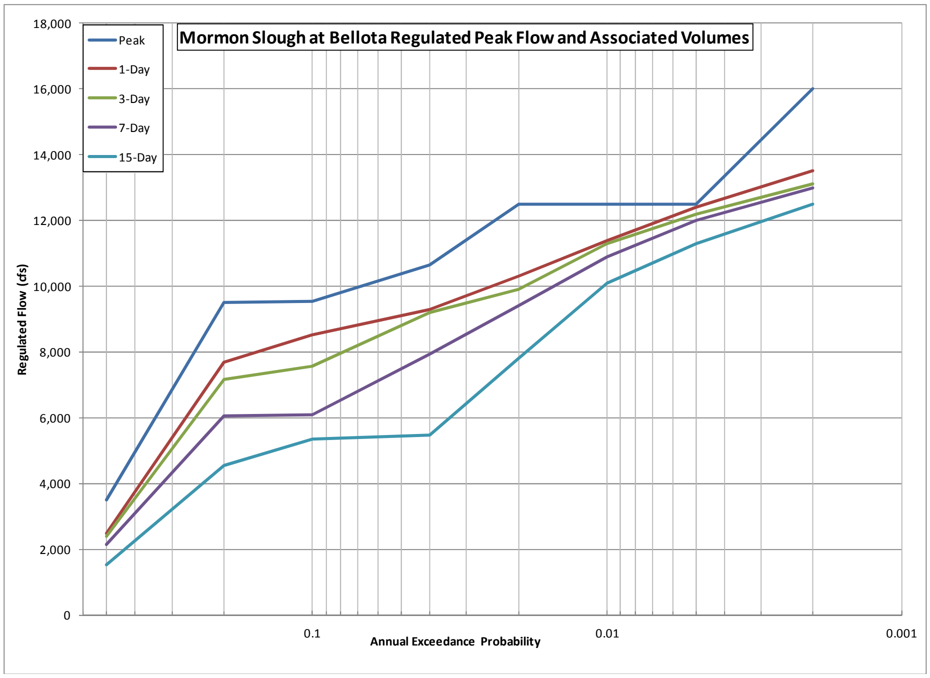


Figure 11. Regulated Peak Flow and Associated Volumes at Mormon Slough at Bellota.

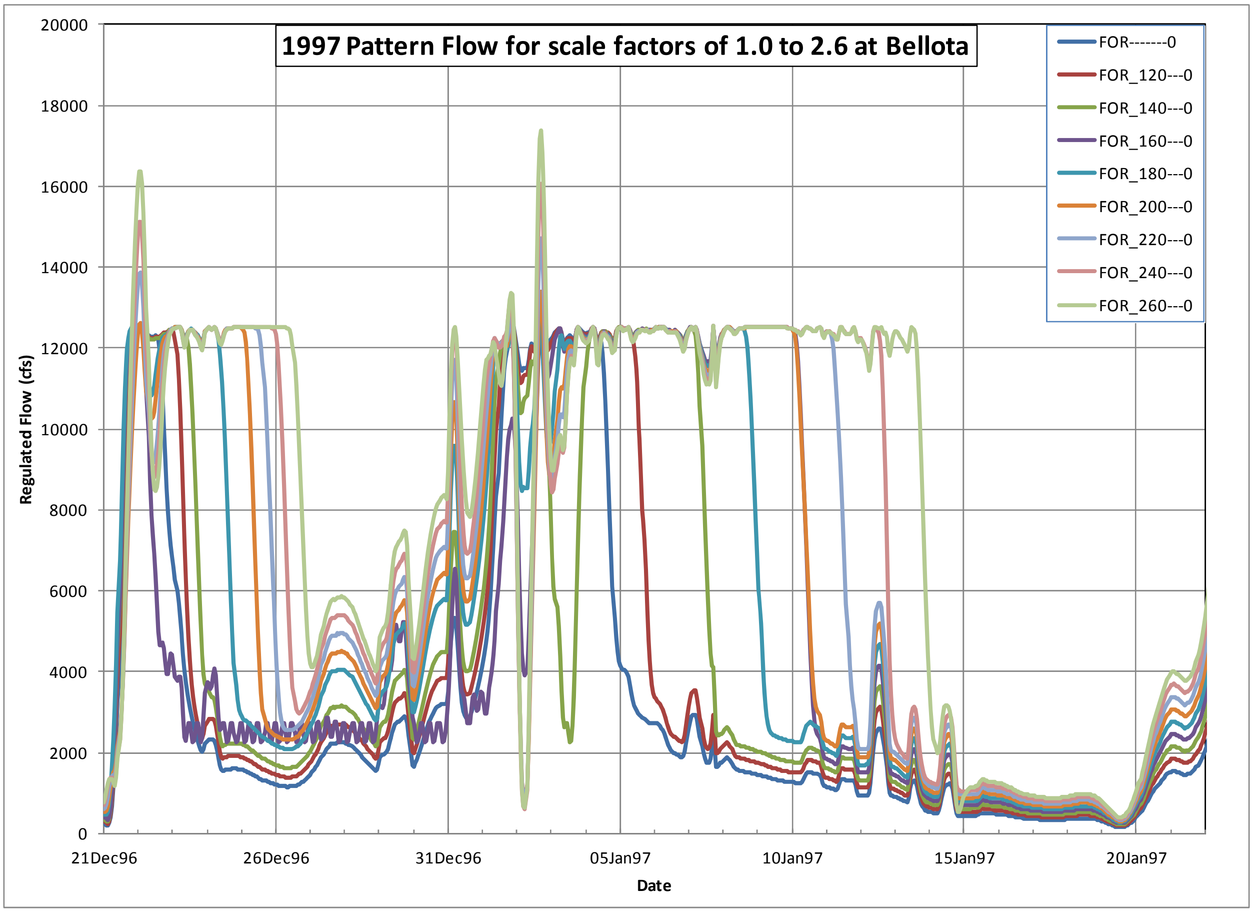


Figure 12. 1997 Pattern Flows for scale factors from 1.0 to 2.6 at Bellota

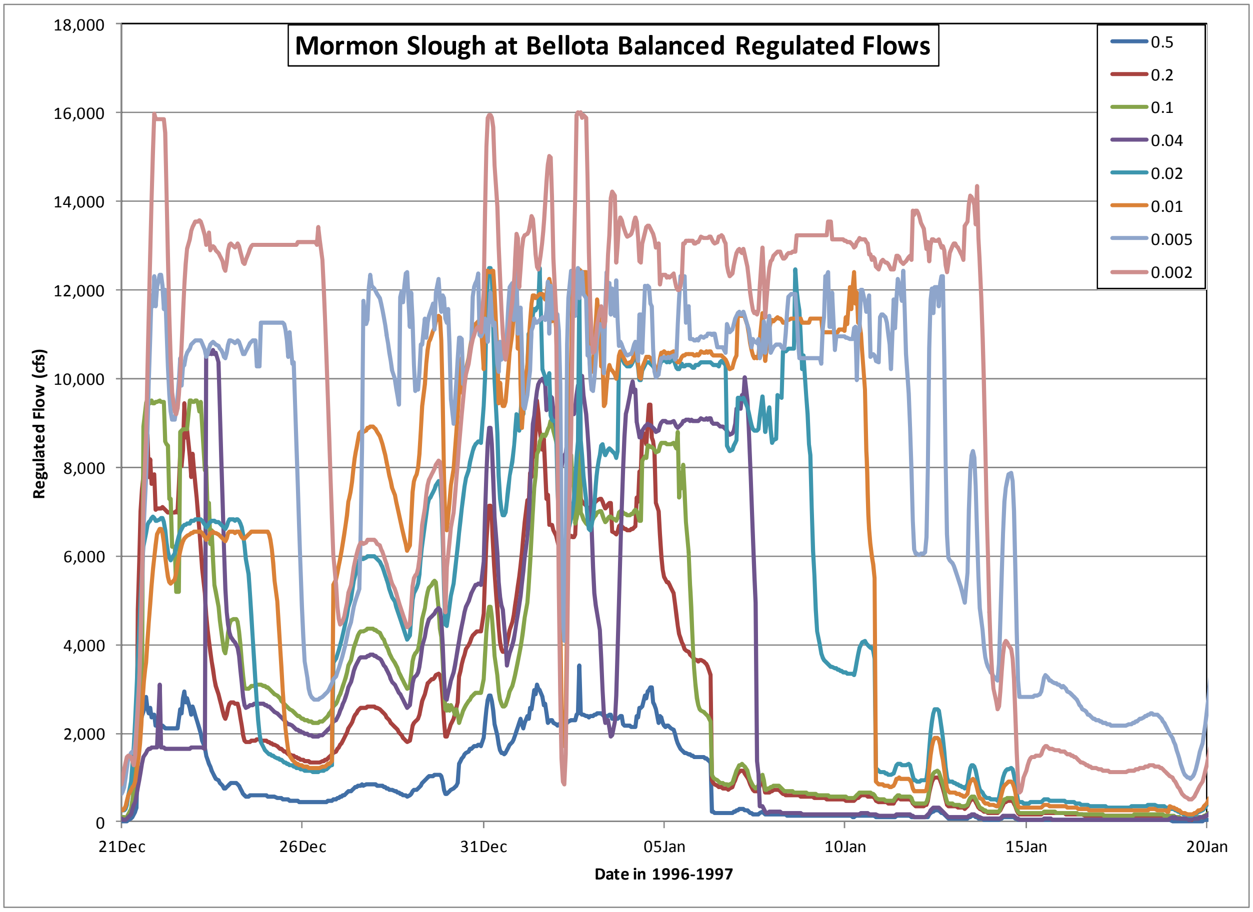


Figure 13. Final Balanced 1997 Pattern Hydrographs at Bellota

Associated files: In

In Corps directory:
W:\Studies\SJQ-020\LSJQR\Working Files\RegulatedFlows\NEW_Data

Filenames:
MSB-RegFlowFreq-1997Event-Hydrographs-30Jan2012.xlsx,

Bellota_TRANSFORM TEMPLATE – DRAFT 30Jan2012.xlsm,

Reconstruct-DFCE_Table4-30Jan2012.xlsx,

Ratios-BellotaLocal-to-ResIn&Cosgrove.xlsx,

Bellota-nday-GraphicalFit-01Feb2012.xlsx,

And this file: LSJR-FS-RegulatedFlows-07Feb2012.docx.

Appendix 1 - Attachment 1

Lower San Joaquin River Feasibility Study
Calaveras River above Bellota
Hydrologic Analysis

US Army Corps
of Engineers.

Sacramento District

07 April 2014

Lower San Joaquin River feasibility study: Calaveras River frequency analysis and hydrographs

June 20, 2011

US Army Corps of Engineers Sacramento District
W91238-09-D-0004, TO 0004

David Ford Consulting Engineers, Inc.
2015 J Street, Suite 200
Sacramento, CA 95811
Ph. 916.447.8779
Fx. 916.447.8780

Engineer's certification

I, Michael Konieczki, hereby certify on 6/20/2011 that I am a professional engineer licensed in the state of California and that the accompanying report was prepared by me or under my supervision.

MICHAEL V. KONIECZKI
No. 74357
Exp. 6/30/2013
CIVIL

Contents

Tables

Figures

Executive summary

Situation

In the lower San Joaquin River feasibility study (LSJR FS) the Sacramento District of the US Army Corps of Engineers (Corps) and the San Joaquin Area Flood Control Agency (SJAFCA) are studying alternative flood risk reduction measures that will provide protection against a flood with a probability of exceedence in any given year equal 0.005 (i.e., a "200-year flood").

The LSJR FS includes hydrologic analyses of the study region. This same region is also being studied in conjunction with a separate project to map the floodplains adjacent to the federal-state levee system in the Central Valley. Because the products of the various hydrologic analyses being conducted in the lower San Joaquin River basin will be used for several purposes by multiple agencies and stakeholders, the firms and agencies involved are using consistent analytical procedures and methods where possible. These procedures are specified in the *Sacramento and San Joaquin river basins: Procedures for hydrologic analysis* (hereinafter, *Procedures document*) and the *Central Valley hydrology study (CVHS): Technical procedures document* (hereinafter, *Technical procedures document*). Attachment 1 provides a table that explains how the procedures detailed in the present document align with the procedural steps detailed in the *Procedures document* and the *Technical procedures document*.

In this report we detail our hydrologic analyses at 2 sites on the Calaveras River: (1) New Hogan Reservoir, and (2) New Hogan's operation point at Bellota. These sites are shown in Figure 1.

Tasks

Our tasks were to: (1) develop a regulated flow-frequency curve and associated volumes at each location, and (2) derive an "expected" outflow hydrograph at New Hogan Reservoir.

Actions

To complete the tasks above, we:

- Developed unregulated volume-frequency curves at New Hogan Reservoir and Bellota following the procedures in *Guidelines for determining flood flow frequency, Bulletin 17B* (IACWD 1982) and EM 1110-2-1415 (USACE 1993) and using a regional skew provided by the Corps.
- Simulated reservoir releases and routed historical and scaled floods, including local flows, on the Calaveras River using an HEC-ResSim model provided by the Corps.
- Fitted, at each location, flow transforms to the event maxima datasets identified from the unregulated flow and simulated release time series.
- Developed, at each location, a regulated flow-frequency curve and associated volumes by applying the flow transforms.
- Developed "expected" outflow hydrographs for New Hogan Reservoir for 8 flood frequencies: p=0.5, p=0.2, p=0.10, p=0.05, p=0.02, p=0.01, p=0.005 and p=0.002. (Here the term expected hydrograph refers to a

New Hogan Reservoir outflow hydrograph with a peak flow that matches the regulated flow-frequency curve and with associated volumes matching those from the family of characteristic curves corresponding to the given regulated peak flow.)

Results

The results of our analysis include:

- Unregulated volume-frequency curves for New Hogan Reservoir (as shown in Figure 2).
- Unregulated volume-frequency curves for the Calaveras River at Bellota (as shown in Figure 3).
- Unregulated-regulated flow transform for New Hogan Reservoir (as shown in Figure 4).
- Regulated flow-frequency curve and associated volumes for New Hogan Reservoir (as shown in Table 1 and in Table 2).
- Unregulated-regulated flow transform for the Calaveras River at Bellota (as shown in Figure 5).
- Regulated flow-frequency curve and associated volumes for the Calaveras River at Bellota (as shown in Table 3 and in Table 4).
- Expected hydrograph properties for New Hogan Reservoir. (Note: these are the same values shown in Table 1).

In addition, these intermediate values and information are included with the original report on DVD:

- HEC-DSS time series of the floods-of-records.
- HEC-DSS time series of the scaled historical floods.
- HEC-DSS time series of developed local flows below New Hogan Reservoir (detailed in Attachment 2).
- The tabulated event maxima datasets for the 2 analysis sites.
- Simulated reservoir releases and routed flows from the HEC-ResSim reservoir simulation model.
- Tabulated unregulated-regulated flow transforms for the 2 analysis sites.
- Tabulated families of regulated characteristic curves for the 2 analysis sites.

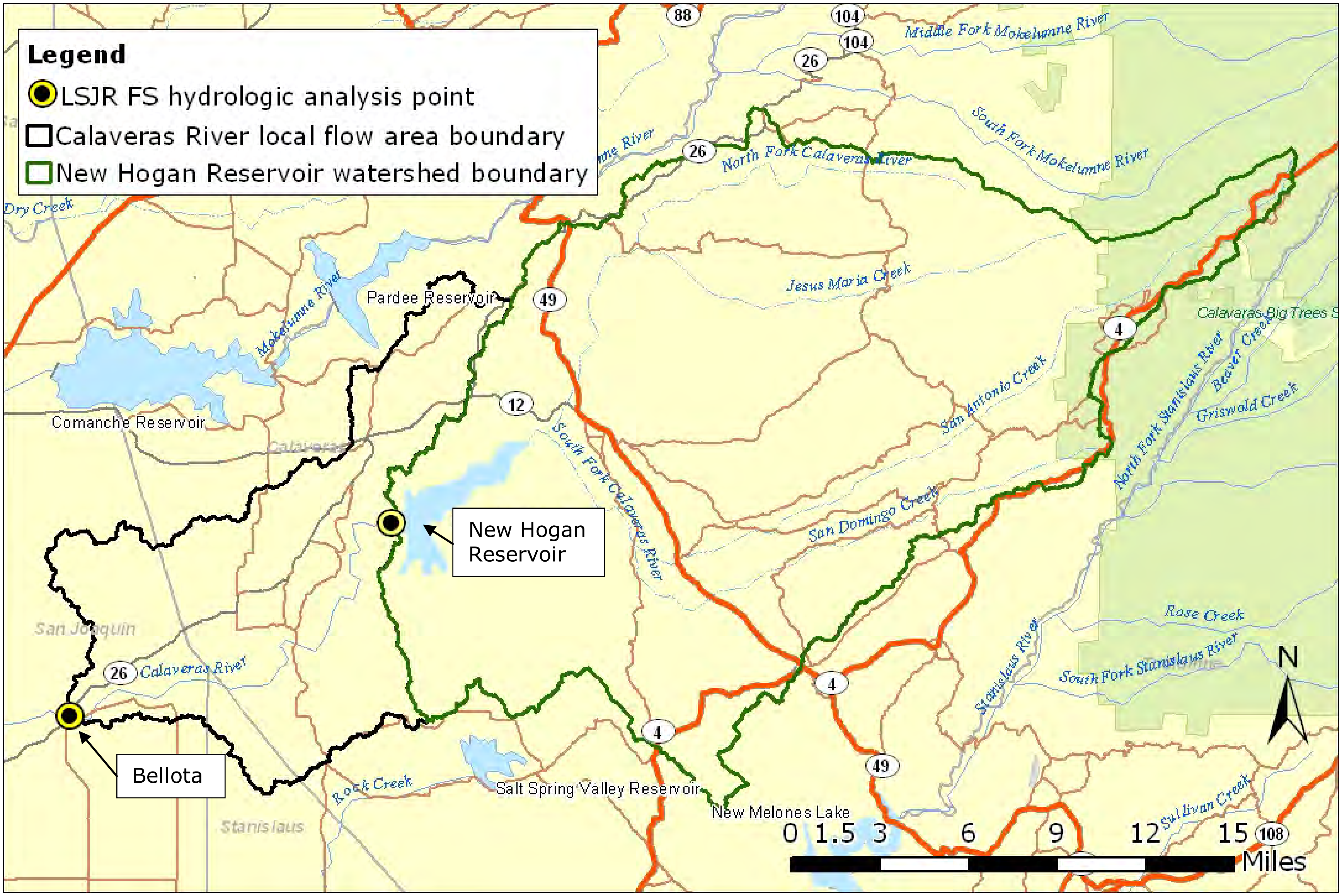
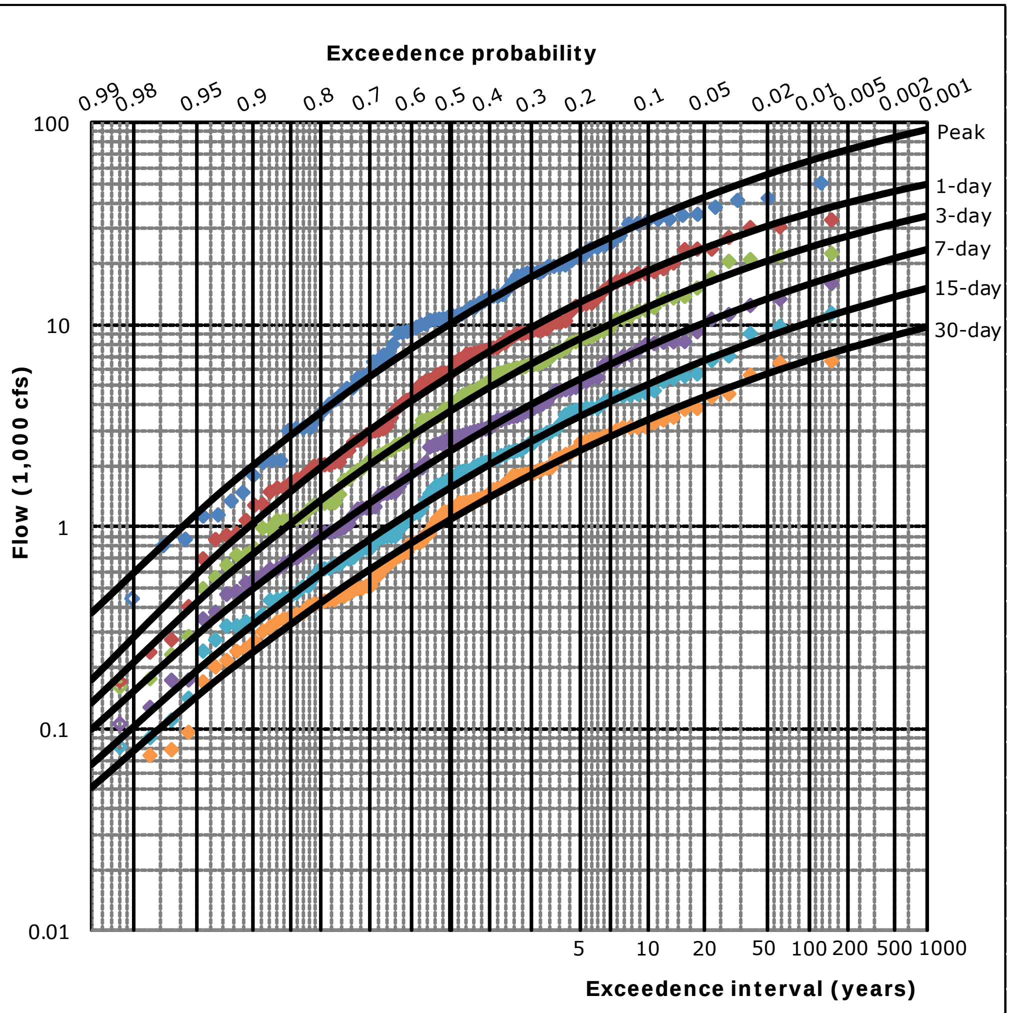


Figure 1. Calaveras River study area



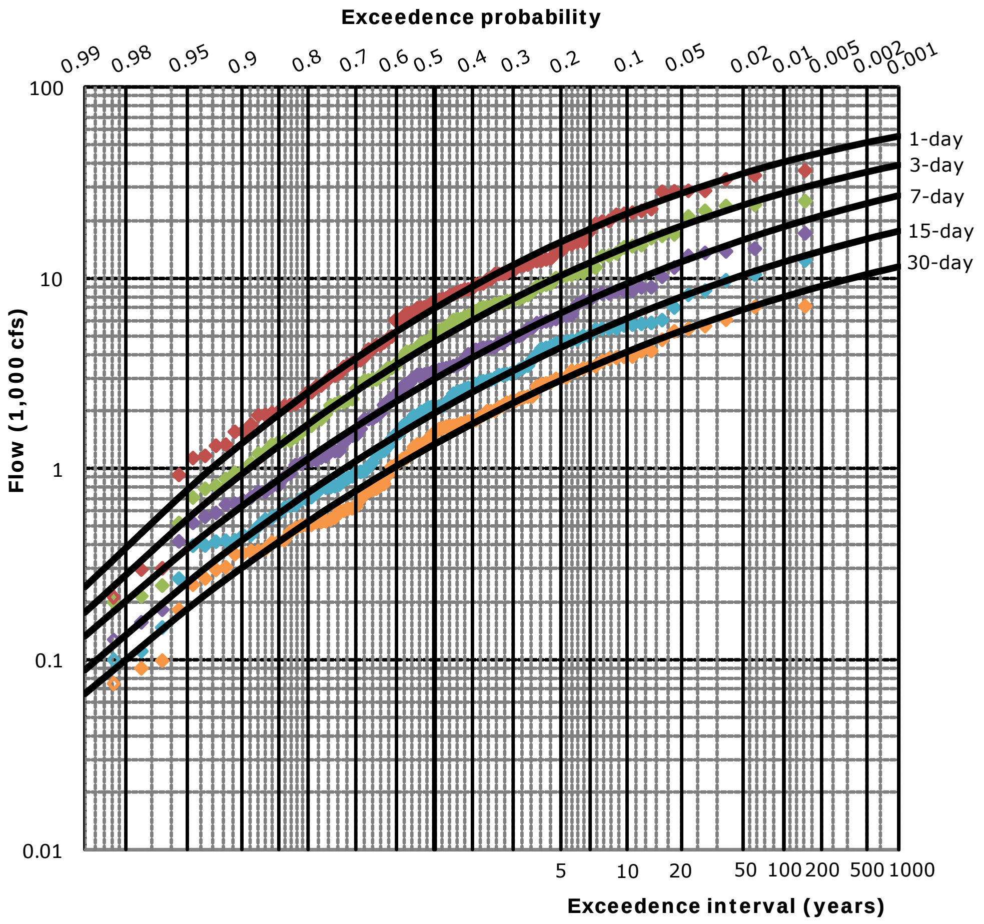
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.946	0.482	-0.727
1-day	3.685	0.501	-0.794
3-day	3.518	0.487	-0.732
7-day	3.324	0.477	-0.651
15-day	3.146	0.473	-0.656
30-day	2.988	0.457	-0.659

Notes:

- Median plotting positions.
- Drainage area: 363 sq. miles.
- Period of systematic record: 1907-2010.
  (Peak flow data intermittent 1930-2010).
- Record lengths
  Peak flows: 86 years.
  Volumes: 104 years.
- Regional skew values developed by USGS.
- Hollow points are censored events.

Figure 2. Unregulated frequency curves: New Hogan Reservoir



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.775	0.482	-0.810
3-day	3.608	0.475	-0.753
7-day	3.417	0.464	-0.666
15-day	3.240	0.461	-0.671
30-day	3.079	0.448	-0.668

Notes:

- Median plotting positions.
- Drainage area: 473 sq. miles.
- Period of systematic record: 1907-2010.
- Record length: 104 years.
- Regional skew values developed by USGS.
- Hollow points are censored events.

Figure 3. Unregulated frequency curves: Calaveras River at Bellota

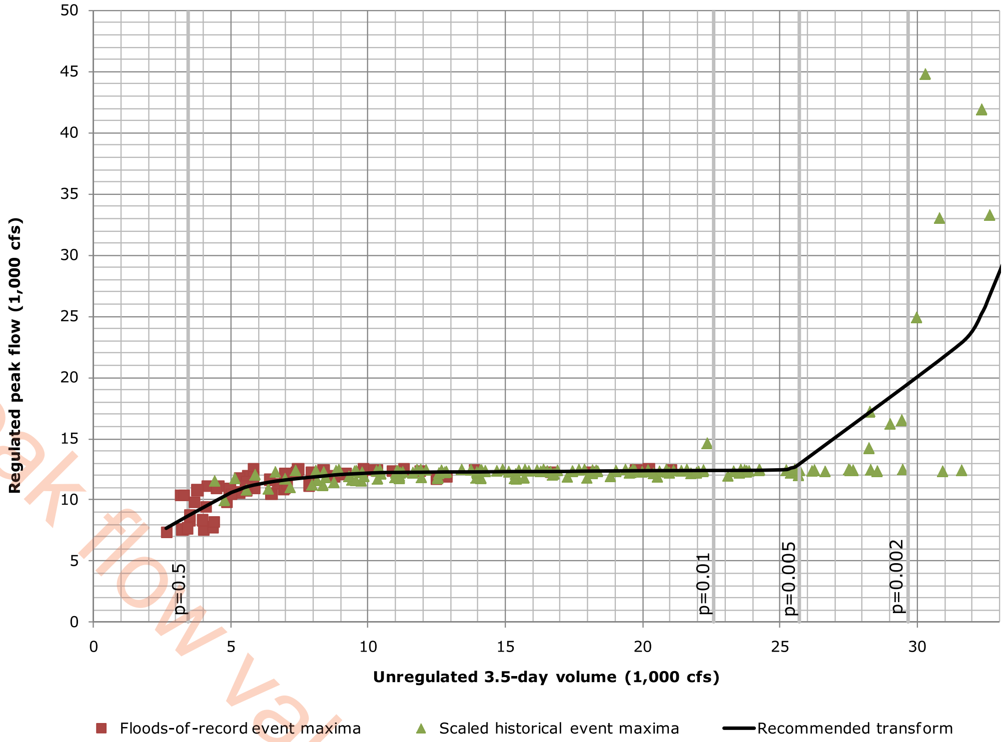


Figure 4. Unregulated-regulated flow transform: New Hogan Reservoir

Table 1. Regulated peak flow-frequency quantiles: New Hogan Reservoir

Annual exceedence probability (1)	**1/annual exceedence probability (2)**	**Regulated peak flow (cfs) (3)**
0.500	2	8,664
0.200	5	11,812
0.100	10	12,214
0.050	20	12,266
0.020	50	12,334
0.010	100	12,367
0.005	200	12,903
0.002	500	19,555

Table 2. Regulated peak flow values and associated volumes: New Hogan Reservoir

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration)				
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)	30-day (cfs) (7)
0.500	8,664	6,212	4,188	2,720	1,843	1,199
0.200	11,812	11,625	10,634	7,457	4,994	3,096
0.100	12,214	12,107	11,582	9,098	5,909	3,963
0.050	12,266	12,140	11,607	9,312	6,032	4,157
0.020	12,334	12,283	11,880	10,275	7,045	5,120
0.010	12,367	12,300	11,916	10,459	7,411	5,263
0.005	12,903	12,900	12,893	12,876	12,026	9,283
0.002	19,555	19,555	19,549	17,462	12,445	9,463

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at New Hogan Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

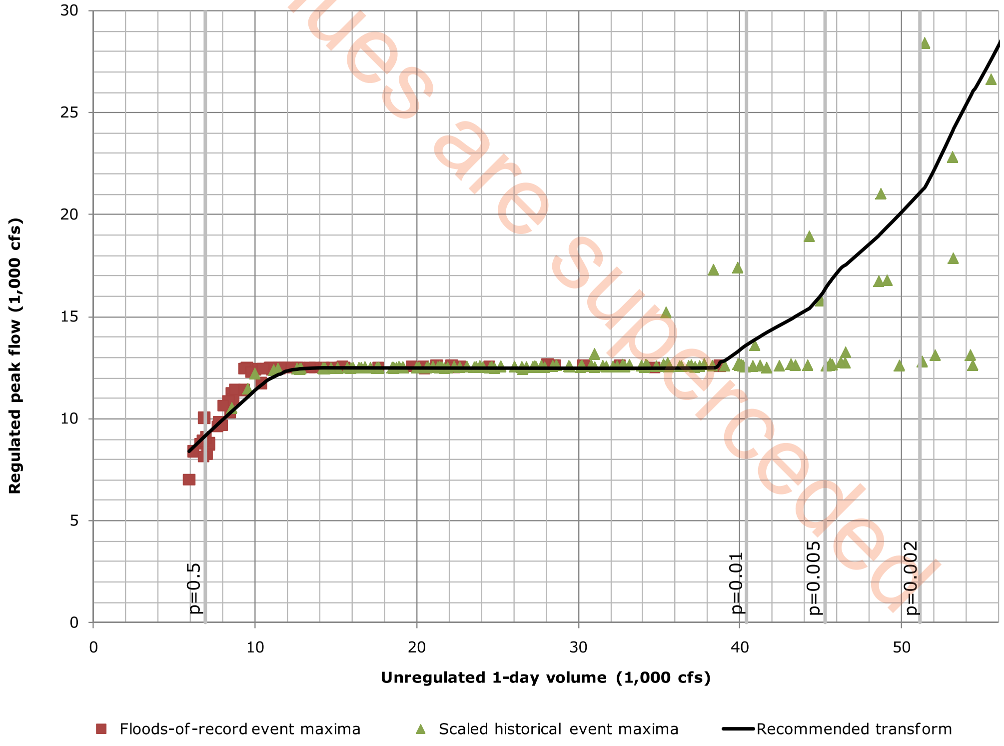


Figure 5. Unregulated-regulated flow transform: Calaveras River at Bellota

Table 3. Regulated peak flow-frequency quantiles: Calaveras River at Bellota

Annual exceedence probability (1)	**1/annual exceedence probability (2)**	**Regulated peak flow (cfs) (3)**
0.500	2	9,163
0.200	5	12,500
0.100	10	12,500
0.050	20	12,500
0.020	50	12,500
0.010	100	13,634
0.005	200	16,409
0.002	500	21,107

Table 4. Regulated peak flow values and associated volumes: Calaveras River at Bellota

Annual exceedence probability of regulated peak flow (1)	**Regulated peak flow (cfs) (2)**	**Associated volumes[1] (as average flow for given duration)**				
		1-day (cfs) (3)	**3-day (cfs) (4)**	**7-day (cfs) (5)**	**15-day (cfs) (6)**	**30-day (cfs) (7)**
0.500	9,163	7,271	4,852	3,163	2,127	1,372
0.200	12,500	12,500	12,500	12,500	12,500	10,000
0.100	12,500	12,500	12,500	12,500	12,500	10,000
0.050	12,500	12,500	12,500	12,500	12,500	10,000
0.020	12,500	12,500	12,500	12,500	12,500	10,000
0.010	13,634	13,174	13,141	12,545	12,515	10,001
0.005	16,409	13,367	13,300	13,300	12,553	10,002
0.002	21,107	15,106	15,106	13,930	12,631	10,005

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at New Hogan Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Watershed description

The watershed that is the subject of this report—the Calaveras River basin—is part of the lower San Joaquin River basin. It is located in Calaveras, San Joaquin, and Stanislaus counties. Located on Calaveras River approximately 28 miles upstream of Stockton, CA, is New Hogan Reservoir, a multipurpose facility with water supply, recreation, and flood control requirements.

The 707 mi^2 Calaveras River basin is shown in Figure 6. The north and south forks of the Calaveras River meet just east of New Hogan Reservoir and continue flowing into the reservoir. The basin comprises 3 major areas:

- The area above New Hogan Reservoir, which includes 363 mi^2 of relatively low-lying area on the western slopes of the Sierra Nevada. Elevations range from 550 ft at the dam to approximately 6,000 ft at the highest point.
- The 110 mi^2 area between New Hogan Reservoir and the downstream operation point at Bellota (the bifurcation of the Old Calaveras River and Mormon Slough approximately 18 miles downstream of the reservoir). The elevation at Bellota is approximately 130 feet.
- The remaining 234 mi^2 area of the Calaveras River and Mormon Slough watershed from Bellota to the confluence with the San Joaquin River. This portion of the watershed is low and flat with little topographic relief. Note: hydrological analysis of this region is being completed by other consultants and agencies and is therefore beyond the scope of the analysis described here.

The channel capacity downstream of New Hogan Reservoir is 12,500 cfs and the reservoir operates to limit flow to this value downstream of the dam and at Bellota (USACE 1983). A control structure exists at Bellota to divert the majority of flows into Mormon Slough. Downstream of this structure lies the Old Calaveras River channel, which is overgrown with vegetation. Flow is diverted into the Old Calaveras River when flow at Bellota reaches 13,500 cfs (USACE 1983).

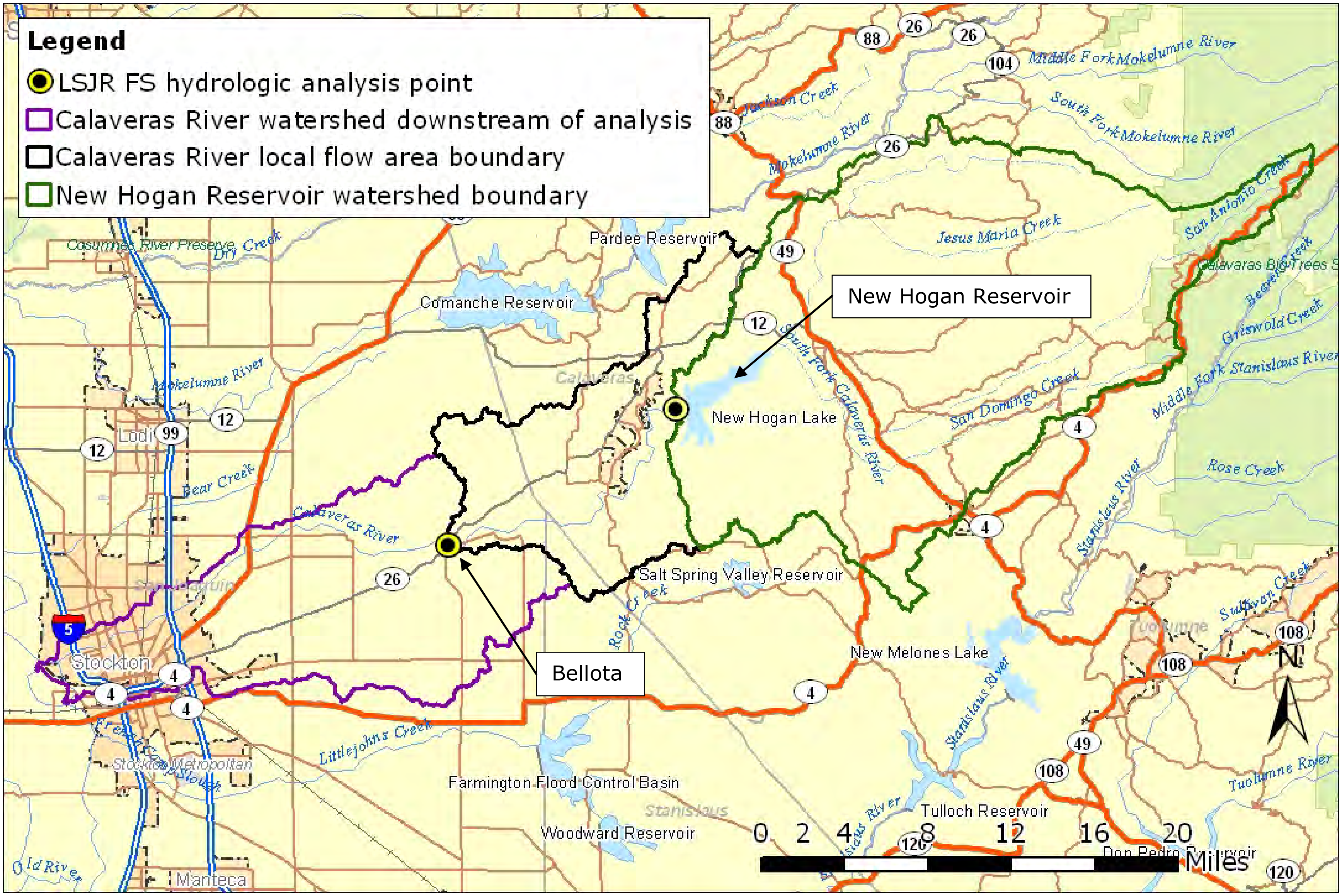


Figure 6. Lower San Joaquin River feasibility study area: Calaveras River

Analysis procedure

Overview of CVHS procedure

The primary tasks for the CVHS are described in the *Procedures document*. More detail for these tasks is provided in the *Technical procedures document*. As a review of those tasks and to provide context for the procedures used in this analysis, here we summarize the procedure steps and categorize them into 2 groups. They are:

- Group 1. Unregulated frequency analysis at selected points. This comprises *Procedures document* Task 1, Task 2 (reservoir simulation models), Task 3, and Task 4. (References throughout this report to numbered tasks use numbers from the *Procedures document*.)
- Group 2. Description of the effects of the regulation (flood control) system to allow conversion of the unregulated frequency curves to regulated flow-frequency curves at the same selected points. This comprises *Procedures document* Task 2 (channel routing models), Task 5, Task 6, and Task 7.

Group 1 focuses on completing a frequency analysis to characterize the annual exceedence probability of a given flow (unregulated). Thus, all statements of probability originate here.

Group 2 reflects the impact of regulation in the system. This second group accounts for various historical storm distributions and reservoir operations, with an emphasis on large events.

Application to the lower San Joaquin River feasibility study

In Figure 7, we illustrate the general work flow of the analysis procedure as applied to the LSJR FS. In this document we note before each analysis step the corresponding CVHS procedures task applicable, if any.

For unregulated frequency analysis for the 2 sites on the Calaveras River, New Hogan Reservoir and Bellota, we:

- (Task 1) Obtained reservoir inflow and streamgage data for use in developing the unregulated flow time series from the Corps.
- (Task 2) Obtained accepted reservoir simulation and channel routing models from the Corps.
- (Task 3) Developed unregulated flow time series at each location corresponding to a period-of-record of floods. This step includes the development of local flows for the ungaged area between New Hogan Dam and Bellota.
- (Task 4) Computed and adopted unregulated 1-, 3-, 7-, 15-, and 30-day volume-frequency curves at each location. Note: we developed peak unregulated flow-frequency curves for New Hogan Reservoir for completeness; they are not required for this analysis.

For regulated system analysis for the 2 sites on the Calaveras River we:

- (Task 5) Developed regulated flow time series at each location by simulating and routing reservoir releases. Here, historical and scaled historical events were used in development of the time series.

- (Task 6) Fitted flow transforms. First, the unregulated and corresponding regulated event maxima datasets were identified (these are data points to which the transforms were fitted). Then, the critical duration of each analysis location was determined using these series. The flow transforms were then developed by fitting curves to the event maxima datasets. Note here, the term flow transforms refers to: (1) the unregulated-regulated flow transform, and (2) the family of regulated characteristic curves.
- (Task 6.4) Applied flow transforms to develop a regulated peak flow-frequency curve and associate volumes for the 1-, 3-, 7-, 15-, and 30-day durations at each location.

For development of the expected hydrograph properties for New Hogan Reservoir outflows we identified the peak regulated flows and associated regulated volume-duration characteristics for 8 exceedence probabilities: p=0.5, p=0.2, p=0.1, p=0.05, p=0.02, p=0.01, p=0.005, and p=0.002.

Attachment 1 provides a table explaining how the procedures detailed here align with the procedural steps detailed in the *Procedures document* and the *Technical procedures document*.

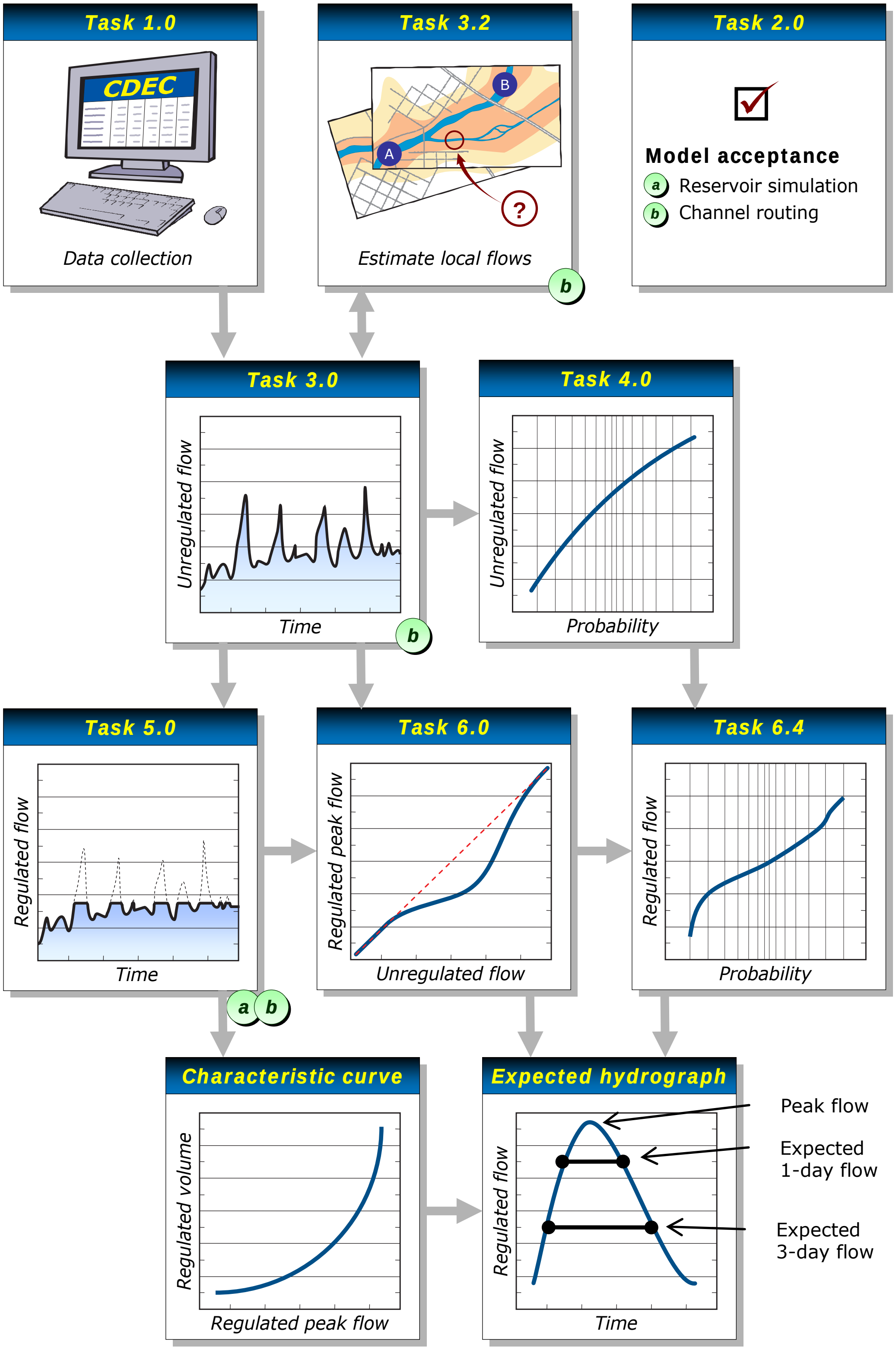


Figure 7. LSJR analysis procedure workflow

Unregulated flow time series development

We constructed unregulated flow time series at each analysis location in the study area and fitted unregulated volume-frequency curves to these series using procedures that are consistent with Corps guidance.

The locations most upstream at which we developed unregulated flow time series were the project reservoirs. Thus, for unregulated conditions, the reservoir inflows were needed.

For development of the unregulated flow time series downstream of the reservoir, a routing model was required to simulate the translation, attenuation, and combination of the unregulated flow hydrographs through the system. These flow hydrographs included the upstream boundary conditions (derived reservoir inflows) and intermediate area boundary conditions (estimated local flows). The routing yielded unregulated flow time series that served as the basis of: (1) the unregulated frequency analysis and (2) the unregulated-regulated flow transform.

For this analysis, we developed an unregulated flow time series for the 2 analysis locations on the Calaveras River by:

- (Task 1) Obtaining daily unregulated reservoir inflow time series developed by the Corps.
- (Task 3.2) Developing local flow time series for the area between New Hogan Reservoir and the reservoir's control point at Bellota (shown in Figure 8).
- (Task 3.3) Completing the unregulated flow time series at each analysis point.

Obtain daily reservoir inflow

We obtained the daily unregulated reservoir inflows from the Corps. The Corps developed the daily unregulated reservoir inflow time series for New Hogan Reservoir using the continuity equation, in which, for a given time step, the average inflow equals the outflow plus the change in reservoir storage. For the calculation of these inflows, the source of the observed reservoir outflows and observed changes in storage was the Corps's database. By convention in the Central Valley, these calculations were completed on a 1-day time step, thus midnight to midnight values were used. This is consistent with the work completed for the *Sacramento and San Joaquin river basins comprehensive study* (Comp Study) completed in 2002 (USACE 2002).

Estimate local flow

For the Calaveras River, local flows needed to be estimated for the area between New Hogan Reservoir and Bellota, shown in Figure 8. The estimation approaches we used were:

- Option 1. Direct calculation of local flow using known releases from New Hogan Reservoir and the observed flows at Bellota, routing hourly flows as necessary. In the case of missing streamgage data, local flows values were interpolated as needed.
- Option 2. Estimation of local flows as:

$$Q_{Local} = 3.2\left(Q_{Cosgrove}\right) \quad (1)$$

where Q_{Local} is the local flow estimate for a given time, and $Q_{Cosgrove}$ is the observed flow at the Cosgrove Creek near Valley Springs, CA, streamgage. The Corps estimates local flows for the purpose of real-time reservoir operations using this option (John High, personal communication, 11/9/2009).

- Option 3. Estimation local flows as:

$$Q_{Local} = 0.226\left(Q_{NHG}\right) \quad (2)$$

where Q_{Local} is the local flow estimate for a given time, and Q_{NHG} is the unregulated inflow to New Hogan Reservoir. We developed this equation as detailed in Attachment 2.

In Table 5 we summarize the selected approaches for local flow estimation on the Calaveras River by water year. This flow represents the total local flow contribution at Bellota. We detail the development of the local flow time series on the Calaveras River in Attachment 2.

Table 5. Selected local flow estimation approaches for the area on the Calaveras River between New Hogan Reservoir and Bellota

Time period (water year) (1)	Time step (2)	Selected approach[1] (3)
1907-1929	Daily	Option 3: 0.226 times reservoir inflow.
1930-1969	Daily	Option 2: 3.2 times Cosgrove Creek flow.
1970-1987	Daily	Option 3: 0.226 times reservoir inflow.
1988	Daily	Option 1: directly calculate local flow.
1989	Daily	Option 3: 0.226 times reservoir inflow.
1990-1993	Daily	Option 1: directly calculate local flow.
1994-1995	Daily	Option 3: 0.226 times reservoir inflow.
1996-2009	Hourly	Option 1: directly calculate local flow.
2010	Daily	Option 2: 3.2 times Cosgrove Creek flow.

1. The approach listed is the predominant method for estimating local flows over the time period given. See Attachment 2 for further detail.

Complete unregulated flow time series

For the unregulated frequency analysis, we used the daily unregulated reservoir inflow time series provided by the Corps directly as the unregulated time series corresponding to New Hogan Reservoir. For the reservoir's operation point on the Calaveras River at Bellota, we combined the daily unregulated inflow time series with the estimated local flows by adding the 2 time series together. We did not route the unregulated reservoir inflows because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the reservoir and the operation point is approximately 7 hours, which is less than the 1-day time step of the inflows. In addition, there is little attenuation of flood peaks in this reach because of its length and channel geometry. We confirmed this by comparing observed releases from New Hogan Reservoir, observed flows on Cosgrove Creek, and observed flows on the Calaveras River at Bellota.

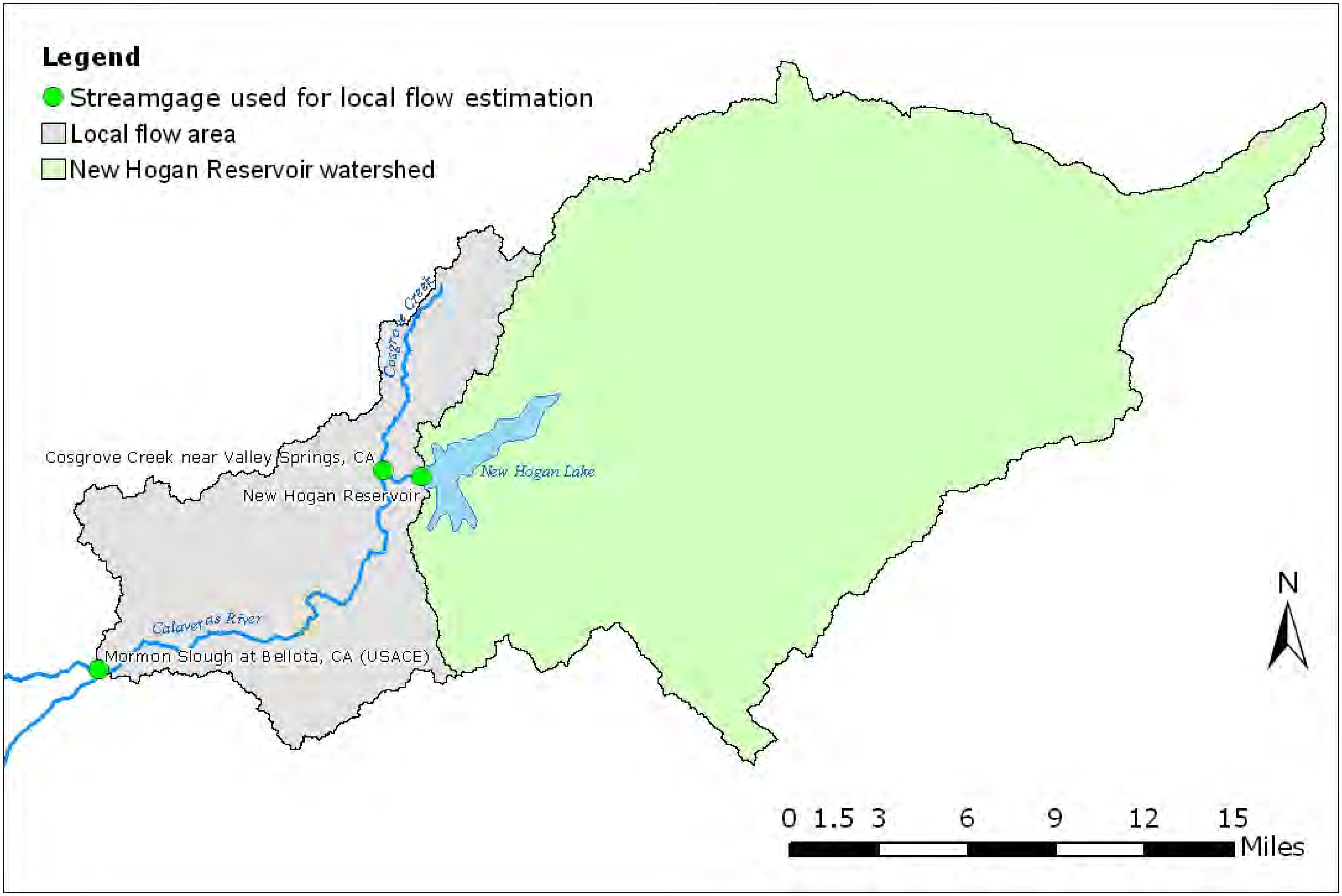


Figure 8. Calaveras River local flow area between New Hogan Reservoir and Bellota and study streamgages

Unregulated frequency analysis

Commonly accepted procedures to develop unregulated flow-frequency curves are specified in *Bulletin 17B* (IACWD 1982). The current standard-of-practice is to fit a Pearson III (LPIII) distribution to the logarithmic transforms of annual maximum series identified from streamgage data. Additional guidance for fitting frequency curves to volumes for a given duration is provided by EM 1110-2-1415 (USACE 1993).

For this analysis, we used the unregulated inflows to New Hogan Reservoir to develop such an annual maximum series. However, because we only had records of regulated flows on the Calaveras River at Bellota, we could not fit a frequency curve directly using this method. Thus, we used the synthesized unregulated flow time series at this location and fitted a volume-frequency curve to that series using procedures that are consistent with Corps guidance.

For this analysis we developed unregulated frequency curves following the procedures specified in *Bulletin 17B* (IACWD 1982), EM 1110-2-1415 (USACE 1993), and the current standards of practice. For each analysis location, we:

- Identified the annual maximum series.
- (Task 4.1) Calculated regional skew values for each duration of interest using relationships developed by the USGS.
- (Task 4.2) Fitted LPIII distributions to the annual maximum series following *Bulletin 17B* procedures and Corps guidance using the expected moment algorithm (EMA) enabled flow-frequency software PeakfqSA, version 0.937. This was developed by Tim Cohn of the USGS and is based on the USGS's flow-frequency software PeakFQ (Cohn 2007).
- Reviewed and adopted the curves, checking them for consistency and comparing them to previously accepted values.

Identify annual maximum series

We identified the annual maximum series by extracting, from the unregulated flow time series, the volumes associated with the 1-, 3-, 7-, 15-, and 30-day durations. This information is detailed in Attachment 3.

Note we developed a peak unregulated flow-frequency curve for New Hogan Reservoir for completeness; however this is not required for this analysis. The peak annual maximum series was provided by the Corps and is included in Attachment 3. In addition, we did not develop a peak flow-frequency curve for the Calaveras River at Bellota because the temporal resolution of the unregulated flow time series, 1 hour to as long as 1 day, is not an appropriate representation of instantaneous unregulated peak flow values.

Calculate regional skew values

For this analysis, we calculated regional skew values for the peak flows and 1-, 3-, 7-, 15-, and 30-day volumes using the relationships developed by the USGS (USGS 2010). In these relationships, the regional skew value is a function of the average basin elevation. The values calculated for each analysis location and duration of interest are shown in Attachment 4.

Fit frequency curves

To fit frequency curves to the annual maximum series we used: (1) the statistics of the logarithmic transforms of unregulated flow time series (mean, standard deviation, and skew), and (2) the regional skew values for the peak flow, and 1-, 3-, 7-, 15-, and 30-day calculated using relationships developed by the USGS (2010). The "at station" statistics were calculated using the EMA option in PeakfqSA.

We fitted the curves using a straightforward *Bulletin 17B* procedure in which all data points were included in the analysis. Low outliers were identified by the *Bulletin 17B* outlier test (implemented automatically by the program). The station statistics were then appropriately adjusted. This includes weighting the station skew and regional skew values by the inverse of their associated errors. This weighting procedure is included in *Bulletin 17B,* and the weighted skew is automatically calculated by the PeakfqSA software used here.

Review and adopt curves

After fitting, we reviewed the frequency curves for consistency and appropriateness. Specifically, we:

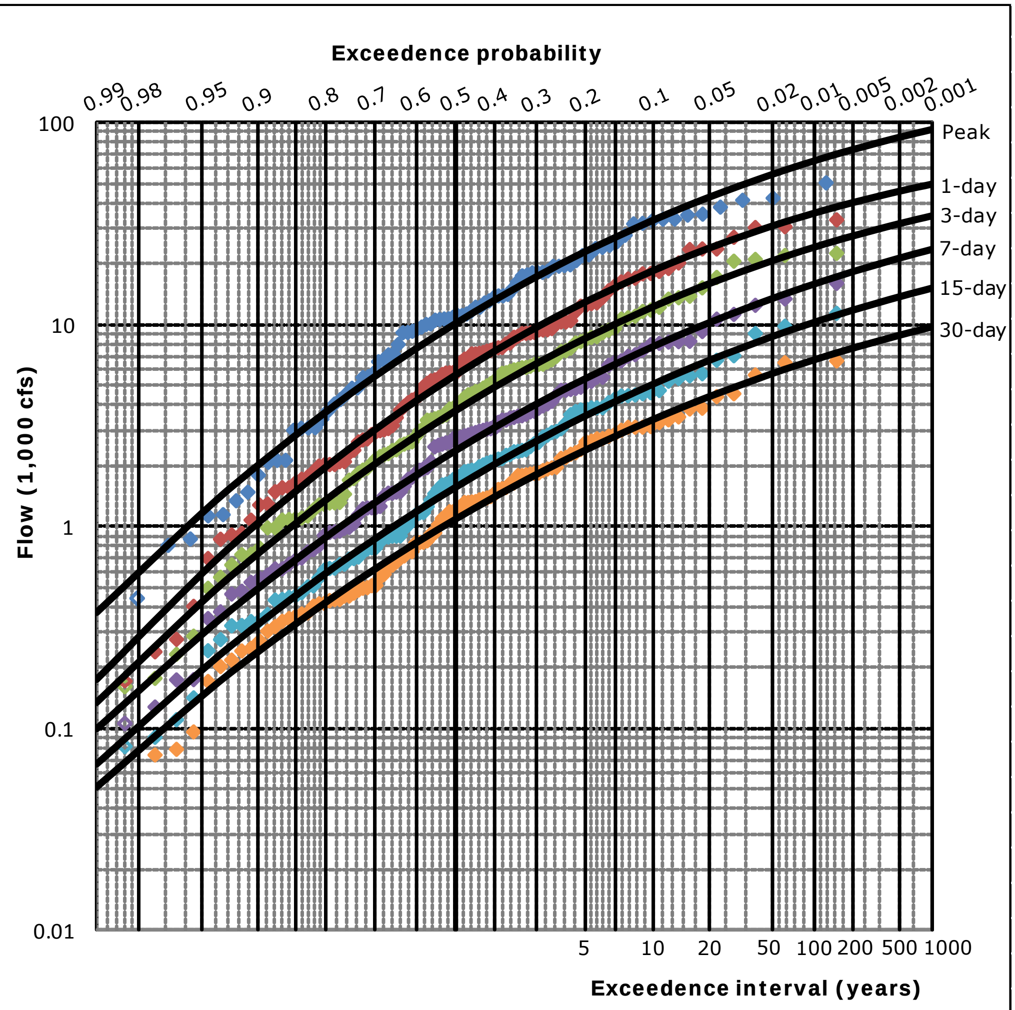
- Compared the curve of a given duration to the curves associated with the other durations at the same analysis location.
- Compared the curves at a given location to the curves at the other analysis location to check for consistency.
- Compared the curves to those published in the Comp Study.

We found the frequency curves on the Calaveras River were consistent between durations at each location. The curves do not "cross," and flow quantiles for a given duration at the downstream location are greater than those of the upstream location, as would be expected.

As a comparison, we considered the volume-frequency curves developed for New Hogan Reservoir in the Comp Study (USACE 2002). The annual maximum series in the Comp Study ended in 1997.

We also found that the flow quantiles of the curves fitted here and those of the Comp Study differ between the 2 sets of volume-duration curves by only 1%-13%. The greatest differences (of 8%-13%) are in the 1-day volume quantiles. The 3-day and 7-day volume quantiles differ by only 1% to 5%. Peak flow-frequency curves varied by as much as 9% because of the increased number of large events included in this analysis as compared to the Comp Study.

We adopted the unregulated frequency curves for the 2 analysis locations, New Hogan Reservoir and Bellota, shown in Figure 9 and Figure 10. These are the curves that use the automatic implementation of the *Bulletin 17B* outlier test. The detailed parameters used to fit these curves are included in Attachment 4.



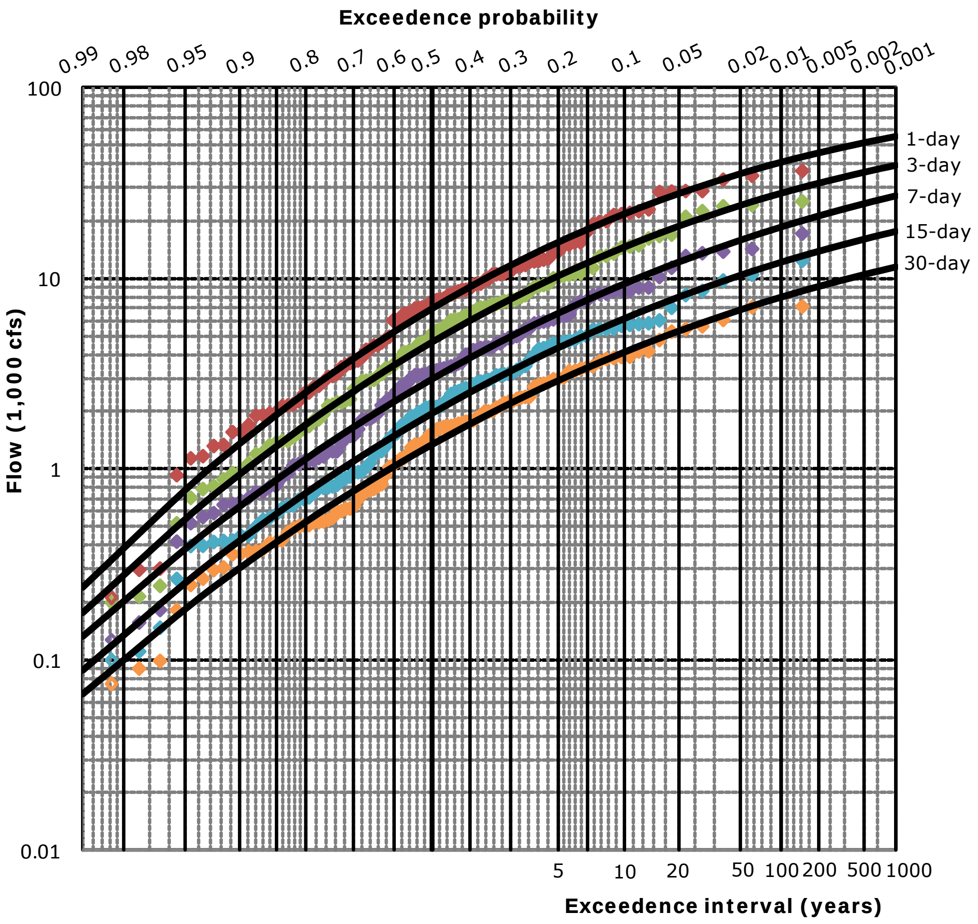
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.946	0.482	-0.727
1-day	3.685	0.501	-0.794
3-day	3.518	0.487	-0.732
7-day	3.324	0.477	-0.651
15-day	3.146	0.473	-0.656
30-day	2.988	0.457	-0.659

Notes:

- Median plotting positions.
- Drainage area: 363 sq. miles.
- Period of systematic record: 1907-2010.
  (Peak flow data intermittent 1930-2010).
- Record lengths
  Peak flows: 86 years.
  Volumes: 104 years.
- Regional skew values developed by USGS.
- Hollow points are censored events.

Figure 9. Unregulated frequency curves: New Hogan Reservoir



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.775	0.482	-0.810
3-day	3.608	0.475	-0.753
7-day	3.417	0.464	-0.666
15-day	3.240	0.461	-0.671
30-day	3.079	0.448	-0.668

Notes:

- Median plotting positions.
- Drainage area: 473 sq. miles.
- Period of systematic record: 1907-2010.
- Record length: 104 years.
- Regional skew values developed by USGS.
- Hollow points are censored events.

Figure 10. Unregulated frequency curves: Bellota

Regulated flow time series development

To develop regulated flow-frequency curves, the unregulated volume-duration-frequency curves are transformed through the unregulated-regulated flow transform. The unregulated-regulated flow transform captures the system's response to large, varied events, and is created using the unregulated and regulated flow time series. To develop the regulated flow time series we took selected historical events from the unregulated flow time series and simulated those in the regulated system. In addition, scaled historical events were used to represent events larger than those seen in the historical record for definition of the flow transforms. We then compiled the maximum unregulated and regulated flows for various durations to develop the event maxima datasets.

For this analysis we developed the regulated flow time series at each analysis location by:

- Smoothing the unregulated flow time series, using those series as boundary conditions to the reservoir simulation model.
- Identifying floods-of-record (discrete events) required to develop the flow transforms.
- Scaling historical events to represent events larger than those in the historical record.
- (Task 5.1 and Task 5.2) Simulating and routing reservoir releases of historical and scaled events.

Smooth unregulated flow time series

The daily unregulated flow time series are appropriate for frequency analysis. However daily upstream and intermediate boundary conditions do not have the temporal resolution required by the CVHS procedures for assessing the effects of regulation, particularly releases as indicated on the emergency spillway release diagram (ESRD). Therefore, the daily reservoir inflows and daily estimated local flows were "smoothed" to hourly time series. This smoothing was completed using a mass balance algorithm that interpolates the shape of the hydrograph and estimates peak hourly flows while maintaining daily volumes consistent with the original time series. These smoothed times series were provided by the Sacramento District Hydrology Section for use in this analysis.

Identify floods-of-record

Events rarer than p=0.5 annual exceedence are needed to define the flow transforms. To develop the flow transforms we used both historical events and scaled historical events. The 60 historical events used were those with 1-day volumes greater than 5,000 cfs (a threshold slightly lower than volume corresponding to the p=0.5 exceedence event.)

To select the subset of events used for scaling, we identified: (1) the 14 large flood events for the San Joaquin River basin (listed in the Comp Study historical storm matrices), and (2) the 5 largest events for the Calaveras River watershed. We list these events in Table 6. In Table 6, column 1 lists the water year of the event, column 2 and column 3 list the associated start and end dates, column 4 lists the 1-day volume, and column 5 indicates the selection basis. We identified these dates by visual inspection of unregulated inflow time series provided by the Corps. The time windows defined by these dates was used for extraction of the event maxima (unregulated and regulated) for development of the flow transforms.

The Comp Study lists both a January and February event for the 1969 water year in the San Joaquin River basin. However, a large February inflow event is not present in the New Hogan Reservoir unregulated inflow time series. Therefore, for this analysis we treat the 1969 flood as a single event.

Table 6. Calaveras River floods-of-record scaled to develop flow transforms

Water year[1] (1)	Start date (2)	End date (3)	1-day max volume (cfs) (4)	Selection basis (5)
1958	3/10/1958	4/30/1958	32,920	Largest inflow event
1938	1/25/1938	2/28/1938	30,450	Largest inflow event
1911	1/10/1911	2/28/1911	30,175	Largest inflow event
1936	2/10/1936	3/24/1936	26,987	Largest inflow event
1907	3/1/1907	4/14/1907	23,641	Largest inflow event
1986	2/10/1986	3/6/1986	23,494	Comp Study storm matrix event
1956	12/15/1955	2/15/1956	20,156	Comp Study storm matrix event
1998	1/1/1998	3/15/1998	16,919	Comp Study storm matrix event
1997	12/1/1996	2/15/1997	16,801	Comp Study storm matrix event
1969[2]	1/5/1969	3/20/1969	14,674	Comp Study storm matrix event
1940	2/11/1940	3/16/1940	13,610	Comp Study storm matrix event
1965	12/18/1964	1/18/1965	12,789	Comp Study storm matrix event
1982	12/28/1981	1/31/1982	12,321	Comp Study storm matrix event
1983	2/25/1983	4/10/1983	10,433	Comp Study storm matrix event
1995	3/1/1995	4/6/1995	10,146	Comp Study storm matrix event
1951	11/12/1950	11/31/1950	9,390	Comp Study storm matrix event
1980	1/1/1980	1/31/1980	8,648	Comp Study storm matrix event
1967	1/20/1967	2/10/1967	6,738	Comp Study storm matrix event
1978	3/1/1978	3/19/1978	5,770	Comp Study storm matrix event

1. Events are in order of increasing 1-day flow volume
2. For the purposes of this analysis we treat the 1969 flood as 1 single event.

Scale historical floods

In addition to the 60 historical floods-of-record, events larger than these recorded were required to develop the flow transforms throughout the full range of interest. To obtain those, we scaled the time series for the subset of historical events listed in Table 6 uniformly by factors at 0.2 intervals from 1.2 through 3.0 for use in simulating reservoir releases. This yielded a total of 10 scaled time series for each event. Both the unregulated reservoir inflow and estimated local flow time series were scaled uniformly to maintain the coincidence and timing of the system.

Scaled historical events were used only for the development of the flow transforms. The events were not used for fitting the unregulated flow frequency curves. This use of scaled historical events is consistent with the guidance in EM 1110-2-1415.

Simulate and route historical and scaled floods

We simulated reservoir operation and routed flows for both the historical floods-of-record and scaled historical events using the computer program HEC-ResSim, version 3.1 Beta III, developed by the USACE Hydrologic Engineering Center (HEC). Given a reservoir network, operating rules and constraints, and a set of inflows and downstream local flows, HEC-ResSim routes the flows through the system and simulates releases for the reservoirs. These releases are based on the rules and constraints defined in the water control manual.

An HEC-ResSim reservoir network includes representation of the physical properties of the reservoirs and links from reservoirs to downstream points of interest. Hydrologic routing model parameters are required to represent the movement of the flood wave between nodes in the network. Required physical properties include elevation-volume relationships, elevation-maximum outflow relationships, and physical limitations of the reservoir outlets.

The operating rules defined for a reservoir for HEC-ResSim include release functions based on reservoir pool elevation, reservoir inflow, and downstream flow constraints. Rate of change constraints are also included in the operation rule sets. For the Calaveras River, New Hogan Reservoir operates to meet downstream flow constraints at Bellota, which is the bifurcation of the Calaveras River and Mormon Slough approximately 18 miles downstream of the reservoir.

Simulate reservoir operation

For this analysis, we used the representation of the Calaveras River system in HEC-ResSim developed by the Corps; that will be used for the CVHS. This includes a representation of the network and the reservoir operation rules. The HEC-ResSim schematic of the Calaveras River system is shown in Figure 11.

For reference, New Hogan Reservoir is operated to maintain flows in the Calaveras River at Bellota below 12,500 cfs. The complete set of operating rules is defined in the New Hogan Reservoir water control manual (USACE 1983).

With this model, we simulated the 19 historical floods-of-record and associated scaled events for a total of 209 simulations. Consistent with the standard-of-practice for such analysis, for the reservoir routings, we used only the dedicated flood control storage space for the attenuation of the reservoir inflows. Thus, at the start of the simulation, the reservoir water surface elevation equals the elevation of the bottom of the flood control pool. The simulation time step for this analysis is 1 hour.

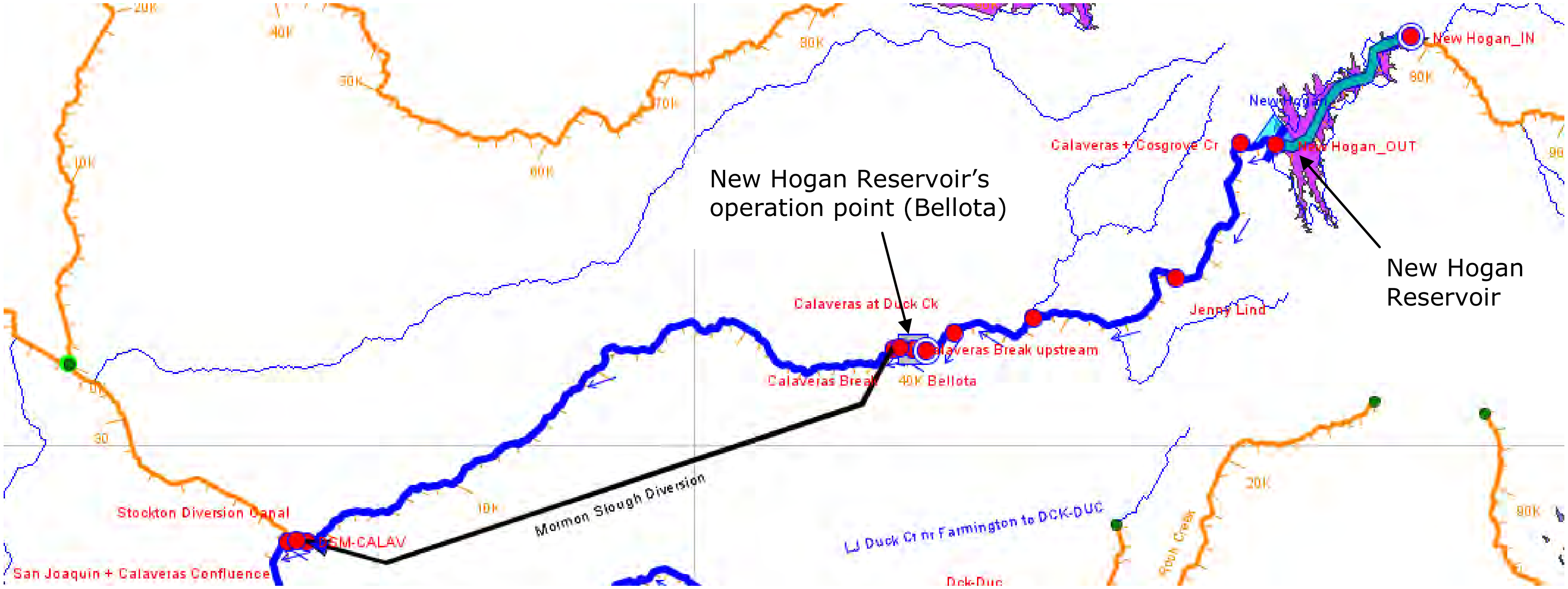
After completing the reservoir simulations, we reviewed the results from the HEC-ResSim computer program. Based on our knowledge of the system operation and water control manual, we reviewed and adjusted the HEC-ResSim computed flows. In several cases, we modified the reservoir releases using both release overrides and HEC-DSS routing computations to fully utilize the downstream channel capacity and available flood storage in the reservoir.

Route reservoir releases

We used Muskingum routing to route flows on the Calaveras River. A detailed channel model of the Calaveras River does not currently exist. Although the *Procedures document* calls for the hydraulic routing of reservoir releases, we found that the Calaveras River can be adequately simulated with hydrologic routing because: (1) the analysis locations on the Calaveras River are not affected by backwater and therefore do not require evaluation of stages to develop regulated flow-frequency curves, and (2) the reservoir release hydrographs do not rise quickly.

We reviewed the reservoir simulations and routings computed the program HEC-ResSim and adjusted as needed to obtain accurate peak regulated flows for the simulation of each event.

The results from the reservoir simulation and routing are provided on DVD with the original report.



New Hogan Reservoir's operation point (Bellota)

New Hogan Reservoir

Figure 11. Screenshot of HEC-ResSim system schematic: Calaveras system

Flow transform fitting and application

Once the regulated flow time series were developed, the next step was to pair, by event, the unregulated and regulated flow time series. Using these pairings, the event properties, such as the volumes for given durations, and in the case of the regulated time series, peak flows, were identified. The result of this pairing and identification was the event maxima dataset. Specifically, the event maxima dataset consists of unregulated and regulated flows of various durations for a given historical or scaled historical event.

Once the event maxima datasets were compiled, a transform curve was fitted to develop the unregulated-regulated flow transforms. This curve translated the unregulated flow of a given quantile to the corresponding regulated flow for that same quantile. This process is illustrated in Figure 12.

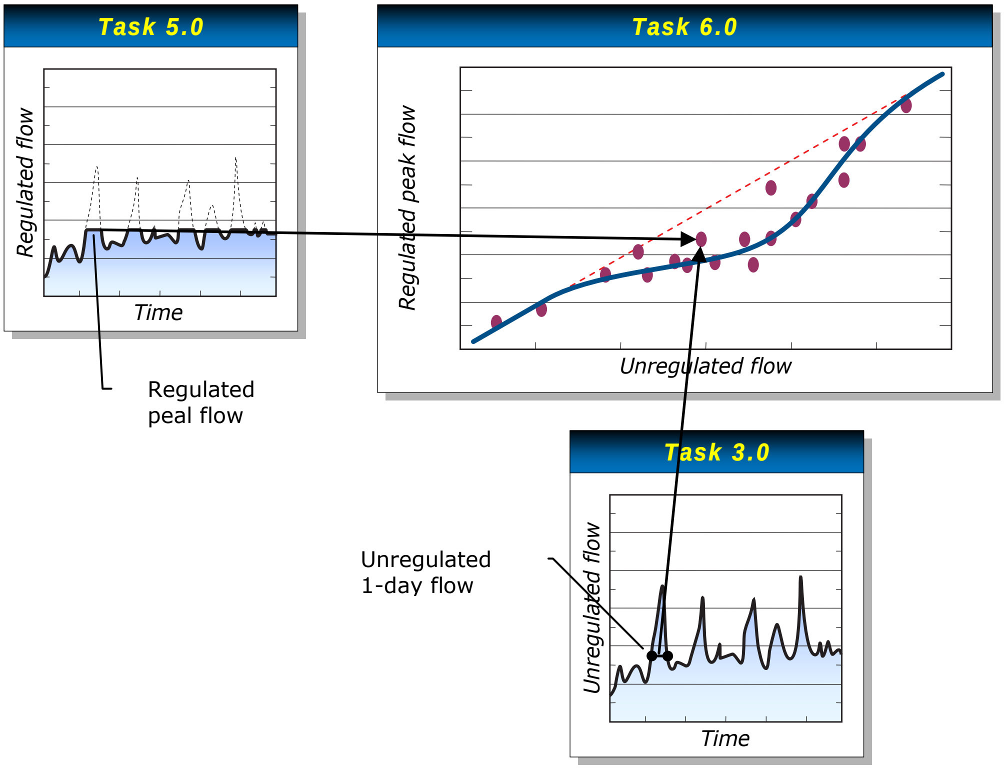


Figure 12. Flow transform development process

For the unregulated-regulated flow transform, the regulated flow value used was the peak flow. The unregulated flow value was the unregulated flow corresponding to the critical duration for that analysis location. The critical duration was found through an analysis of unregulated and regulated flows for historical and scaled historical events.

Additional transform curves were fitted to develop the family of characteristic curves. These curves identified the associated regulated volume duration characteristics of a given peak regulated flow.

For this analysis, we developed the flow transforms by:

- (Task 6.1) Identifying unregulated and regulated event maxima for the floods-of-record.
- (Task 6.2) Fitting the unregulated-regulated flow transform for each duration of interest.
- Determining the critical duration to identify the appropriate unregulated-regulated transform to use at each analysis location.
- Fitting the family of characteristic curves.
- Reviewing and accepting the flow transforms.

We then applied the flow transforms to the unregulated frequency curves to develop the regulated flow-frequency curves (Task 6.4).

Identify event maxima datasets

We identified the event maxima datasets using inspection and HEC-DSS utilities. For each analysis location, we:

- Identified the properties of the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations for unregulated flows associated with the floods-of-record. The durations we used are consistent with those specified in the *Technical procedures document* for analyzing critical duration.
- Identified the peak regulated flows from the regulated flow time series of the historical floods-of-record and scaled historical events. Note that here, peak regulated flow corresponds to the maximum hourly value regulated flow time series, and not a true instantaneous peak.
- Identified the properties of the 1-, 3-, 7-, 15-, and 30-day durations for regulated flows associated with the historical floods-of-record and scaled historical events. We did not include all the durations used in the critical duration analysis consistent with those specified in the *Technical procedures document* and the current standard-of-practice for flow-frequency analysis.

The event maxima datasets are tabulated in an MS Excel file on a DVD provided with the original report. The tabulated information lists each historical and scaled historical event used in this analysis and the associated volumes for the (1) unregulated flow volumes corresponding to the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations, and (2) regulated flow volumes corresponding to the peak, 1-, 3-, 7-, 15-, and 30-day durations.

Fit unregulated-regulated flow transforms

We developed the unregulated-regulated flow transforms for the 2 analysis locations by fitting transform curves through the pairs of event unregulated volumes and regulated peak flows. The unregulated volumes used were the average flows associated with the durations previously noted. We fitted these curves to the data pairs of historical and scaled events using the robust locally weighted scatterplot smoothing (LOWESS) regression technique. (The LOWESS procedure is detailed in the *Technical procedure document*.)

Here, we fitted these transforms for the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations. The event maxima datasets include both historical and scaled events to define the extreme end of the flow transform curves. Fitting of the transforms are detailed in Attachment 5.

The CVHS analysis procedure requires 1 single unregulated-regulated transform for statements of probability. To identify which duration is most appropriate, the critical duration for the given analysis location must be determined as described in the next subsection.

Determine critical duration

We determined critical duration at each analysis location by: (1) applying the unregulated-regulated flow transforms to the unregulated flow–frequency curves to develop hypothetical regulated flow-frequency curves, and (2) identifying the duration of the unregulated annual maximum series that consistently estimates the largest flow for each probability. In selecting the critical duration, we considered both the "goodness of fit" of each transform and which duration estimates the greater peak regulated flows. This procedure is described in more detail in Attachment 5.

From this analysis we determined that the critical duration at New Hogan Reservoir is 3.5 days and at Bellota is 1 day. Thus, the appropriate unregulated-regulated flow transforms used in this analysis were associated with these durations. The critical duration associated with the downstream operation point is shorter than that of the reservoir because of the effects of local flow.

After determining the critical duration associated with each analysis location, we reviewed and adjusted the unregulated-regulated flow transforms initially fitted with the LOWESS procedure as detailed in Attachment 5. We then adopted the flow transforms for New Hogan Reservoir and Bellota shown in Figure 13 and Figure 15. In Figure 13 and Figure 15, some scaled historical event maxima for more common events, i.e., annual exceedence probabilities greater than p=0.01, have regulated peaks exceeding the channel capacity (12,500 cfs) because of large local flows.

Fit family of regulated characteristic curves

We developed the families of regulated characteristic curves for New Hogan Reservoir and at Bellota by fitting most likely curves through the pairs of event regulated volumes as average flows and regulated peak flows, similar to the procedure we used to fit the unregulated-regulated transforms. The data pairs (from the event maxima datasets) we used include both historical and scaled events to define the extreme ends of the flow transform curve.

The family of regulated characteristic curves for New Hogan Reservoir and Bellota are shown in Figure 14 and *Figure* 16, and are detailed in Attachment 6. These curves associate regulated peak flows to regulated characteristic volumes. We fitted characteristic curves for the 1-, 3-, 7-, 15-, and 30-day durations. We compare these families of curves in Figure 17.

On the Calaveras River, the typical duration of releases from New Hogan Reservoir for events in the given range of interest is less than 15 days. Therefore we include the 15-day and 30-day characteristic curves here for completeness, and in keeping with the CVHS procedures.

For New Hogan Reservoir, the 1-day and 3-day regulated volume characteristic curves are almost the same for regulated peaks of approximately 14,000 cfs-22,00 cfs, as shown in Figure 14. This is expected for ranges of regulated peaks because large inflow volumes associated with the events will result in similar releases for the shorter durations while the reservoir is able to maintain control. Similarly, the characteristic curves at Bellota are the same for ranges of regulated peaks, as shown in *Figure* 16.

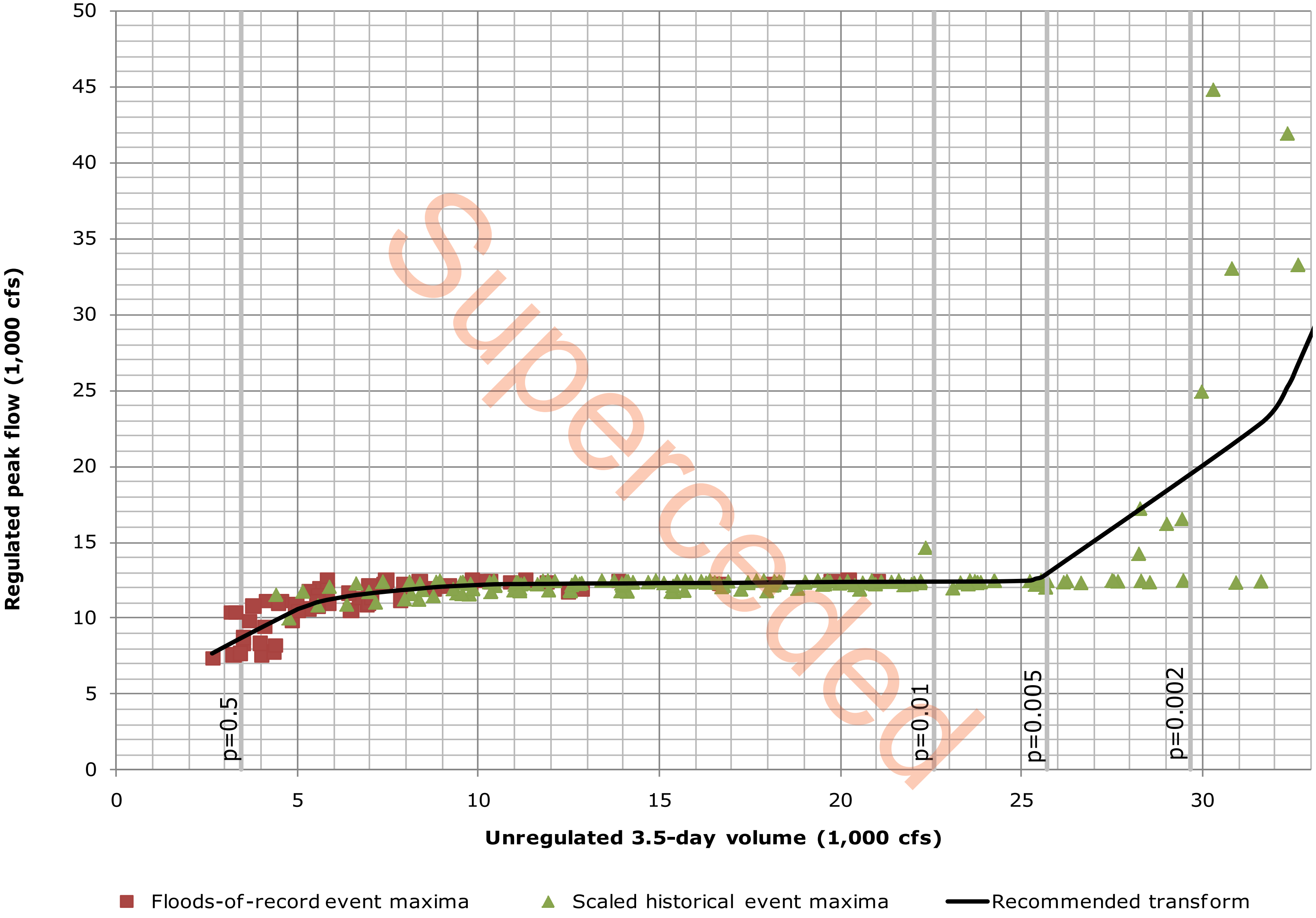


Figure 13. Unregulated-regulated flow transform: New Hogan Reservoir

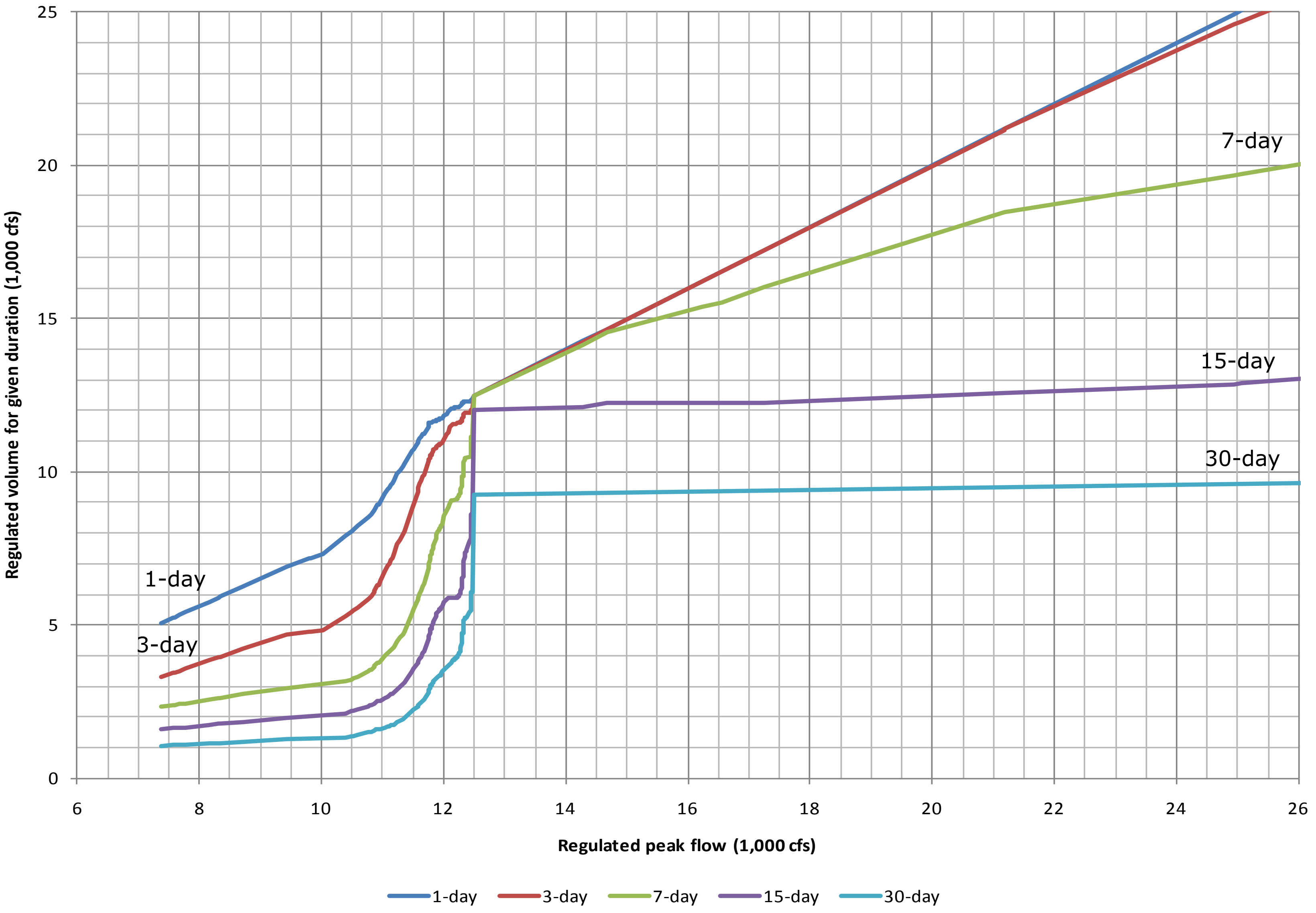


Figure 14. Family of regulated characteristic curves: New Hogan Reservoir

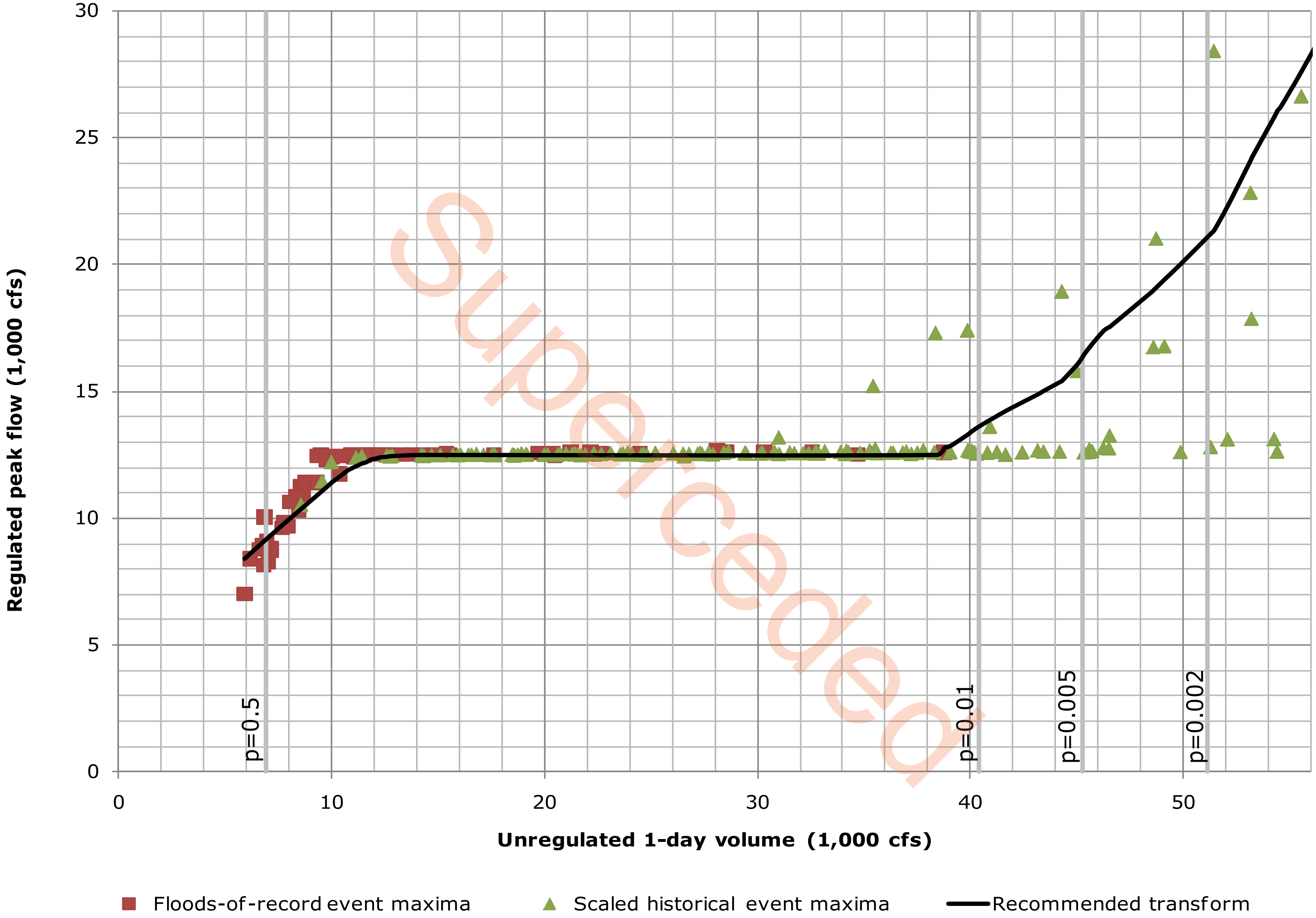


Figure 15. Unregulated-regulated flow transform: Calaveras River at Bellota

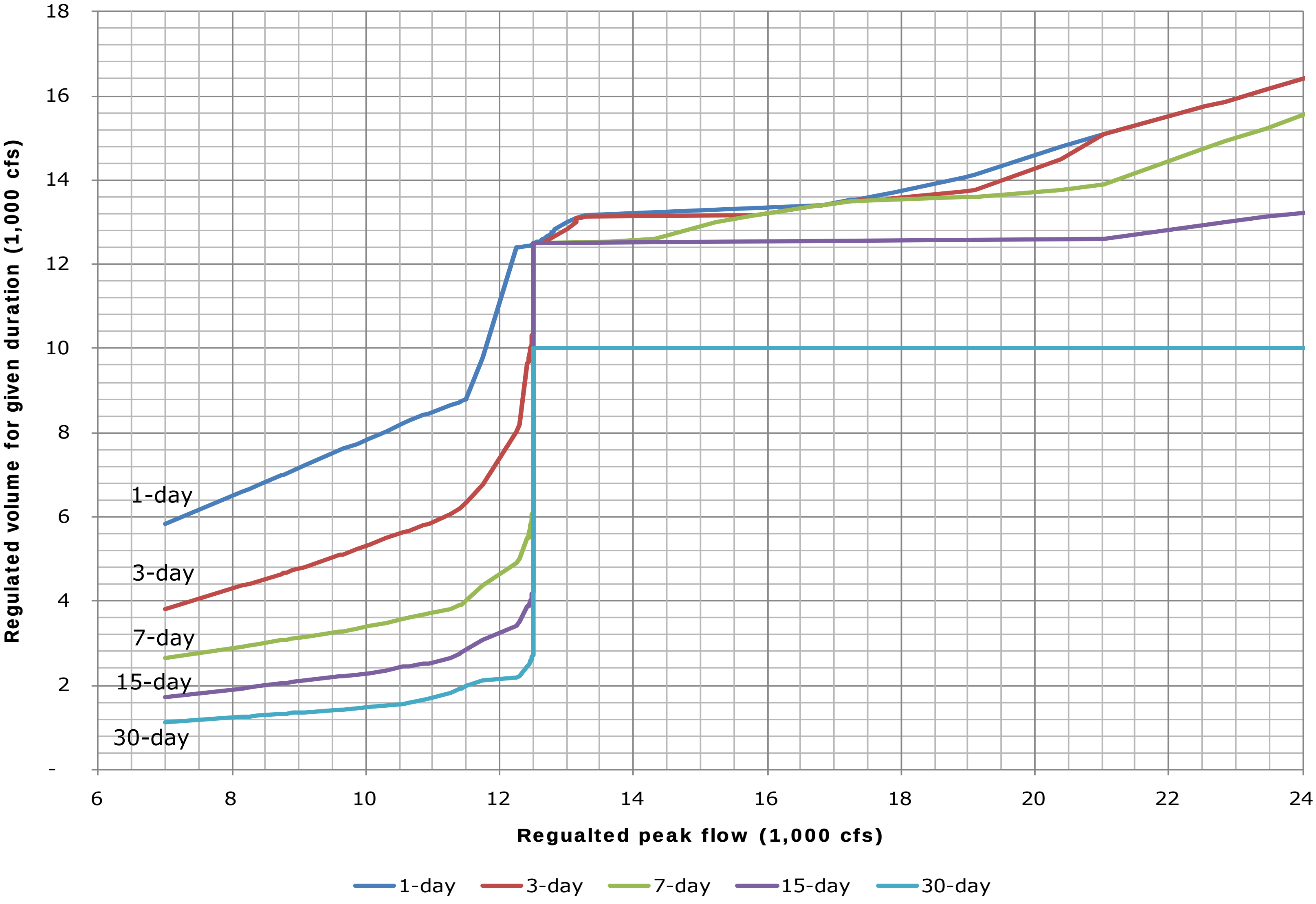


Figure 16. Family of regulated characteristic curves: Calaveras River at Bellota

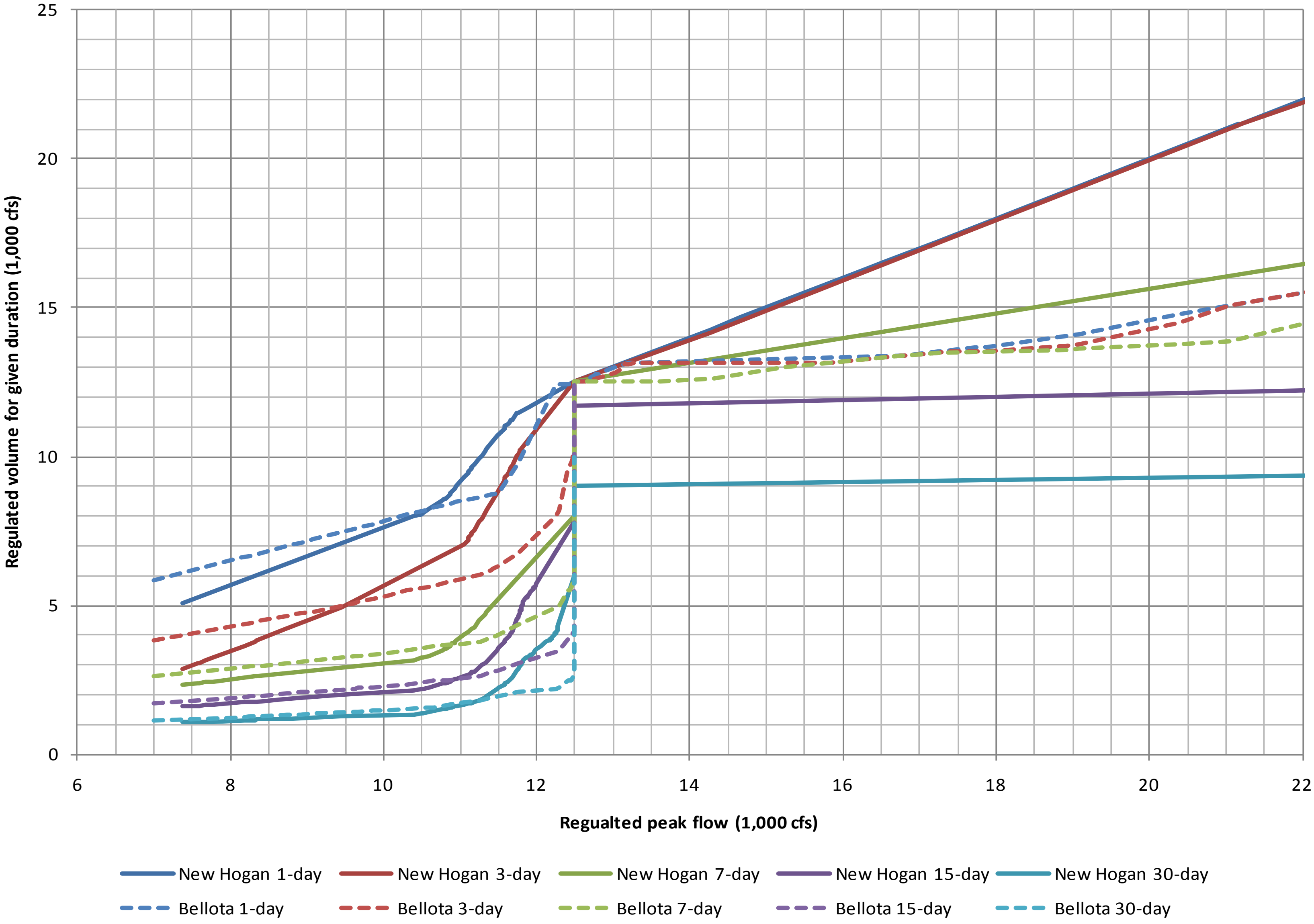


Figure 17. Comparison of the families of characteristic curves for New Hogan Reservoir and Bellota

Review and adopt flow transforms

After fitting the flow transforms and characteristic curves, we reviewed the resulting functions for consistency. Specifically, we compared each transform to (1) the transforms associated with different durations at the same analysis location, and (2) the transforms at the other analysis location. We found:

- The unregulated-regulated flow transforms were consistent between analysis location, i.e., the regulated peak flow for a given quantile at the downstream location was greater than that of the upstream location.
- At New Hogan Reservoir, the family of regulated characteristic curves were consistent between durations, i.e., they do not cross. This is expected.
- At Bellota, the initially fitted 3-day and 7-day curves crossed the 1-day curve. Therefore we set the 3-day characteristic curve equal the 1-day curve over their ranges of intersection, and the 7-day curve equal the 3-day curve over their initial range of intersection.
- The flow transforms at Bellota were sensitive to large peaks in local flow such as those computed directly for the 1997, 1998, and 2006 events. For scaled versions of these events, the local flow exceeded channel capacity before the New Hogan Reservoir flood control pool was filled.

Based on this review, we adopted these flow transforms for the 2 analysis locations.

Apply flow transforms

We developed a regulated peak flow-frequency curve and the associated regulated 1-, 3-, 7-, 15-, and 30-day volumes at New Hogan Reservoir and at Bellota by combining the appropriate information from the unregulated frequency curves, the flow transforms, and the families of regulated characteristic curves. The regulated flow-frequency curves for New Hogan Reservoir and Bellota are shown in Table 7 and Table 9 and their associated volumes are tabulated in Table 8 and Table 10.

To apply the flow transforms and develop regulated flow-frequency curve associated volumes at each analysis location we:

- Identified the unregulated flow quantiles associated with the critical duration that correspond to the probabilities of interest.
- Identified the regulated peak flows that correspond to the flow quantiles identified in the previous step using the flow transform.
- Identified the regulated flow characteristics that correspond to the regulated peaks identified in the previous step using the family of regulated characteristic curves.

Table 7. Regulated peak flow-frequency quantiles: New Hogan Reservoir

Annual exceedence probability (1)	**1/annual exceedence probability (2)**	**Regulated peak flow (cfs) (3)**
0.500	2	8,664
0.200	5	11,812
0.100	10	12,214
0.050	20	12,266
0.020	50	12,334
0.010	100	12,367
0.005	200	12,903
0.002	500	19,555

Table 8. Regulated peak flow values and associated volumes: New Hogan Reservoir

Annual exceedence probability of regulated peak flow (1)	**Regulated peak flow (cfs) (2)**	**Associated volumes[1] (as average flow for given duration)**				
		1-day (cfs) (3)	**3-day (cfs) (4)**	**7-day (cfs) (5)**	**15-day (cfs) (6)**	**30-day (cfs) (7)**
0.500	8,664	6,212	4,188	2,720	1,843	1,199
0.200	11,812	11,625	10,634	7,457	4,994	3,096
0.100	12,214	12,107	11,582	9,098	5,909	3,963
0.050	12,266	12,140	11,607	9,312	6,032	4,157
0.020	12,334	12,283	11,880	10,275	7,045	5,120
0.010	12,367	12,300	11,916	10,459	7,411	5,263
0.005	12,903	12,900	12,893	12,876	12,026	9,283
0.002	19,555	19,555	19,549	17,462	12,445	9,463

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at New Hogan Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Table 9. Regulated peak flow-frequency quantiles: Calaveras River at Bellota

Annual exceedence probability (1)	1/annual exceedence probability (2)	Regulated peak flow (cfs) (3)
0.500	2	9,163
0.200	5	12,500
0.100	10	12,500
0.050	20	12,500
0.020	50	12,500
0.010	100	13,634
0.005	200	16,409
0.002	500	21,107

Table 10. Regulated peak flow values and associated volumes: Calaveras River at Bellota

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration)				
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)	30-day (cfs) (7)
0.500	9,163	7,271	4,852	3,163	2,127	1,372
0.200	12,500	12,500	12,500	12,500	12,500	10,000
0.100	12,500	12,500	12,500	12,500	12,500	10,000
0.050	12,500	12,500	12,500	12,500	12,500	10,000
0.020	12,500	12,500	12,500	12,500	12,500	10,000
0.010	13,634	13,174	13,141	12,545	12,515	10,001
0.005	16,409	13,367	13,300	13,300	12,553	10,002
0.002	21,107	15,106	15,106	13,930	12,631	10,005

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at New Hogan Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Expected hydrograph properties

The expected (design) hydrograph for a given exceedence probability is a New Hogan Reservoir outflow hydrograph with a peak flow that matched the regulated flow-frequency curve (as shown in Table 7) and with associated volumes matching those from the family of characteristic curves corresponding to the given regulated peak flow (as shown in Table 8). The properties of the expected hydrographs for the p=0.5, p=0.2, p=0.1, p=0.05, p=0.02, p=0.01, p=0.005, and the p=0.002 exceedence probabilities are shown in Table 11.

An expected hydrograph can be formed by applying these properties to a specific hydrograph shape. As part of future work, we will identify specific historical event patterns to which the expected hydrograph properties can be applied. For this identification, we will follow the example event selection procedure provided in the *CVHS Product uses* document (USACE 2009c) and .

Options for expected hydrograph development and application using study products were submitted by Ford Engineers to the Corps on June 23, 2010. From that memorandum, the Corps selection Option 1: Selected event-based reservoir release hydrographs.

Table 11. Expected hydrograph properties: New Hogan Reservoir outflow

Annual exceedence probability of regulated peak flow (1)	**1/annual exceedence probability of peak flow (2)**	**Regulated peak flow (cfs) (3)**	**Associated volumes[1] (as average flow for given duration)**				
			1-day (cfs) (4)	**3-day (cfs) (5)**	**7-day (cfs) (6)**	**15-day (cfs) (7)**	**30-day (cfs) (8)**
0.500	2	8,664	6,212	4,188	2,720	1,843	1,199
0.200	5	11,812	11,625	10,634	7,457	4,994	3,096
0.100	10	12,214	12,107	11,582	9,098	5,909	3,963
0.050	20	12,266	12,140	11,607	9,312	6,032	4,157
0.020	50	12,334	12,283	11,880	10,275	7,045	5,120
0.010	100	12,367	12,300	11,916	10,459	7,411	5,263
0.005	200	12,903	12,900	12,893	12,876	12,026	9,283
0.002	500	19,555	19,555	19,549	17,462	12,445	9,463

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at New Hogan Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Results

The results of this frequency analysis include:

- Unregulated frequency curves for New Hogan Reservoir (as shown in Figure 9).
- Unregulated frequency curves for the Calaveras River at Bellota (as shown in Figure 10).
- Unregulated-regulated flow transform for New Hogan Reservoir (as shown in Figure 13).
- Regulated flow-frequency curve and associated volumes for New Hogan Reservoir (as shown in Table 7 and in Table 8).
- Unregulated-regulated flow transform for the Calaveras River at Bellota (as shown in Figure 15).
- Regulated flow-frequency curve and associated volumes for the Calaveras River at Bellota (as shown in Table 9 and in Table 10).
- Expected hydrograph properties for New Hogan Reservoir (as shown in Table 11).

In addition, these intermediate data are included with the original report on DVD:

- HEC-DSS time series of the floods-of-records.
- HEC-DSS time series of the scaled historical floods.
- HEC-DSS time series of developed local flows below New Hogan Reservoir (detailed in Attachment 2).
- The tabulated event maxima datasets for the 2 analysis sites.
- Simulated reservoir releases and routed flows from the HEC-ResSim reservoir simulation model.
- Tabulated unregulated-regulated flow transforms for the 2 analysis sites.
- Tabulated families of regulated characteristic curves for the 2 analysis sites.

References

Beard, Leo R. (1962). *Statistical methods in hydrology.* Hydrologic Engineering Center, US Army Corps of Engineers, Davis, CA.

Bradley, Allen A. Jr., and Potter, Kenneth W. (2004). *PVSTATS, user manual version 3.1.* University of Wisconsin-Madison, Department of Civil and Environmental Engineering, Madison, WI.

Cleveland, William S. (1979). "Robust locally weighted regression and smoothing scatter plots." *Journal of the American Statistical Association*, 74(368) 829-836.

Cleveland, William S. (1985). Lowess.f [Fortran file]. Bell Laboratories. Murray Hill, NJ.

Cohn, Tim. (2007). PeakfqSA, version 0.937 [Software]. <http://www.timcohn.com/TAC_Software/PeakfqSA/>.

Goldman, David M. (2001). *"Quantifying uncertainty in estimates of regulated flood frequency curves." State of the practice – proceedings of the World Water and Environmental Resources Congress,* ASCE, Reston, VA.

Helsel, D. R., and Hirsch, R. M. (2002). *Statistical methods in water resources,* US Geological Survey, Reston, VA.

Interagency Advisory Committee on Water Data (IACWD). (1982). *Guidelines for determining flood flow frequency, Bulletin 17B.* US Geological Survey, Reston, VA.

US Army Corps of Engineers (USACE). (1983). *New Hogan Dam and Lake, Calaveras River, California, Water control manual, Appendix III to Master water control manual, San Joaquin River Basin, California,* Sacramento District, Sacramento, CA.

USACE. (1990). *Calaveras River, California: Reconnaissance report*, Sacramento District, Sacramento, CA.

USACE. (1993). *Hydrologic frequency analysis, EM 1110-2-1415*, Washington, D.C.

USACE. (1994). *Engineering and design-hydrologic engineering studies design, EP 1110-2-9*, Washington, D.C.

USACE. (1997). *Hydrologic engineering requirements for reservoirs, EM 1110-2-1420*, Washington, D.C.

USACE. (2002). *Sacramento and San Joaquin river basins comprehensive study, December 2002 interim report* ("Comp study"), USACE, Sacramento District, Sacramento, CA.

USACE. (2004). *Farmington Dam and Reservoir, Littlejohn Creek, California, Water control manual, Appendix IV to Master water control manual, San Joaquin River Basin, California,* Sacramento District, Sacramento, CA.

USACE. (2009a). *Central Valley hydrology study (CVHS): Technical procedures document* ("Technical procedures document"), prepared by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (2009b). *CVHS product uses* ("Uses document"), prepared by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (2009c). *Sacramento and San Joaquin river basins: Procedures for hydrologic analysis ("Procedures document"),* prepared by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (2010). *Hydrologic engineering management plan for the Lower San Joaquin River feasibility study*, *prepared by David Ford Consulting Engineers, Inc., Sacramento, CA.*

USACE. (2011a). *Central Valley hydrology study (CVHS): Technical procedures document,* prepared by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (2011b). *Central Valley hydrology study (CVHS) technical procedures document Attachment B: Unregulated time series development,* prepared by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (2011c). *Central Valley hydrology study (CVHS) Technical procedures document Attachment C: Regulated time series development,* prepared by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (2011d). *Central Valley hydrology study (CVHS) Technical procedures document Attachment D: Flow frequency analysis,* prepared by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (2011e). *Central Valley hydrology study (CVHS) Technical procedures document Attachment E, Development of flow and stage transforms,* prepared by David Ford Consulting Engineers, Inc., Sacramento, CA.

Attachment 1: Correspondence of procedural steps

Table 12 shows how the procedural steps in this document correspond to the steps in the *Procedures document* and the *Technical procedures document*.

Table 12. Correspondence of procedural steps for the LSJR FS, the CVHS "Procedures document," and the CVHS "Technical procedures document"

This step in the hydrologic analysis at New Hogan Reservoir… (1)	**Corresponds to this action in the *Procedures document*… (2)**	**And/or this action in the *Technical procedures document*… (3)**
Develop unregulated flow time series	Task 3.0	Attachment B: Unregulated flow time series development
• Estimate local flows	Task 3.2	• Application and distribution of local flows
• Route and complete unregulated flow time series at analysis locations	Task 3.3	• Procedures for routing flows through the system
Develop unregulated frequency curves	Task 4.0	Attachment D: Frequency analysis
Develop regulated flow time series	Task 5.0	Attachment C: Regulated time series development
• Identify floods-of-record • Scaling of historical reservoir inflows	Task 6.2	Attachment E: Development of flow and stage transforms • Determination of historical event scaling for extrapolating unregulated-regulated flow transform
• Simulation of reservoir releases for historical and scaled events	Task 5.1, Task 6.2	• Procedures for routing regulated flows through the system
Develop flow transforms	Task 6.0	Attachment E: Development of flow and stage transforms
• Identify annual maximum series	Task 6.1	—
• Assess reservoir critical duration	—	Attachment E: Development of flow and stage transforms • Identification of critical duration at analysis points Attachment F: Procedure for critical duration calculation

This step in the hydrologic analysis at New Hogan Reservoir… (1)	**Corresponds to this action in the *Procedures document*… (2)**	**And/or this action in the *Technical procedures document*… (3)**
• Fit unregulated-regulated flow transform • Fit family of regulated characteristic curves	Task 6.3	Attachment E: Development of flow and stage transforms • Procedure for fitting a "most likely" transform through the datasets
• Apply flow transforms to develop regulated-flow-frequency curves	Task 6.4	—
Develop expected hydrographs[1]	—	—

Notes:

1. Options for expected hydrograph development using study products were submitted by Ford Engineers to the Corps on June 23, 2010. From that memorandum, the Corps selection Option 1: Selected event-based reservoir release hydrographs.

Attachment 2: Calaveras River local flow development

Overview

For the Calaveras River, we estimated local flows for the area between New Hogan Reservoir and Bellota, shown in Figure 8. For this area, we used 3 options to estimate local flow:

- Option 1. Direct calculation of local flow.
- Option 2: Estimation of local flow as a function of observed flow on Cosgrove Creek. Note: the Corps currently estimates local flow as 3.2 times observed (gaged) flow at Cosgrove Creek near Valley Springs, CA.
- Option 3. Estimation of local flow as a function of New Hogan Reservoir inflow.

Option 1 is the most accurate option for local flow estimation. To determine which of the other 2 options for local flow estimation is more appropriate to use, we:

- Reviewed the streamgage and reservoir inflow data provided by the Corps. In Table 13 we list the streamgages that were used in estimating local flows on the Calaveras River. Column 1 lists the streamgage ID whose corresponding name is listed in column 2, column 3 lists the data type (e.g., daily or hourly), column 4 lists the applicable time period of the streamgage data, and column 5 lists notes on the data.
- Coordinated with Corps staff regarding streamgage data quality.
- Identified the data type (e.g., daily or hourly) of the provided data.
- Identified the overlapping time periods for each streamgage by time step.
- Estimated local flow by direct calculation (Option 1).
- Compared the directly calculated local flow time series to observed flows on Cosgrove Creek and New Hogan Reservoir inflows.
- Identified, for Option 2 and Option 3, alternative functions for estimating local flow including:
  - Direct multipliers based on ratios of peak flows for selected large events.
  - Direct multipliers based on drainage area ratios.
  - Linear functions determined by regression analysis.
  - Exponential functions determined by regression analysis.
  - Linear functions of logarithmic transforms of flow determined by regression.
- Estimated local flow time series using the possible functions identified.
- Estimated a local flow time series using the observed flow on Cosgrove Creek and the 3.2 multiplier used by the Corps.

- Compared the estimated local flow time series to the directly calculated local flow time series.
- Identified the function for each option that most reasonably estimates local flows.

Table 13. Streamgages reviewed for use in estimating local flows on the Calaveras River: data were provided by Corps on 6/22/2010 as part of the CVHS.

USGS or CDEC ID (1)	**Streamgage name (2)**	**Data type (3)**	**Time period (water year) (4)**	**Notes (5)**
—	New Hogan Reservoir unregulated inflow	Daily	1907-2010	Values computed by Corps. Data start January 1, 1907.
NHG	New Hogan Dam (reservoir outflow)	Daily	1963-2009	
		Hourly	1995-2009	Data start January 1, 1995.
MRS	Mormon Slough at Bellota (USACE gage)	Daily	1988-2010	No data reported for the 1994 and 1995 flood season. Some data values are missing. Streamgage data are influenced by regulation.
		Hourly	1996-2010	Some data values are missing.
11308900	Calaveras River below New Hogan Dam near Valley Springs, CA	Daily	1961-2009	Data start January 1, 1961. Streamgage data are influenced by regulation.
11309000	Cosgrove Creek near Valley Springs, CA	Daily	1930-1969	
			1991-2010	Data start January 1, 1991. No data reported for the 1994 and 1995 flood season. Some data values are missing.
11309500	Calaveras River at Jenny Lind, CA	Daily	1907-1966	Data start January 1, 1907. Some data values are missing, particularly in the summer months. Streamgage data are influenced by regulation.
11310500	Calaveras River near Stockton, CA	Daily	1926	Data for 1 major flood event only. Streamgage data are influenced by regulation.
			1944-1950	Data for 1 major flood event only for each water year. Streamgage data are influenced by regulation.
			1976-1986	Some data values are missing. Streamgage data are influenced by regulation.

Event selection for local flow estimation analysis

As previously noted, local flows developed were used to support the development of an unregulated-regulated flow transform and a family of regulated characteristic curves. A key aspect in the development of these was the scaling of the largest events, i.e., the 19 events previously indentified for the Calaveras River.

Thus, the local flows estimated for these large events needed to be reasonable and as accurate as possible. To assess this, we used the local flows calculated directly corresponding to the largest events possible as a basis of comparison. Specifically, we used the 1997, 1998, and 2006 water year events whenever possible. Although the 2006 event is not included in 19 events previously indentified (because it is the 10th largest event on record on the Calaveras River and occurred after the completion of the Comp Study), it was useful in developing local flows. We defined the 2006 water year event as starting on 3/24/2006 and ending on 4/30/2006.

Local flow estimation Option 1: Calculate local flows directly

The preferred option for estimating local flows was to calculate directly flows using streamgage data. In general, this was completed on the Calaveras River using known releases from New Hogan Reservoir and the observed flows at Bellota. This was completed only for the time periods when data overlap.

In the case of daily data, local flows were calculated directly by subtracting the reservoir releases from the gaged flows. Any resulting negative values were then set to 0. Routing of the daily observed outflows (using the 1-hour hydrologic routing model of the Calaveras River) was not necessary because the total travel time between New Hogan Reservoir and Bellota is less than 1-day.

Accepted travel time estimates between New Hogan Reservoir and Bellota are: (1) 3 hours as indicated in the New Hogan Reservoir water control manual (Corps 1983), and (2) 7.05 hours as indicated by the sum of Muskigum K values from the HEC-ResSim model provided by the Corps. This longer travel time was attributed to the availability of hourly streamgage data after 1995 used to calibrate the reservoir simulation and hydrologic routing model of the Calaveras River, and was adopted for this analysis.

In the case of hourly data, reservoir releases were first routed from New Hogan downstream to the gage at Bellota. These routed releases were then subtracted from the observed flows to calculate local flow directly. Again, any resulting negative values are then set to 0. We used hydrologic routing to estimate local flows on the Calaveras River. Specifically, we used HEC-DSS math utilities and the Muskingum routing parameters from the CVHS HEC-ResSim model as shown in Table 14. In Table 14, column 2 lists the reach, column 3 lists the Muskingum K values in hours, column 4 lists the Muskingum X, and column 5 the number of subreaches.

In Table 15 we summarize how local flows were calculated directly by time period and data type. In Table 15, column 2 lists the data type, column 3 the overlapping time period, and column 4 the components for calculating local flows.

In Figure 18 through Figure 20 we compared the daily and hourly inferred local flows for the 1997, 1998, and 2006 water year events. (These events are the 3 largest of the overlapping time period for which we could calculate both daily and hourly local flows.) In Figure 18 through Figure 20 the daily local flows are shown in red, the hourly local flows in blue, and the daily differences in their volumes (daily local flows minus hourly local flows) in green. From these comparisons we see (1) that the timing of the hourly and daily local flows are similar, and (2) the differences in volume appear to be greatest around the largest local flows associated with the event. These differences in volumes are small compared to the total volume of unregulated inflow to New Hogan Reservoir.

Table 14. Calaveras River Muskingum routing parameters between New Hogan Reservoir and Bellota

ID (1)	**Reach (2)**	**Muskingum K (hours) (3)**	**Muskingum X (4)**	**Number of subreaches (5)**
1	New Hogan Reservoir to Cosgrove Creek[1]	—	—	—
2	Cosgrove Creek to Jenny Lind	1.05	0.2	1
3	Jenny Lind to Indian Creek	2.5	0.2	2
4	Indian Creek to Duck Creek	2.0	0.2	2
5	Duck Creek to Bellota	1.5	0.2	2
6	**Total**	**7.05**	**—**	**—**

Notes:

1. There was no routing for this reach.

Table 15. Summary of direct calculation of local flows on the Calaveras River

ID (1)	**Data type (2)**	**Overlapping time period[1] (water year) (3)**	**Calculate local flows directly by:[2] (4)**
1	Daily	1988-2009	Subtracting known outflows from New Hogan Reservoir from observed flows at Bellota
2	Hourly	1996-2009	Routing known outflows from New Hogan Reservoir, then subtracting these routed flows from observed flows at Bellota

Notes:

1. Because of missing values, local flow may not be calculated directly for the entire period listed. In such cases flows are either interpolated using the directly calculated flow, or Option 2 or Option 3 depending on data availability.
2. Any resultant negative values were set to 0.

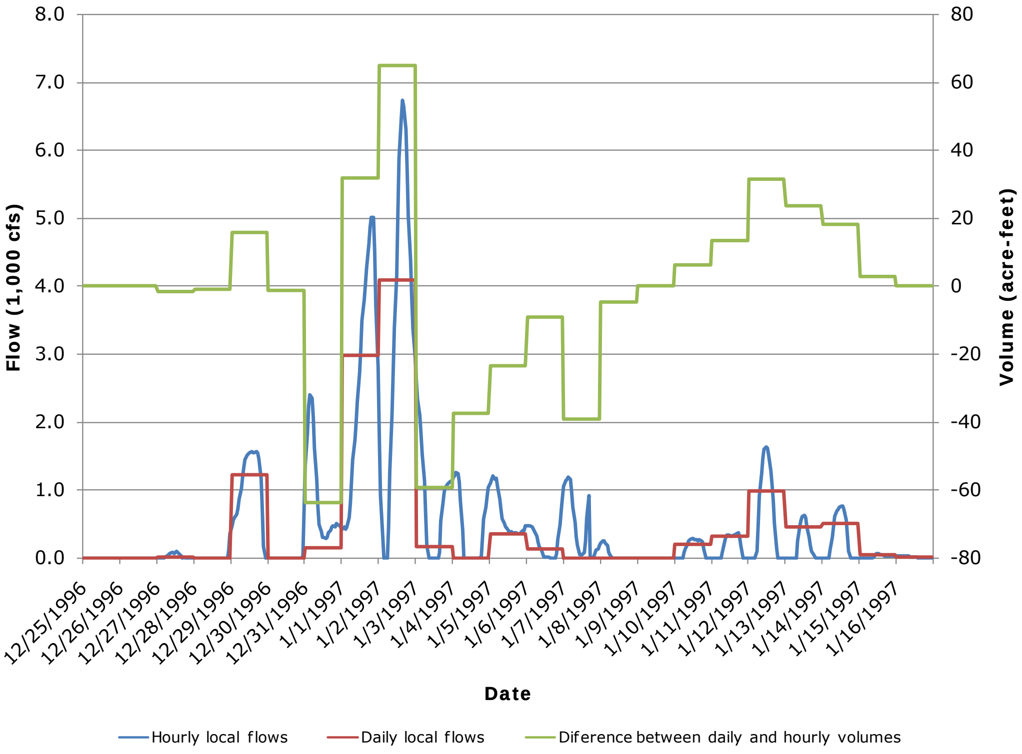


Figure 18. Calaveras River 1997 event directly calculated local flows

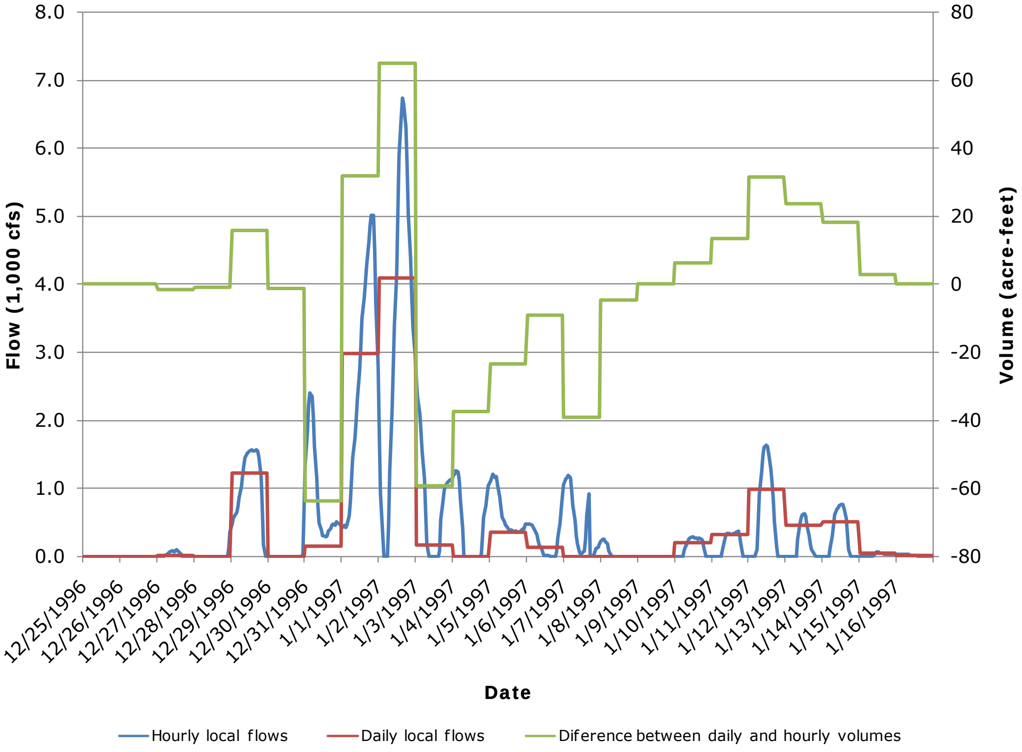


Figure 19. Calaveras River 1998 event directly calculated local flows

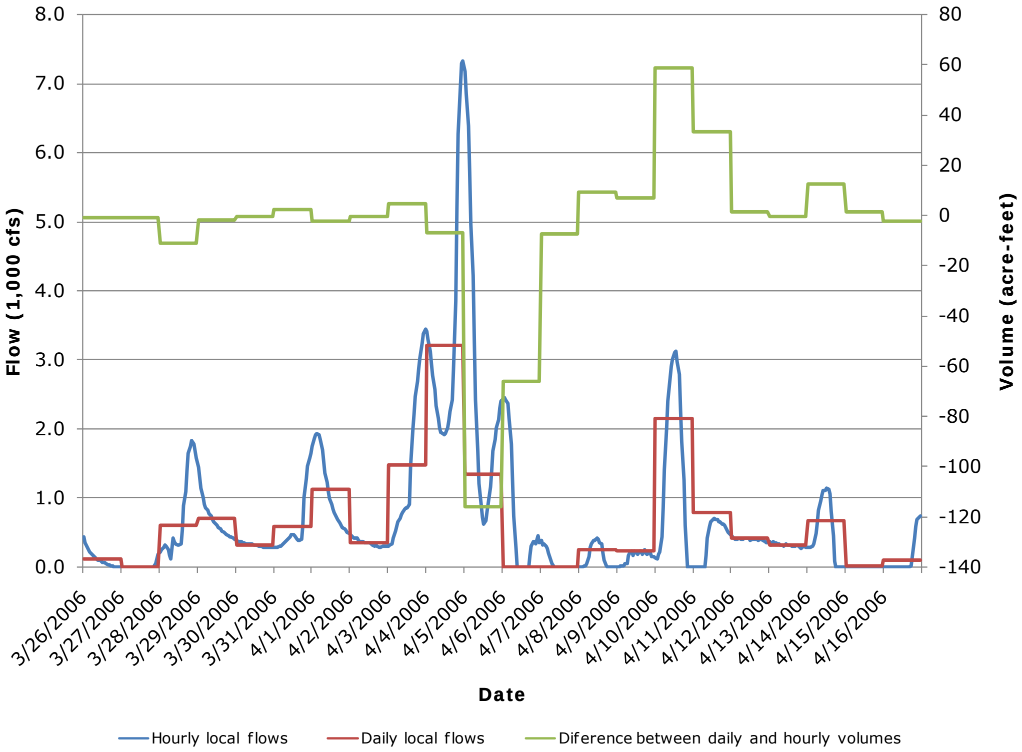


Figure 20. Calaveras River 2006 event directly calculated local flows

Local flow estimation Option 2: Estimate local flows as a function of observed flows of Cosgrove Creek

In the cases where local flows could not be calculated directly, we estimated local flows using nearby streamgages. As noted above, the Corps already estimates local flows using coefficients for reservoir operations on the Calaveras River as 3.2 times the observed flow at the Cosgrove Creek near Valley Springs, CA, streamgage. Because the estimation of local flows is important to simulate accurately reservoir operations we need to either (1) verify the coefficients used by the Corps to estimate such flows, or (2) adopt new coefficients. We completed this task by:

- Calculating local flows directly as detailed in the previous subsection.
- Comparing the directly calculated local flow time series to observed flows on Cosgrove Creek for selected large events occurring in the overlapping period of record.
- Identifying an average ratio of maximum 1-day flows on Cosgrove Creek to directly calculated peak local flows for selected large events.
- Estimating local flow time series using the average ratio identified as a multiplier of unregulated reservoir inflow.
- Estimating local flow time series using a drainage area ratio between the local flow area and Cosgrove Creek watershed as a multiplier to observed flows on Cosgrove Creek.

- Completing regression analyses that relate the directly calculated local flows to the observed flows on Cosgrove Creek for the overlapping periods of record. (Note that the Bear Creek near Lockeford, CA streamgage was also considered for regression analysis, however none of the record overlaps the period for which we can infer local flows directly and therefore the data were not used.)
- Identifying the best fitted functions from the regression analysis for estimation of local flows.
- Estimating local flow time series using the identified functions.
- Estimating a local flow time series using the observed flow on Cosgrove Creek and the 3.2 multiplier used by the Corps.
- Comparing the estimated local flow time series to the directly calculated local flow time series.
- Identifying the function that most reasonably estimates local flows.

Based on this analysis, we identified the best relation for estimating local flows using observed flow on Cosgrove Creek to be the function currently used by the Corps. Thus, we estimated local flows as:

$$Q_{Local} = 3.2\left(Q_{Cosgrove}\right) \tag{3}$$

where Q_{Local} is the local flow estimate for a given time, and $Q_{Cosgrove}$ is the observed flow at the Cosgrove Creek near Valley Springs, CA, streamgage for that same time. All estimated local flows using this option were on a daily basis. We did not lag or route the estimated flows because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the Cosgrove Creek gage and Bellota is approximately 7 hours, which is less than the 1-day time step of the observed flows.

Local flow estimation Option 3: Estimate local flows as a function of unregulated inflow to New Hogan Reservoir

In the cases where local flows could not be inferred directly or estimated using nearby streamgages, we estimated local flows using reservoir inflows. We determined the function that most reasonably estimates local flow using the same procedure previously detailed for estimating flows as a function of observed flows on Cosgrove Creek.

Based on this analysis, we identified the best function for estimating local flows using unregulated inflows to New Hogan Reservoir as:

$$Q_{Local} = 0.226\left(Q_{NHG}\right) \tag{4}$$

where Q_{Local} is the local flow estimate for a given time, and Q_{NHG} is the unregulated inflow to New Hogan Reservoir. All estimated local flows using this option were on a daily basis. We did not lag or route the estimated flows because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the Cosgrove Creek gage and Bellota is approximately 7 hours, which is less than the 1-day time step of the inflows.

Local flow estimation details

The selected estimation approaches, in order of best estimate of local flow, are:

- Option 1. Calculate local flow directly using known releases from New Hogan Reservoir and the observed flows at Bellota, routing hourly flows as necessary. Note in the case of missing streamgage data, local flows values were interpolated as needed.
- Option 2. Estimate local flow as 3.2 times the observed flow at the Cosgrove Creek near Valley Springs, CA, streamgage.
- Option 3. Estimate local flow as 0.226 times the unregulated inflow to New Hogan Reservoir.

We detail the development of the local flow time series for New Hogan Reservoir in Table 16. Column 1 notes the time period for which the option listed in column 3 will be used to estimate local flow, and column 2 lists the time step (hourly or daily) of the developed local flow time series. We interpolated local flows using other estimated local flows as appropriate. The hourly and daily time series were combined and these finalized time series stored as hourly data in HEC-DSS.

Table 16. Local flow time series calculation details by time period

Time period (date) (1)	Time step (2)	Approach to be used (3)
1/1/1907-9/30/1929	Daily	Option 3: 0.226 times reservoir inflow.
10/1/1929-9/30/1969	Daily	Option 2: 3.2 times Cosgrove Creek flow.
10/1/1969-12/31/1987	Daily	Option 3: 0.226 times reservoir inflow.
1/1/1988-9/19/1988	Daily	Option 1: directly infer local flow.
9/20/1988-3/25/1989	Daily	Option 3: 0.226 times reservoir inflow.
3/26/1989-3/29/1989	Daily	Option 1: directly infer local flow.
3/30/1989-5/1/1989	Daily	Option 3: 0.226 times reservoir inflow.
5/2/1989-8/13/1989	Daily	Option 1: directly infer local flow.
8/14/1989-1/3/1990	Daily	Option 3: 0.226 times reservoir inflow.
1/4/1990-2/27/1991	Daily	Option 1: directly infer local flow.
2/28/1991-3/6/1991	Daily	Option 2: 3.2 times Cosgrove Creek flow.
3/7/1991	Daily	Option 1: directly infer local flow
3/8/1991-3/11/1991	Daily	Option 2: 3.2 times Cosgrove Creek flow.
3/12/1991-3/25/1991	Daily	Option 1: directly infer local flow.
3/27/1991-9/30/1991	Daily	Option 1: directly infer local flow.
10/1/1991-12/31/1991	Daily	Option 2: 3.2 times Cosgrove Creek flow.
1/1/1992-11/1/1993	Daily	Option 1: directly infer local flow.
11/2/1993-6/1/1995	Daily	Option 3: 0.226 times reservoir inflow.
6/2/1995-10/20/1995	Daily	Option 2: 3.2 times Cosgrove Creek flow.
10/21/1995-12/15/1995	Hourly	Option 1: directly infer local flow.
12/16/1995-12/20/1995	Daily	Option 2: 3.2 times Cosgrove Creek flow.
12/21/1995	Hourly	Option 1: directly infer local flow.
12/22/1995	Daily	Option 3: 0.226 times reservoir inflow.
12/23/1995	Daily	Option 2: 3.2 times Cosgrove Creek flow.
12/24/1995-12/25/1995	Hourly	Option 1: directly infer local flow.
12/26/1995-1/2/1996	Daily	Option 2: 3.2 times Cosgrove Creek flow.
1/3/1996-8/13/2009	Hourly	Option 1: directly infer local flow.
8/14/2009-3/14/2010	Daily	Option 2: 3.2 times Cosgrove Creek flow.
3/15/2010-7/8/2010	Daily	Option 3: 0.226 times reservoir inflow.

Attachment 3: Annual maximum series for unregulated frequency curves

Here we list the series of annual maximum unregulated volume values that we used in development of the unregulated frequency curves for New Hogan Reservoir and at Bellota. In addition, we include here the unregulated peak inflow annual maximum series for New Hogan Reservoir. Development of a peak flow-frequency curves is not required for development of the regulated flow-frequency curves. However, we developed such curves for completeness.

Annual maximum series

For the New Hogan Reservoir, the unregulated reservoir inflow time series was used as the basis of the unregulated frequency analysis. The Corps provided the finalized unregulated inflow time series for New Hogan Reservoir on 7/12/2010. From this time series, we extracted the 1-, 3-, 7-, 15-, and 30-day volume data. We list these values for New Hogan Reservoir in Table 17. In the table, column 1 lists the water year, and columns 2 through 11 list the date, if available, and the volume, as average flow for the given duration, in cfs. The dates listed in Table 17 correspond to the start of the duration.

To develop annual maximum series for New Hogan Reservoir's operation point on the Calaveras River at Bellota, we combined the unregulated inflow time series with the estimated local flows by adding the 2 time series together using HEC-DSS math utilities. Note that we did not route the unregulated reservoir inflows because the travel time between the reservoir and the operation point is less than the time step of the inflows: 1 day.

Using these data, we computed the 1-, 3-, 7-, 15-, and 30-day volume-duration data using HEC-SSP version 1.1. We list these values for Bellota in Table 18. In the table, column 1 lists the water year, and columns 2 through 11 list the date, if available, and the volume, as average flow for the given duration, in cfs. The dates listed in Table 18 correspond to the start of the duration.

In addition, we reviewed the computed values for consistency. Specifically, we checked that the extracted value for a given duration is less than the values associated with each shorter duration in a given water year. For both analysis locations, we found that the computed values for each water year decrease as duration increases.

Table 17. New Hogan Reservoir annual maximum series for unregulated volume-frequency analysis

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1907	3/19/1907	23,641	3/19/1907	13,508	3/23/1907	9,285	3/24/1907	7,065	4/2/1907	4,550
1908	2/10/1908	2,028	2/11/1908	1,122	2/14/1908	620	1/28/1908	473	2/12/1908	429
1909	1/21/1909	17,875	1/22/1909	8,188	1/26/1909	5,176	1/27/1909	4,474	2/12/1909	3,374
1910	12/9/1909	7,150	12/9/1909	3,344	12/11/1909	2,098	12/15/1909	1,463	1/3/1910	919
1911	1/31/1911	30,175	2/1/1911	20,489	1/31/1911	10,686	2/3/1911	6,714	2/10/1911	4,402
1912	3/13/1912	1,076	3/15/1912	642	3/19/1912	480	3/20/1912	369	4/4/1912	249
1913	1/19/1913	1,278	1/19/1913	779	1/21/1913	557	1/29/1913	345	2/13/1913	202
1914	2/21/1914	8,745	2/21/1914	6,179	1/28/1914	3,972	1/28/1914	2,793	1/29/1914	1,926
1915	2/1/1915	8,092	2/3/1915	6,922	2/3/1915	4,480	2/11/1915	3,610	2/26/1915	2,320
1916	3/20/1916	9,543	3/22/1916	4,520	1/30/1916	2,978	1/28/1916	2,594	2/7/1916	2,197
1917	2/21/1917	18,932	2/23/1917	13,742	2/27/1917	8,302	3/6/1917	4,631	3/20/1917	2,729
1918	3/11/1918	16,241	3/12/1918	11,737	3/13/1918	6,641	3/21/1918	3,859	3/24/1918	2,279
1919	2/11/1919	7,150	2/12/1919	3,802	2/16/1919	1,844	2/24/1919	1,022	3/11/1919	849
1920	3/17/1920	2,854	3/23/1920	2,386	3/22/1920	1,908	3/24/1920	1,263	3/30/1920	835
1921	1/18/1921	23,641	1/20/1921	10,943	1/23/1921	5,251	1/31/1921	3,267	2/15/1921	1,951
1922	2/20/1922	9,024	2/11/1922	7,608	2/14/1922	3,873	2/23/1922	3,068	3/9/1922	1,804
1923	12/13/1922	6,756	12/14/1922	5,234	12/16/1922	2,931	12/21/1922	1,632	1/5/1923	1,093
1924	2/6/1924	173	2/8/1924	162	2/12/1924	105	2/15/1924	81	2/15/1924	61
1925	2/6/1925	12,685	2/7/1925	6,333	2/10/1925	3,296	2/18/1925	2,073	3/5/1925	1,370
1926	2/14/1926	2,941	2/14/1926	2,508	2/18/1926	1,494	2/17/1926	978	2/28/1926	642
1927	2/4/1927	5,747	2/5/1927	3,495	2/21/1927	2,571	2/18/1927	1,658	3/4/1927	1,355
1928	3/25/1928	10,283	3/26/1928	6,490	3/30/1928	4,187	4/7/1928	2,371	4/22/1928	1,314
1929	2/4/1929	1,557	2/5/1929	980	2/8/1929	578	2/15/1929	325	2/17/1929	218
1930	3/6/1930	3,460	3/7/1930	3,053	3/10/1930	1,758	3/9/1930	1,151	3/24/1930	714

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1931	2/15/1931	866	2/17/1931	492	2/21/1931	380	2/28/1931	244	3/15/1931	171
1932	12/28/1931	11,600	2/9/1932	8,430	2/11/1932	5,501	2/14/1932	3,905	2/28/1932	2,177
1933	1/28/1933	1,866	1/30/1933	1,688	1/30/1933	1,262	2/3/1933	751	2/18/1933	509
1934	1/2/1934	5,262	1/2/1934	3,556	1/4/1934	2,490	3/5/1934	1,364	3/9/1934	831
1935	3/8/1935	7,270	4/10/1935	5,745	4/10/1935	4,065	4/18/1935	2,941	5/2/1935	1,893
1936	2/23/1936	26,987	2/24/1936	21,856	2/26/1936	12,506	2/26/1936	11,470	3/2/1936	6,484
1937	2/6/1937	17,805	2/7/1937	15,114	2/10/1937	7,987	2/16/1937	5,462	2/27/1937	3,490
1938	2/11/1938	30,450	2/13/1938	20,914	2/16/1938	13,451	2/15/1938	9,114	3/4/1938	5,637
1939	2/8/1939	2,387	2/9/1939	1,281	2/13/1939	751	2/14/1939	506	3/1/1939	350
1940	3/4/1940	13,610	2/29/1940	10,597	3/4/1940	8,262	3/8/1940	4,750	3/4/1940	2,800
1941	4/4/1941	9,036	3/3/1941	6,660	3/6/1941	4,742	3/8/1941	2,983	3/9/1941	2,629
1942	1/27/1942	15,522	1/28/1942	11,557	1/30/1942	8,104	2/7/1942	5,287	2/21/1942	3,128
1943	1/21/1943	12,420	1/23/1943	9,336	3/11/1943	8,229	3/19/1943	5,619	3/26/1943	3,825
1944	2/3/1944	6,498	2/5/1944	4,471	2/8/1944	2,608	2/16/1944	1,617	3/2/1944	1,021
1945	12/23/1944	4,221	12/24/1944	3,351	12/28/1944	2,757	1/5/1945	1,881	1/19/1945	1,185
1946	3/10/1946	1,295	3/12/1946	980	3/16/1946	654	3/18/1946	448	4/8/1946	403
1947	3/25/1947	1,557	4/8/1947	1,071	4/25/1947	946	5/2/1947	890	5/3/1947	832
1948	3/3/1948	4,469	3/5/1948	2,287	3/8/1948	1,243	3/16/1948	892	3/31/1948	697
1949	2/6/1949	2,683	2/6/1949	2,209	2/10/1949	1,469	2/18/1949	902	2/15/1949	750
1950	11/18/1949	9,390	11/20/1949	6,320	11/23/1949	3,377	12/17/1949	1,913	12/16/1949	1,788
1951	11/18/1950	9,390	11/20/1950	6,320	11/23/1950	3,377	12/17/1950	1,913	12/16/1950	1,788
1952	1/15/1952	7,610	1/16/1952	4,819	1/18/1952	3,484	1/26/1952	2,415	1/26/1952	1,821
1953	1/14/1953	1,992	1/15/1953	1,273	1/19/1953	909	1/21/1953	698	1/28/1953	510
1954	2/14/1954	1,717	2/15/1954	1,097	2/19/1954	809	3/30/1954	693	4/7/1954	558
1955	1/1/1955	2,095	1/20/1955	1,078	1/22/1955	701	1/30/1955	435	1/30/1955	373

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1956	12/23/1955	20,156	12/24/1955	13,299	12/28/1955	7,493	1/5/1956	4,134	1/17/1956	2,864
1957	3/6/1957	7,446	3/7/1957	5,410	3/8/1957	3,072	3/10/1957	2,031	3/24/1957	1,185
1958	4/3/1958	32,920	4/4/1958	22,402	4/7/1958	16,071	4/11/1958	9,898	4/13/1958	6,617
1959	2/11/1959	5,823	2/20/1959	3,446	2/22/1959	2,779	2/25/1959	2,128	3/11/1959	1,314
1960	2/8/1960	4,099	2/10/1960	2,779	2/14/1960	1,426	2/15/1960	789	2/23/1960	452
1961	3/17/1961	277	3/18/1961	232	3/21/1961	175	3/29/1961	142	4/13/1961	96
1962	2/15/1962	7,377	2/16/1962	4,116	2/16/1962	3,053	2/22/1962	1,894	3/10/1962	1,323
1963	2/1/1963	9,416	2/2/1963	6,079	2/5/1963	2,854	2/14/1963	1,547	4/26/1963	1,205
1964	1/22/1964	2,623	1/23/1964	1,828	1/27/1964	1,041	2/3/1964	612	2/17/1964	359
1965	12/23/1964	12,789	12/24/1964	8,666	12/28/1964	5,504	1/6/1965	3,902	1/17/1965	2,722
1966	12/30/1965	2,020	12/31/1965	1,720	1/3/1966	984	1/8/1966	626	1/23/1966	369
1967	1/22/1967	6,738	1/23/1967	3,991	4/24/1967	2,900	2/4/1967	2,172	4/29/1967	1,832
1968	2/21/1968	1,647	2/22/1968	1,301	2/23/1968	938	3/1/1968	560	3/17/1968	435
1969	1/21/1969	14,674	1/22/1969	9,511	1/26/1969	7,000	2/2/1969	4,579	2/17/1969	3,103
1970	1/21/1970	7,200	1/16/1970	5,072	1/22/1970	3,548	1/28/1970	2,852	2/8/1970	1,642
1971	12/2/1970	2,983	12/4/1970	2,256	12/5/1970	1,967	12/12/1970	1,176	12/27/1970	929
1972	12/25/1971	4,922	12/27/1971	2,366	12/28/1971	1,486	1/4/1972	791	1/18/1972	434
1973	1/16/1973	7,695	2/12/1973	5,936	2/16/1973	3,730	2/18/1973	2,268	2/14/1973	1,842
1974	3/2/1974	9,124	3/3/1974	4,946	3/7/1974	2,738	3/15/1974	1,722	3/30/1974	1,101
1975	3/25/1975	5,783	3/27/1975	3,401	3/27/1975	2,538	3/28/1975	1,732	4/5/1975	1,259
1976	3/2/1976	240	3/3/1976	176	3/6/1976	128	3/13/1976	91	3/13/1976	74
1977	3/16/1977	112	11/14/1976	63	2/27/1977	38	3/21/1977	29	3/22/1977	28
1978	3/5/1978	5,770	3/6/1978	4,322	1/20/1978	2,622	1/19/1978	1,734	3/7/1978	1,329
1979	2/22/1979	5,388	2/23/1979	4,643	2/25/1979	2,827	3/4/1979	2,183	3/15/1979	1,441
1980	1/14/1980	8,648	1/14/1980	7,385	1/18/1980	4,744	1/24/1980	2,630	3/15/1980	1,630

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1981	1/29/1981	3,160	1/30/1981	2,148	2/2/1981	1,229	2/5/1981	654	4/2/1981	414
1982	1/5/1982	12,321	2/17/1982	9,059	4/4/1982	4,845	4/12/1982	3,808	4/14/1982	2,648
1983	3/13/1983	10,433	3/2/1983	7,318	3/5/1983	4,913	3/14/1983	3,738	3/27/1983	3,108
1984	12/25/1983	8,029	12/27/1983	5,712	12/30/1983	3,712	1/6/1984	2,099	1/1/1984	1,407
1985	2/8/1985	3,769	2/10/1985	1,892	2/14/1985	953	2/22/1985	511	4/4/1985	416
1986	2/17/1986	23,494	2/19/1986	17,022	2/21/1986	11,280	2/27/1986	5,752	3/16/1986	3,858
1987	3/6/1987	1,761	3/7/1987	1,201	3/11/1987	619	3/19/1987	455	4/3/1987	303
1988	1/17/1988	403	1/18/1988	285	1/21/1988	175	1/24/1988	111	2/3/1988	79
1989	3/25/1989	927	3/27/1989	725	3/30/1989	465	3/16/1989	324	3/31/1989	319
1990	2/17/1990	695	2/18/1990	558	2/22/1990	352	3/17/1990	277	3/17/1990	271
1991	3/26/1991	3,939	3/26/1991	2,955	3/28/1991	1,721	4/1/1991	1,091	4/2/1991	666
1992	2/15/1992	5,114	2/15/1992	2,611	2/17/1992	1,938	2/25/1992	1,180	3/11/1992	747
1993	1/13/1993	5,317	1/15/1993	3,831	1/19/1993	3,063	1/21/1993	2,398	1/27/1993	1,538
1994	2/20/1994	909	2/20/1994	722	2/24/1994	531	3/3/1994	340	3/7/1994	242
1995	3/11/1995	10,146	3/12/1995	8,592	3/15/1995	4,792	3/24/1995	3,896	4/1/1995	2,406
1996	2/21/1996	5,653	2/22/1996	4,658	2/25/1996	3,009	3/5/1996	1,991	2/23/1996	1,527
1997	1/2/1997	16,801	1/3/1997	10,759	1/5/1997	6,316	1/4/1997	4,465	1/28/1997	3,273
1998	2/3/1998	16,919	2/4/1998	8,069	2/8/1998	6,548	2/16/1998	4,317	2/27/1998	3,000
1999	2/9/1999	9,084	2/9/1999	5,840	2/13/1999	3,457	2/21/1999	2,361	3/8/1999	1,560
2000	2/14/2000	7,667	2/14/2000	5,974	2/17/2000	3,534	2/25/2000	2,503	3/11/2000	1,965
2001	3/5/2001	2,094	3/6/2001	1,303	3/9/2001	771	3/6/2001	623	3/11/2001	497
2002	1/3/2002	2,027	1/4/2002	1,439	1/4/2002	1,241	1/4/2002	710	1/12/2002	452
2003	12/16/2002	1,488	12/18/2002	1,087	12/21/2002	685	12/30/2002	438	5/11/2003	339
2004	2/26/2004	3,011	2/28/2004	2,039	3/2/2004	1,246	3/3/2004	779	3/16/2004	484
2005	3/23/2005	10,277	3/24/2005	6,101	3/28/2005	3,614	1/13/2005	2,286	1/28/2005	1,384

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
2006	4/4/2006	18,294	4/5/2006	12,106	4/7/2006	7,121	4/8/2006	4,518	4/23/2006	3,101
2007	2/27/2007	2,715	2/28/2007	1,937	3/3/2007	1,147	3/8/2007	652	3/10/2007	468
2008	1/28/2008	2,313	1/29/2008	1,309	2/3/2008	995	2/6/2008	843	2/21/2008	494
2009	3/4/2009	4,310	3/5/2009	2,592	3/8/2009	1,470	3/9/2009	902	3/14/2009	629
2010	1/22/2010	3,054	1/22/2010	2,547	1/25/2010	1,591	2/1/2010	904	2/16/2010	580

Table 18. Calaveras River at Bellota annual maximum series for unregulated volume-frequency analysis

Water year (1)	**Date of 1-day max volume (2)**	**1-day max volume (cfs) (3)**	**Date of 3-day max volume (4)**	**3-day max volume (cfs) (5)**	**Date of 7-day max volume (6)**	**7-day max volume (cfs) (7)**	**Date of 15-day max volume (8)**	**15-day max volume (cfs) (9)**	**Date of 30-day max volume (10)**	**30-day max volume (cfs) (11)**
1907	3/19/1907	28,983	3/19/1907	16,561	3/23/1907	11,383	3/24/1907	8,661	4/2/1907	5,578
1908	2/10/1908	2,486	2/11/1908	1,375	2/14/1908	760	1/28/1908	580	2/12/1908	526
1909	1/21/1909	21,914	1/22/1909	10,038	1/26/1909	6,345	1/27/1909	5,485	2/12/1909	4,137
1910	12/9/1909	8,766	12/9/1909	4,100	12/11/1909	2,572	12/15/1909	1,793	1/3/1910	1,126
1911	1/31/1911	36,995	2/1/1911	25,119	1/31/1911	13,101	2/3/1911	8,231	2/10/1911	5,397
1912	3/13/1912	1,320	3/15/1912	787	3/19/1912	589	3/20/1912	453	4/4/1912	305
1913	1/19/1913	1,567	1/19/1913	955	1/21/1913	683	1/29/1913	422	2/13/1913	248
1914	2/21/1914	10,722	2/21/1914	7,576	1/28/1914	4,869	1/28/1914	3,424	1/29/1914	2,362
1915	2/1/1915	9,920	2/3/1915	8,487	2/3/1915	5,492	2/11/1915	4,425	2/26/1915	2,844
1916	3/20/1916	11,699	3/22/1916	5,541	1/30/1916	3,651	1/28/1916	3,180	2/7/1916	2,694
1917	2/21/1917	23,210	2/23/1917	16,848	2/27/1917	10,178	3/6/1917	5,678	3/20/1917	3,346
1918	3/11/1918	19,911	3/12/1918	14,390	3/13/1918	8,141	3/21/1918	4,732	3/24/1918	2,795
1919	2/11/1919	8,766	2/12/1919	4,662	2/16/1919	2,260	2/24/1919	1,252	3/11/1919	1,041
1920	3/17/1920	3,499	3/23/1920	2,926	3/22/1920	2,340	3/24/1920	1,549	3/30/1920	1,023
1921	1/18/1921	28,983	1/20/1921	13,416	1/23/1921	6,438	1/31/1921	4,006	2/15/1921	2,392
1922	2/20/1922	11,063	2/11/1922	9,327	2/14/1922	4,748	2/23/1922	3,762	3/9/1922	2,211
1923	12/13/1922	8,283	12/14/1922	6,417	12/16/1922	3,594	12/21/1922	2,001	1/5/1923	1,340
1924	2/6/1924	212	2/8/1924	198	2/12/1924	129	2/15/1924	99	2/15/1924	74
1925	2/6/1925	15,552	2/7/1925	7,764	2/10/1925	4,041	2/18/1925	2,541	3/5/1925	1,679
1926	2/14/1926	3,605	2/14/1926	3,075	2/18/1926	1,831	2/17/1926	1,199	2/28/1926	788
1927	2/4/1927	7,046	2/5/1927	4,285	2/21/1927	3,153	2/18/1927	2,033	3/4/1927	1,662
1928	3/25/1928	12,607	3/26/1928	7,957	3/30/1928	5,133	4/7/1928	2,907	4/22/1928	1,611
1929	2/4/1929	1,909	2/5/1929	1,201	2/8/1929	709	2/15/1929	399	2/17/1929	267
1930	3/6/1930	3,719	3/7/1930	3,364	3/10/1930	1,966	3/9/1930	1,320	3/23/1930	814

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1931	2/15/1931	927	2/16/1931	522	2/21/1931	418	2/28/1931	267	3/15/1931	183
1932	12/28/1931	12,285	2/9/1932	9,182	2/11/1932	6,107	2/14/1932	4,291	2/28/1932	2,386
1933	1/28/1933	1,959	1/30/1933	1,807	1/30/1933	1,373	2/3/1933	807	2/18/1933	542
1934	12/30/1933	6,058	1/1/1934	4,090	1/4/1934	2,838	3/5/1934	1,518	3/9/1934	927
1935	3/8/1935	7,430	4/10/1935	6,052	4/10/1935	4,358	4/17/1935	3,121	5/2/1935	1,997
1936	2/23/1936	28,648	2/24/1936	23,679	2/26/1936	13,565	2/26/1936	12,451	3/2/1936	7,023
1937	2/6/1937	19,366	2/7/1937	16,090	2/10/1937	8,591	2/16/1937	5,853	2/27/1937	3,766
1938	2/11/1938	33,263	2/12/1938	22,349	2/16/1938	14,296	2/15/1938	9,795	3/4/1938	6,030
1939	2/8/1939	2,522	2/9/1939	1,406	2/13/1939	816	2/14/1939	546	2/28/1939	372
1940	3/4/1940	13,646	2/29/1940	11,312	3/4/1940	8,606	3/8/1940	5,011	3/4/1940	2,966
1941	4/4/1941	10,534	3/3/1941	7,072	3/6/1941	4,994	3/8/1941	3,128	3/9/1941	2,765
1942	1/27/1942	17,509	1/28/1942	12,913	1/30/1942	8,951	2/7/1942	5,797	2/21/1942	3,398
1943	1/21/1943	13,940	1/23/1943	10,340	3/11/1943	8,966	3/19/1943	6,061	3/25/1943	4,122
1944	2/3/1944	6,587	2/5/1944	4,528	2/8/1944	2,707	2/16/1944	1,684	3/2/1944	1,090
1945	12/23/1944	4,259	12/24/1944	3,373	12/28/1944	2,781	1/5/1945	1,905	1/19/1945	1,200
1946	12/21/1945	1,338	3/12/1946	983	3/16/1946	658	3/18/1946	451	4/8/1946	423
1947	3/25/1947	1,562	4/8/1947	1,075	4/25/1947	947	5/2/1947	890	5/3/1947	833
1948	3/3/1948	4,469	3/5/1948	2,287	3/8/1948	1,244	3/17/1948	900	3/31/1948	749
1949	2/6/1949	2,704	2/6/1949	2,236	2/10/1949	1,495	2/18/1949	919	2/15/1949	762
1950	11/18/1949	9,390	11/20/1949	6,320	11/23/1949	3,377	12/17/1949	1,913	12/16/1949	1,788
1951	11/18/1950	11,646	11/20/1950	7,694	12/9/1950	4,212	12/17/1950	2,490	12/17/1950	2,245
1952	1/15/1952	8,449	1/16/1952	5,405	1/18/1952	3,985	1/26/1952	2,855	1/26/1952	2,139
1953	1/14/1953	2,191	1/15/1953	1,402	1/19/1953	1,067	1/21/1953	832	1/28/1953	603
1954	2/14/1954	1,986	2/15/1954	1,228	2/19/1954	903	3/30/1954	751	4/7/1954	601
1955	1/1/1955	2,735	1/20/1955	1,681	1/21/1955	1,101	1/29/1955	645	1/30/1955	527

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1956	12/23/1955	22,716	12/24/1955	14,792	12/28/1955	8,324	1/5/1956	4,610	1/17/1956	3,254
1957	3/6/1957	7,737	3/7/1957	6,001	3/8/1957	3,413	3/10/1957	2,235	3/24/1957	1,298
1958	4/3/1958	34,868	4/4/1958	24,018	4/7/1958	17,188	4/11/1958	10,513	4/13/1958	7,085
1959	2/11/1959	6,252	2/19/1959	3,826	2/22/1959	3,109	2/25/1959	2,342	3/11/1959	1,434
1960	2/8/1960	4,233	2/10/1960	2,898	2/14/1960	1,485	2/15/1960	816	2/23/1960	466
1961	3/17/1961	299	3/18/1961	246	3/21/1961	183	3/29/1961	148	4/13/1961	99
1962	2/15/1962	8,141	2/16/1962	4,601	2/16/1962	3,493	2/23/1962	2,140	3/10/1962	1,505
1963	2/1/1963	10,568	2/2/1963	6,670	2/5/1963	3,128	2/14/1963	1,735	4/26/1963	1,341
1964	1/22/1964	3,045	1/23/1964	2,233	1/27/1964	1,242	2/2/1964	715	2/16/1964	414
1965	12/23/1964	14,895	12/24/1964	9,950	12/28/1964	6,263	1/6/1965	4,333	1/17/1965	3,012
1966	12/30/1965	2,276	12/31/1965	1,940	1/3/1966	1,110	1/8/1966	700	2/27/1966	412
1967	1/22/1967	7,813	1/23/1967	4,760	4/24/1967	3,303	2/4/1967	2,635	4/29/1967	2,092
1968	2/21/1968	2,133	2/22/1968	1,626	2/23/1968	1,113	3/1/1968	651	3/17/1968	503
1969	1/21/1969	15,548	1/21/1969	10,261	1/26/1969	7,612	2/2/1969	4,996	2/17/1969	3,446
1970	1/21/1970	8,827	1/16/1970	6,218	1/22/1970	4,350	1/28/1970	3,496	2/8/1970	2,014
1971	12/2/1970	3,657	12/4/1970	2,765	12/5/1970	2,412	12/12/1970	1,441	12/27/1970	1,139
1972	12/25/1971	6,034	12/27/1971	2,901	12/28/1971	1,822	1/4/1972	969	1/18/1972	532
1973	1/16/1973	9,434	2/12/1973	7,278	2/16/1973	4,573	2/18/1973	2,781	2/14/1973	2,259
1974	3/2/1974	11,186	3/3/1974	6,064	3/7/1974	3,357	3/15/1974	2,111	3/30/1974	1,350
1975	3/25/1975	7,090	3/27/1975	4,169	3/27/1975	3,112	3/28/1975	2,124	4/5/1975	1,543
1976	3/2/1976	294	3/3/1976	216	3/6/1976	157	3/13/1976	111	3/13/1976	90
1977	3/16/1977	137	11/14/1976	77	2/27/1977	47	3/21/1977	36	3/22/1977	34
1978	3/5/1978	7,074	3/6/1978	5,299	1/20/1978	3,214	1/19/1978	2,126	3/7/1978	1,629
1979	2/22/1979	6,606	2/23/1979	5,693	2/25/1979	3,466	3/4/1979	2,676	3/15/1979	1,766
1980	1/14/1980	10,602	1/14/1980	9,054	1/18/1980	5,816	1/24/1980	3,224	3/15/1980	1,999

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1981	1/29/1981	3,874	1/30/1981	2,633	2/2/1981	1,507	2/5/1981	802	4/2/1981	508
1982	1/5/1982	15,106	2/17/1982	11,106	4/4/1982	5,940	4/12/1982	4,669	4/14/1982	3,247
1983	3/13/1983	12,791	3/2/1983	8,972	3/5/1983	6,024	3/14/1983	4,583	3/27/1983	3,811
1984	12/25/1983	9,844	12/27/1983	7,003	12/30/1983	4,551	1/6/1984	2,573	1/1/1984	1,726
1985	2/8/1985	4,621	2/10/1985	2,320	2/14/1985	1,168	2/22/1985	627	4/4/1985	509
1986	2/17/1986	28,804	2/19/1986	20,869	2/21/1986	13,830	2/27/1986	7,052	3/16/1986	4,730
1987	3/6/1987	2,159	3/7/1987	1,472	3/11/1987	759	3/19/1987	558	4/3/1987	371
1988	4/22/1988	8,595	4/24/1988	8,126	4/26/1988	7,278	4/26/1988	5,733	4/27/1988	5,231
1989	3/25/1989	1,137	3/27/1989	817	3/30/1989	522	3/16/1989	398	3/31/1989	380
1990	3/3/1990	1,167	3/5/1990	709	3/9/1990	561	3/11/1990	425	3/17/1990	360
1991	5/14/1991	7,875	5/15/1991	6,864	5/19/1991	4,914	5/23/1991	3,156	6/11/1991	1,742
1992	2/15/1992	6,982	5/8/1992	3,447	5/9/1992	3,013	6/12/1992	2,669	6/26/1992	1,695
1993	5/5/1993	7,550	5/6/1993	7,021	5/6/1993	5,450	5/6/1993	3,330	1/27/1993	1,857
1994	10/7/1993	1,705	2/20/1994	885	2/24/1994	652	3/3/1994	417	3/7/1994	296
1995	3/11/1995	12,439	3/12/1995	10,533	3/15/1995	5,875	3/24/1995	4,777	4/1/1995	2,950
1996	2/21/1996	6,569	2/22/1996	5,185	2/25/1996	3,251	3/5/1996	2,133	2/23/1996	1,670
1997	1/2/1997	20,116	1/3/1997	13,031	1/5/1997	7,579	1/4/1997	5,455	1/29/1997	3,868
1998	2/3/1998	22,236	2/5/1998	10,599	2/9/1998	8,332	2/16/1998	5,470	2/27/1998	3,856
1999	2/9/1999	11,835	2/9/1999	7,401	2/13/1999	4,228	2/21/1999	2,895	3/8/1999	1,911
2000	2/14/2000	9,281	2/14/2000	7,554	2/17/2000	4,336	2/25/2000	2,998	3/11/2000	2,327
2001	3/5/2001	3,167	3/7/2001	1,823	3/9/2001	1,048	3/7/2001	801	3/11/2001	618
2002	1/3/2002	3,431	1/4/2002	2,174	1/4/2002	1,894	1/4/2002	1,032	1/12/2002	621
2003	12/17/2002	1,920	12/18/2002	1,337	12/21/2002	810	12/30/2002	503	1/13/2003	358
2004	2/27/2004	4,806	2/28/2004	3,147	3/2/2004	1,827	3/3/2004	1,084	3/16/2004	639
2005	3/23/2005	12,358	3/24/2005	7,321	3/28/2005	4,321	1/13/2005	2,950	1/29/2005	1,796

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
2006	4/4/2006	21,665	4/6/2006	14,613	4/7/2006	8,540	4/12/2006	5,409	4/23/2006	3,657
2007	2/27/2007	3,081	2/28/2007	2,147	3/3/2007	1,237	3/8/2007	695	3/10/2007	489
2008	1/28/2008	2,870	1/29/2008	1,553	2/3/2008	1,148	2/6/2008	968	2/21/2008	557
2009	3/4/2009	4,956	3/5/2009	2,935	3/8/2009	1,623	3/9/2009	980	3/14/2009	693
2010	1/20/2010	4,467	1/22/2010	3,664	1/25/2010	2,166	2/1/2010	1,225	2/16/2010	774

Peak annual maximum series

To develop the peak inflow annual maximum series for New Hogan Reservoir, we reviewed the data provided by the Corps and other sources that contain annual maximum series, including:

- New Hogan Reservoir water control manual (USACE 1983a), hereafter referred to as New Hogan WCM.
- Calaveras River reconnaissance report (USACE 1990).
- Peak flow data provided by the Corps on 6/11/2010.

We summarize in Table 19 the data we identified for use in developing flow-frequency curves for New Hogan. Column 1 lists the time period for which data were identified, and column 2 lists the source of these data.

Table 19. Data sources of peak inflow annual maximum series data identified for use in developing flow-frequency curves for New Hogan Reservoir

Time period (water year) (1)	Data source used (2)
1907-1929[1]	Data provided by Corps on 6/11/2010
1930-1979[2]	New Hogan WCM (USACE 1983a)
1980-1988	Calaveras River reconnaissance report (USACE 1990)
1989-2010	Data provided by Corps on 6/11/2010

Notes:

1. Data missing for the 1924 water year.
2. Data missing for the periods 1944-1955, 1960-1963, and 1970 water years.

We list the peak inflow values and, where possible, their associated dates of occurrence, for New Hogan Reservoir in Table 20. In the table, column 1 lists the water year; column 2 lists the date, if available; and column 3 lists the value in cfs.

We did not develop a peak flow-frequency curve for the Calaveras River at Bellota because a series of annual maximum peak flows at this location is not available. A peak unregulated flow-frequency curve is not required for this analysis.

Table 20. New Hogan Reservoir annual maximum peak inflows

Water year (1)	Date of peak inflow (2)	Peak inflow (cfs) (3)
1907	3/19/1907	34,600
1908	2/10/1908	2,110
1909	1/21/1909	33,000
1910	12/9/1910	11,200
1911	1/31/1911	50,000
1912	3/13/1912	1,120
1913	1/19/1913	1,330
1914	1/22/1914	12,100
1915	2/2/1915	9,190
1916	1/17/1916	22,000
1917	2/21/1917	31,300
1918	3/12/1918	21,800
1919	2/11/1919	11,000
1920	3/17/1920	2,970
1921	1/18/1921	37,900
1922	2/9/1922	24,500
1923	12/13/1923	7,030
1924	—	—
1925	2/6/1925	27,500
1926	2/13/1926	12,700
1927	2/3/1927	19,300
1928	3/25/1928	17,300
1929	2/4/1929	3,060
1930	3/5/1930	10,500
1931	2/15/1931	860
1932	2/6/1932	13,000
1933	1/29/1933	2,060
1934	1/1/1934	4,800
1935	3/7/1935	11,000
1936	2/22/1936	35,000
1937	2/6/1937	14,000
1938	2/11/1938	41,000
1939	2/7/1939	1,780
1940	2/27/1940	18,000
1941	4/4/1941	10,800
1942	1/27/1942	18,300
1943	3/6/1943	14,900
1944-1955	—	—

Water year (1)	Date of peak inflow (2)	Peak inflow (cfs) (3)
1956	12/23/1955	31,500
1957	3/6/1957	7,912
1958	4/2/1958	42,000
1959	2/11/1959	6,640
1960-1963	—	—
1964	1/22/1964	4,820
1965	12/23/1964	20,600
1966	12/30/1965	3,720
1967	1/21/1967	17,500
1968	2/21/1968	3,040
1969	1/21/1969	19,300
1970	—	—
1971	12/2/1970	5,480
1972	12/25/1971	9,050
1973	1/16/1973	13,500
1974	3/2/1974	18,000
1975	3/25/1975	9,650
1976	3/2/1976	440
1977	3/16/1977	200
1978	3/5/1978	10,600
1979	2/22/1979	9,940
1980	1/14/1980	17,900
1981	1/29/1981	6,500
1982	3/31/1982	23,600
1983	3/13/1983	19,454
1984	12/25/1983	10,440
1985	2/8/1985	7,100
1986	2/19/1986	32,444
1987	3/6/1987	3,055
1988	1/17/1988	800
1989	3/25/1989	1,467
1990	2/17/1990	1,135
1991	3/26/1991	10,003
1992	2/15/1992	10,581
1993	1/18/1993	11,572
1994	1/22/1994	2,108
1995	3/10/1995	19,616
1996	2/21/1996	9,070
1997	1/3/1997	23,920

Water year (1)	Date of peak inflow (2)	Peak inflow (cfs) (3)
1998	2/3/1998	33,055
1999	2/9/1999	16,129
2000	1/25/2000	13,762
2001	3/5/2001	3,375
2002	1/2/2002	4,221
2003	12/16/2002	4,010
2004	1/1/2004	5,423
2005	3/23/2005	12,107
2006	4/4/2006	25,555
2007	2/26/2007	5,688
2008	1/28/2008	4,490
2009	3/4/2009	9,424
2010	12/25/2009	13,785

Attachment 4: Fitting the unregulated frequency curves

Overview

The purpose of this attachment is to describe the steps taken to fit unregulated frequency curves to annual maximum series. We developed unregulated frequency curves following the procedures specified in *Bulletin 17B* (IACWD 1982), guidance detailed in EM 1110-2-1415 (USACE 1993), and the current standards of practice. Specifically, we:

- Identified the annual maximum series.
- (Task 4.1) Calculated regional skew values for each duration of interest using relationships developed by the USGS.
- (Task 4.2) Fitted LPIII distributions to the annual maximum series following *Bulletin 17B* procedures and Corps guidance using PeakfqSA, the USGS's flow-frequency software with the expected moments algorithm (EMA) option enabled developed by Tim Cohn of the USGS (Cohn 2007).
- Reviewed and adopted the curves, checking them for consistency and comparing them to previously accepted values.

Regional skew values

Bulletin 17B recommends the use of a regional skew value in fitting LPIII distributions to maintain consistency of frequency curves. *Bulletin 17B* also states that such a value can be developed using regression techniques. For the CVHS, the USGS, in cooperation with the Corps, has developed regression equations for regional skew values (USGS 2010). In general, there are 2 equation forms, 1 for peak flows, and 1 for volumes. The coefficients for the volumes change with duration.

The regional skew associated with peak flows is calculated as:

$$\gamma = -0.62 + 1.30\left(1 - e^{\left(-\left(\frac{Elev}{6500}\right)^{2}\right)}\right) \tag{5}$$

where γ is the regional skew value and *Elev* is the average basin elevation in ft (NAVD 88). The associated average variance of prediction (AVP) is 0.14. AVP is analogous to mean square error (MSE) for the purpose of weighting regional and station skew values.

The regional skew associated with volumes is calculated as

$$\gamma = \beta_0 + \beta_1\left(1 - e^{\left(-\left(\frac{Elev}{3600}\right)^{12}\right)}\right) \tag{6}$$

where γ is the regional skew value, *Elev* is the average basin elevation in ft (NAVD 88), and β_0 and β_1 are coefficients based on the duration of interest as shown in Table 21. The associated AVP also varies with duration and is also shown in Table 21.

For this analysis, we used these equations to develop regional skew values for the Calaveras River as shown in Table 22. We used GIS tools to compute average basin elevations for use in the regional skew computations.

Table 21. Duration skew equation parameters

Parameter (1)	**1-day regional skew (2)**	**3-day regional skew (3)**	**7-day regional skew (4)**	**15-day regional skew (5)**	**30-day regional skew (6)**
β_0	-0.7340	-0.6901	-0.5872	-0.6445	-0.6322
β_1	0.6778	0.6764	0.5822	0.5375	0.4277
AVP	0.0485	0.0576	0.0490	0.0521	0.0615

Table 22. Regional skew values

Location (1)	**Elevation (ft) (2)**	**Peak flow regional skew (3)**	**1-day regional skew (4)**	**3-day regional skew (5)**	**7-day regional skew (6)**	**15-day regional skew (7)**	**30-day regional skew (8)**
New Hogan Reservoir	2010.31	-0.501	-0.733	-0.690	-0.587	-0.644	-0.632
Bellota	1662.53	-0.538	-0.734	-0.690	-0.587	-0.644	-0.632

Fitting the curves

As a first step, the curves were fitted using a straightforward *Bulletin 17B* procedure in which all data points were included in the analysis and low outliers were identified by the *Bulletin 17B* outlier test and the station statistics appropriately adjusted. This includes weighting the station skew and regional skew values by the inverse of their associated errors. This weighting procedure is included in *Bulletin 17B* and the weighted skew is automatically calculated by PeakfqSA, which we used here.

We found the frequency curves on the Calaveras River were consistent between durations at each location. The curves do not "cross," and flow quantiles for a given duration at the downstream location were greater than those of the upstream location, as would be expected.

As a comparison, we considered the volume-frequency curves developed for Farmington Reservoir in the Comp Study (USACE 2002). The annual maximum series in the Comp Study ended in 1997.

We then compared the curves fitted at New Hogan Reservoir to the corresponding curves from the Comp Study (USACE 2002). We found that the flow quantiles of the curves fitted here and those of the Comp Study differ between the 2 sets of volume-duration curves by only 1%-13%. The greatest differences (of only 8%-13%) are in the 1-day volume quantiles. The 3-day and 7-day volume quantiles differ by only 1% to 5%. Peak flow-frequency curves varied by as much as 9% because of the increased number of large events included in this analysis as compared to the Comp Study.

Results

The final parameters and statistics used to fit LPIII distributions to develop the unregulated frequency curves at New Hogan Reservoir (shown in Figure 9) are shown in Table 23.

The final parameters and statistics used to fit LPIII distributions to develop the unregulated frequency curves at Bellota (shown in Figure 10) are shown in Table 24.

Table 23. Parameters and statistics to fit unregulated frequency curves: New Hogan Reservoir

Statistic (1)	**Peak flows (2)**	**1-day volumes (3)**	**3-day volumes (4)**	**7-day volumes (5)**	**15-day volumes (6)**	**30-day volumes (7)**
Station mean[1]	3.946	3.684	3.518	3.324	3.146	2.988
Station standard deviation[1]	0.485	0.502	0.488	0.478	0.473	0.459
Station skew[1]	-1.027	-0.979	-0.819	-0.806	-0.682	-0.706
Station skew associated MSE[2]	0.160	0.126	0.107	0.105	0.093	0.095
Regional skew[3]	-0.501	-0.733	-0.690	-0.587	-0.644	-0.632
Regional skew associated AVP[4]	0.140	0.049	0.058	0.049	0.052	0.062
Mean[5]	3.947	3.685	3.518	3.324	3.146	2.988
Standard deviation[5]	0.482	0.501	0.488	0.477	0.473	0.458
Weighted skew[5,6]	-0.727	-0.794	-0.731	-0.651	-0.646	-0.659
Number of systematic events	86	104	104	104	104	104
Number of high outliers	0	0	0	0	0	0
Number of EMA iterations	2	2	2	2	2	2
Number of low outliers	0	2	2	2	2	2
Number of zero events	0	0	0	0	0	0
Number of missing events	18	0	0	0	0	0
Number of EMA censored observations	1	1	1	1	1	1
Corresponding censored events[7]	1). 1977	1). 1977	1). 1977	1). 1977	1). 1977	1). 1977
Record length	104	104	104	104	104	104

Notes:

1. Statistic calculated using the series of logarithmic transforms and EMA without regional skew; rounded to nearest thousandth.
2. Mean square error; rounded to nearest thousandth.
3. Regional skew values calculated using relationships developed by the USGS; rounded to nearest thousandth.
4. Average variance of prediction, analogous to MSE; rounded to nearest thousandth.
5. Statistic calculated using the series of logarithmic transforms and EMA with regional skew; rounded to nearest thousandth.
6. Skew value calculated by weighting the station and regional skew values inversely proportional to their associated errors: (MSE and AVP) and EMA; rounded to nearest thousandth.
7. Events are listed by water year in order of increasing flow or volume.

Table 24. Parameters and statistics to fit unregulated frequency curves: Bellota

Statistic (1)	**1-day volumes (2)**	**3-day volumes (3)**	**7-day volumes (4)**	**15-day volumes (5)**	**30-day volumes (6)**
Station mean[1]	3.774	3.607	3.417	3.239	3.079
Station standard deviation[1]	0.487	0.476	0.465	0.461	0.448
Station skew[1]	-1.112	-0.898	-0.875	-0.729	-0.731
Station skew associated MSE[2]	0.145	0.116	0.113	0.097	0.096
Regional skew[3]	-0.734	-0.690	-0.587	-0.644	-0.632
Regional skew associated AVP[4]	0.049	0.058	0.049	0.052	0.062
Mean[5]	3.775	3.608	3.417	3.240	3.079
Standard deviation[5]	0.482	0.475	0.464	0.461	0.448
Weighted skew[5,6]	-0.810	-0.753	-0.666	-0.671	-0.668
Number of systematic events	104	104	104	104	104
Number of high outliers	0	0	0	0	0
Number of EMA iterations	2	2	2	2	2
Number of low outliers	0	0	0	0	0
Number of zero events	0	0	0	0	0
Number of missing events	0	0	0	0	0
Number of EMA censored observations	2	1	1	1	1
Corresponding censored events[7]	1). 1977 2). 1976	1). 1977	1). 1977	1). 1977	1). 1977
Record length	104	104	104	104	104

Notes:

1. Statistic calculated using the series of logarithmic transforms and EMA without regional skew; rounded to nearest thousandth.
2. Mean square error; rounded to nearest thousandth.
3. Regional skew values calculated using relationships developed by the USGS; rounded to nearest thousandth.
4. Average variance of prediction, analogous to MSE; rounded to nearest thousandth.
5. Statistic calculated using the series of logarithmic transforms and EMA with regional skew; rounded to nearest thousandth.
6. Skew value calculated by weighting the station and regional skew values inversely proportional to their associated errors: (MSE and AVP) and EMA; rounded to nearest thousandth.
7. Events are listed by water year in order of increasing volume.

Attachment 5: Unregulated-regulated flow transforms and critical duration assessment

Fit unregulated-regulated flow transforms

We developed the unregulated-regulated flow transforms for the 2 analysis locations by fitting transform curves through data pairs from the event maxima datasets. Specifically, we fitted transforms to pairs of unregulated volumes (as average flows) and regulated peak flows. For this analysis, we used unregulated volumes associated with the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations. We fitted these curves to the data pairs of historical and scaled events using the robust locally weighted scatterplot smoothing (LOWESS) regression technique. (The LOWESS procedure is detailed in the *Technical procedure document*.)

Here, we used the LOWESS algorithm developed by William Cleveland (Cleveland 1985). We complied an executable of the algorithm, implemented in Fortran. This executable was tested using example data included in the Fortran file.

We used an iterative process to fit these transforms. Specifically we:

- Fitted a candidate transform using the LOWESS regression technique.
- Calculated the mean squared error (MSE) associated with the candidate transform.
- Modified the LOWESS parameters using guidance provided in the literature (Bradley and Potter 2004, Cleveland 1979).
- Fitted another candidate transform and calculated the associated MSE.
- Compared this new transform to the old transform(s) visually and based on MSE.
- Repeated the previous steps until the parameters resulting in the best fit, as determined visually and based on MSE, were identified.

Determine critical duration

For a regulated system, the critical duration is the unregulated flow duration-frequency curve that best characterizes the peak regulated flow-frequency curve at a downstream point. To determine critical duration for each location, we:

- Fitted flow transforms to the event maxima datasets, as detailed in the previous subsection.
- Applied these flow transforms to develop hypothetical regulated flow-frequency curves.
- Identified the duration of the unregulated annual maximum series that estimates the largest flow for each probability of interest, as shown in column 1 of Table 25. Here, we considered 2 criteria: (1) the "goodness of fit" of each transform, and (2) which duration estimates the greater peak regulated flows

Table 25. *Synthesis of information used to determine critical duration*

Annual exceedence probability (1)	Unregulated flow duration (in days) that estimates the largest flow quantile at New Hogan Reservoir (2)	Unregulated flow duration (in days) that estimates the largest flow quantile at Bellota (3)
0.500	2.5	3.5
0.200	1	3
0.100	1	3
0.050	1	2
0.020	1	1
0.010	5	1
0.005	3.5	1
0.002	3.5	2.5

After considering all the durations noted above, for New Hogan Reservoir, we focused on durations of 10 days or less because: (1) the typical unregulated inflow event duration is less than 15 days, and (2) the flow transforms for durations of 10 days or less better fit the event maxima data pairs based on MSE and visual inspection. In addition, the scaled historical event unregulated volumes associated with the longer durations tend to include volumes of additional flood waves after the peak reservoir release. These later flood waves do not contribute to the inflow volumes that drive the reservoir releases, unlike multiple flood waves prior to the peak reservoir releases that are considered. Here, we defined a flood event as the time from when the pool elevation rises from and returns to the top of conversation pool (bottom of flood control pool). For Bellota, we looked at durations equal or less than the critical duration at New Hogan because the addition of unregulated local flows will not cause the critical duration to increase.

In selection of the critical duration, we gave more weight to the durations that estimated the largest flow quantiles for the p=0.01, p=0.005, and p=0.002 annual exceedence events. We used these probabilities because New Hogan Reservoir has large flood storage volume, and regulated peak flows associated with more common events are driven by local flow peaks, not reservoir inflow volumes for a given duration.

From this analysis we determined that the critical duration at New Hogan Reservoir is 3.5 days and at Bellota is 1 day. Thus, the appropriate unregulated-regulated flow transforms used in this analysis were associated with these durations. The critical duration associated with the downstream operation point is shorter than that of the reservoir because of the effects of local flow.

As a "reality check" on our critical duration values, we simulated events, with the HEC-ResSim model, that corresponded to specific volumes associated with a given duration and annual exceedence probability. This is an alternative option for assessing critical duration as detailed in Attachment F of the *Technical Procedures document* as "Method 2: Limited sample, specific volume-duration event scaling." For this check, we scaled reservoir inflows for 4 event patterns (1958, 1986, 1997, and 1998) to the 1-, 3-, 5, and 7-day unregulated flows for the p=0.01 and p=0.005 annual exceedence probabilities. We found: (1) the resulting regulated peaks sensitive to hydrograph shape, and (2) the scaling to the 1-day and 3-day durations estimated largest regulated peak flows. These results are consistent with the adopted critical duration values for the 2 analysis locations.

Review and adopt transforms

After determining the critical duration associated with each analysis location, we reviewed the unregulated-regulated flow transforms initially fitted with the LOWESS procedure to: (1) check for appropriateness, and (2) identify the need for adjustments, if any. As part of this review we:

- Compared event hydrographs of the simulated events that correspond to the transitional areas of the transform (i.e., where the objective peak flows are being constrained, or where peak releases become larger than the objective).
- Fitted additional transforms omitting scaled historical events with scale factors of 2 or less.
- Identified and compared the unregulated volumes that define the "break points" where large floods-of-record and their scaled versions were not controlled by the reservoir because of (1) lack of storage capacity, or (2) local flows larger than the channel capacity.
- Split the unregulated-regulated flow transform initially fitted with LOWESS into 2 ranges using this break point.
- Calculated the MSE for these 2 ranges for each initially fitted LOWESS curve.
- Identified which LOWESS curves have the least MSE for each range.

At New Hogan Reservoir, we found: (1) the LOWESS fitted curves with smoothing coefficients of 0.2 have the lowest MSE for the range of unregulated flows for which the downstream objective flow is met, and (2) the LOWESS fitted curves with smoothing coefficients of 0.5 or greater have a lower MSE for the range in which the downstream objective flow is being exceeded.

Therefore, we blended the 2 "best-fit" LOWESS fitted curves at this break point. We linearly interpolated through the 2 points tangent each curve with the controlling point of tangency nearest to the average "break point" previously identified. We then adjusted the transforms so that the regulated peak flow does not decrease as unregulated volume increases. This blending is seen in Figure 21. In Figure 21 we show the unregulated-regulated flow transforms in black dashes, the floods-of-record event maxima in red squares, the historical scaled event maxima in green triangles, and the initial LOWESS fitted flow transforms in blue and orange for comparison. The blue and orange lines represent the LOWESS fitted curves that best fits event maxima for the more common events and for the more rare events. In Figure 21, the dashed black line represents the recommended transform, including the portion that was blended.

At Bellota, we found that the LOWESS fitted curves with a smoothing coefficient of 0.2 had lowest MSE for ranges of unregulated 1-day volumes both larger and smaller than that associated with the "break point." However, we found that the transform associated with smoothing coefficient of 0.2 does not visually fit the data above this range of interest. Therefore, we completed a sensitivity analysis and found that a smoothing coefficient of 0.24 most appropriately represents this upper range. We blended the 2 “best-fit” LOWESS fitted curves at this point of transition. This blending is seen in Figure 22, in which blue and orange lines represent the LOWESS fitted curves that best fits event maxima for the more common events and for the more rare events.

As a final check, we re-applied the transform to compute the associated regulated flow quantiles. We compared these quantiles to those associated with the original fit, and those associated with the candidate transforms for the other unregulated volumes. For New Hogan Reservoir, we computed a 25% decrease in the p=0.002 quantile. For Bellota, we computed a 1% decrease in the p=0.05 and p=0.02 quantiles. In addition, we re-analyzed the critical duration using the adjusted transform for each analysis location and found them to be consistent with the initial fittings.

Based on this review, we adopted flow transforms for New Hogan Reservoir and Bellota shown in Figure 21 and Figure 22. The tabulated curves are in an MS Excel file on DVD with the original report.

In Figure 21 and Figure 22 we show the unregulated-regulated flow transforms in black, the floods-of-record event maxima in red squares, the historical scaled event maxima in green triangles, and the initial LOWESS fitted flow transforms in blue and orange for comparison. We also show in grey in Figure 21 and Figure 22 the corresponding unregulated volume-duration quantiles for annual exceedence probabilities of interest. In Figure 21 and Figure 22, some scaled historical event maxima for more common events, i.e., annual exceedence probabilities greater than p=0.01, have regulated peaks exceeding the channel capacity (12,500 cfs) because of large local flows.

We show in Table 26 and Table 27 the parameters we used to fit these transforms and the resulting mean square errors. Highlighted in grey in Table 26 and Table 27 are the LOWESS fitted curves with smoothing coefficients listed in column 1 used in fitting the final unregulated-regulated flow transforms over the ranges specified in columns 4 and 5.

Table 26. LOWESS parameters and resulting errors for fitting of unregulated-regulated flow transforms: New Hogan Reservoir

Smoothing coefficient[1] (1)	Number of iterations[2] (2)	Delta[3] (3)	Minimum threshold (1,000 cfs) (4)	Maximum threshold (1,000 cfs) (5)	Total number of data pairs (6)	MSE[4] (7)
0.2	2	0	3	30	250	2,697,208
			3	26	190	312,921
			26	30	60	35,055,387
0.5			3	30	250	2,057,807
			3	26	190	736,368
			26	30	60	19,991,618
Adopted transform			**3**	**30**	**—**	**1,554,705**

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 4 and 5.

Table 27. LOWESS parameters and resulting errors for initial fitting of unregulated-regulated flow transforms: Bellota

Smoothing coefficient[1] **(1)**	**Number of iterations**[2] **(2)**	**Delta**[3] **(3)**	**Minimum threshold (1,000 cfs) (4)**	**Maximum threshold (1,000 cfs) (5)**	**Total number of data pairs (6)**	**MSE**[4] **(7)**
0.2	2	0	6	52	194	10,050,441
			6	43	185	5,897,329
			43	52	9	48,302,794
			52	56	7	326,549,103
0.24			6	52	194	10,158,012
			6	43	185	6,049,790
			43	52	9	57,996,907
			52	56	7	309,817,008
Adopted transform			**6**	**52**	**—**	**10,121,872**

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 4 and 5.

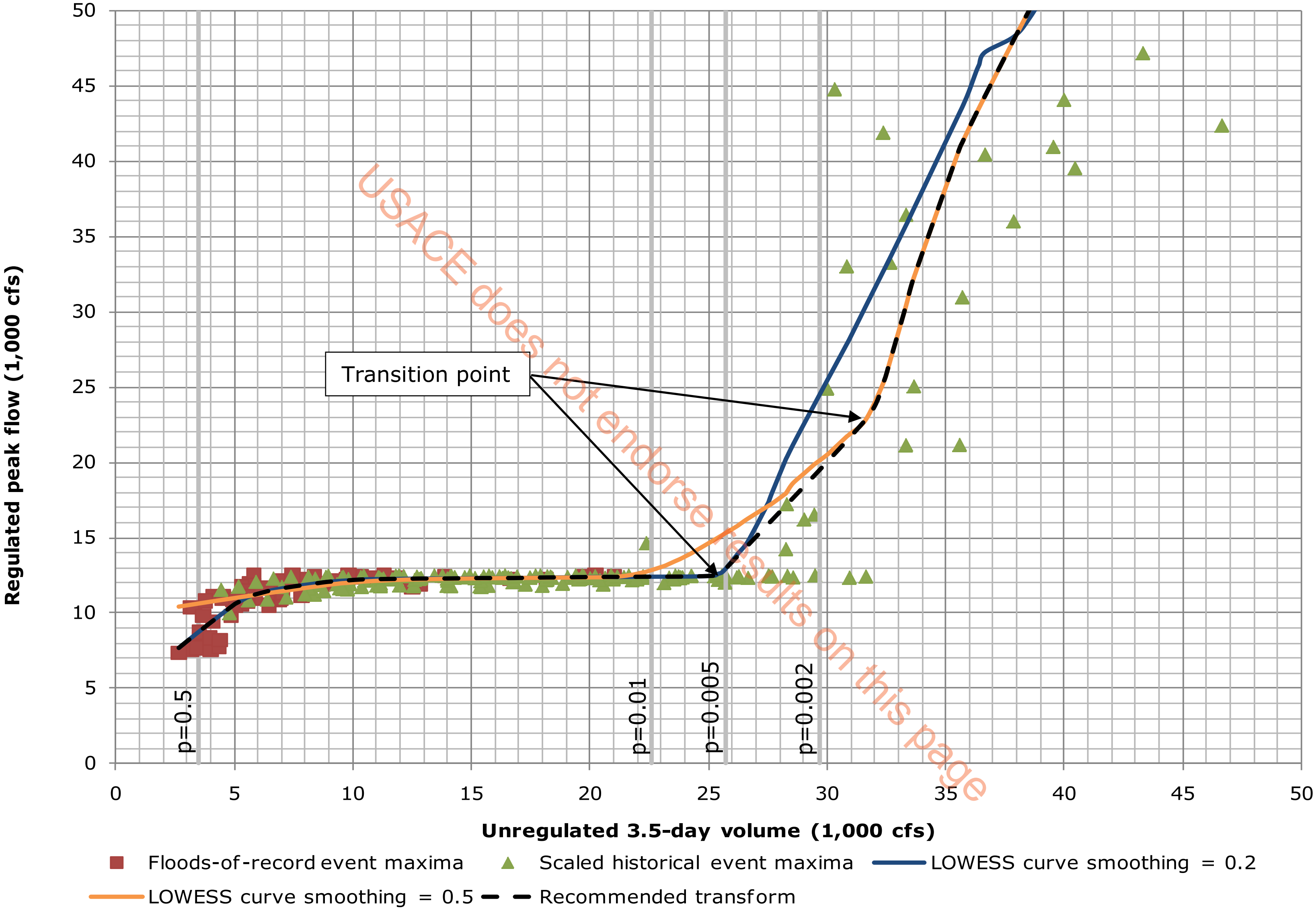


Figure 21. Unregulated-regulated flow transform and LOWESS fitted curves: New Hogan Reservoir

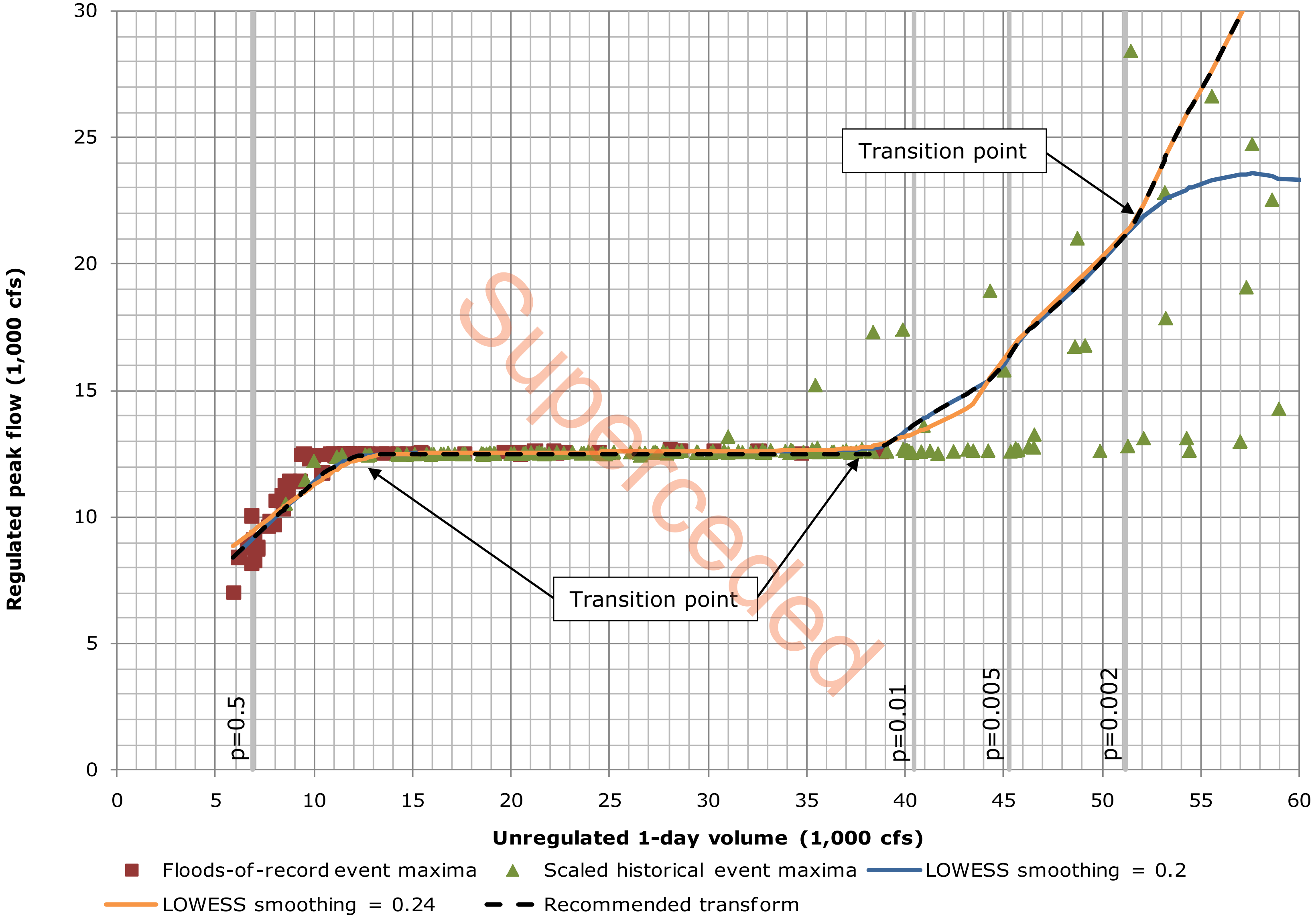


Figure 22. Unregulated-regulated flow transform and LOWESS fitted curve: Bellota

Attachment 6: Family of regulated characteristic curves

Fit the characteristic curves

We used the families of regulated characteristic curves to relate a given regulated peak flow to likely associated regulated volumes at each analysis location. We developed the families of regulated characteristic curves for New Hogan Reservoir and at Bellota by fitting transform curves through the pairs of event regulated volumes, as average flows, and regulated peak flows. The fitting is similar to how we developed the unregulated-regulated transforms detailed in Attachment 5. The datasets we used include both historical and scaled events to define the extreme ends of the flow transform curve.

We initially fitted these curves to the data pairs of historical and scaled events using the LOWESS regression technique and parameters shown in Table 28 and Table 29 for New Hogan Reservoir and at Bellota. In this initial fitting we used the entire event maxima dataset for the given analysis location. Because the flows of interest correspond to events equal or larger than the p=0.5 event, but less than or equal to the p=0.002 event, we truncated the datasets of event pairs to the minimum and maximum regulated flow thresholds specified in columns 5 and 6 of Table 28 and Table 29 for selection of the appropriate LOWESS smoothing coefficient to use in developing the characteristic curves. Highlighted in grey in Table 28 and Table 29 are the LOWESS fitted curves with smoothing coefficients listed in column 2 used in fitting the final characteristic curves for the duration specified in column 1 over the range with minimum and maximum flow thresholds specified in columns 5 and 6.

Review and adopt the characteristic curves

We reviewed and adjusted the curves initially fitted with the LOWESS procedure using the same process detailed for fitting the unregulated-regulated flow transforms. Here, the only difference is that the "break point" is defined by the downstream objective flow (12,500 cfs). Thus the mean square errors in the LOWESS fitted curves were compared over these 2 ranges for each characteristic curve.

From this review we found:

- The family of regulated characteristic curves were consistent between durations at New Hogan Reservoir. That is, they do not cross.
- The family of regulated characteristic curves we initially fitted with LOWESS were inconsistent for events with regulated peaks larger than the channel capacity constraint of 12,500 cfs. This inconsistency is a result of the effect large local flows have at this operation point. Specifically, such large peak local flows contribute to relatively high regulated peak flows for the associated regulated volumes. Therefore, the slope of the characteristic curves at Bellota is less than that seen in the characteristic curves at New Hogan Reservoir, particularly for shorter durations.
- After initially fitting the curves at Bellota, we found that the 3-day and 7-day curves crossed the 1-day curve. Therefore we set the 3-day

characteristic curve equal the 1-day curve at their initial point of intersection, and the 7-day curve equal the 3-day curve at their initial point of intersection.

- The fit of the curves at Bellota was sensitive to large peaks in local flow such as those computed directly for the 1997, 1998, and 2006 events.
- The characteristic 1-, 3, and 7-day volumes at Bellota for events with annual exceedence probabilities equal p=0.002 are less than the characteristic volume associated with New Hogan Reservoir for the same annual exceedence probability because of this effect large local flows had on the fit of the characteristic curves. However, the regulated peak flow at Bellota is always equal or larger than the peak at New Hogan Reservoir for the same exceedence probability.

Based on this review, we adopted the adjusted families of curves.

We show in Figure 23 through Figure 27 the regulated characteristic curves corresponding to New Hogan Reservoir. In addition, we include tabulations of this family of regulated characteristic curves in an MS Excel file on the DVD included with the original report.

We show in Figure 28 though Figure 32 regulated characteristic curves corresponding to Bellota. In addition, we include tabulations of this family of regulated characteristic curves in an MS Excel file on the DVD included with the original report.

In Figure 23 through Figure 32 we show the characteristic curves in black, the floods-of-record event maxima in red squares, the historical scaled event maxima in green triangles, and the initial LOWESS fitted flow curves in blue for comparison.

Table 28. LOWESS parameters for fitting the family of regulated characteristic curves and resulting errors: New Hogan Reservoir

Duration (days) (1)	**Smoothing coefficient[1] (2)**	**Number of iterations[2] (3)**	**Delta[3] (4)**	**Minimum threshold (1,000 cfs) (5)**	**Maximum threshold (1,000 cfs) (6)**	**Total number of data pairs (7)**	**LOWESS curve MSE[4] (8)**	**Characteristic curve MSE (9)**
1	0.2	2	0	8	22	201	285,737	295,465
3	0.7						1,833,013	1,995,342
7	0.2						4,004,463	4,767,174
15	0.2						3,939,439	6,168,764
30	0.2						2,420,500	3,930,845

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to “save intermediate computations,” and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 5 and 6.

Table 29. LOWESS parameters for fitting the family of regulated characteristic curve and resulting errors: Bellota

Duration (days) (1)	Smoothing coefficient[1] (2)	Number of iterations[2] (3)	Delta[3] (4)	Minimum threshold (1,000 cfs) (5)	Maximum threshold (1,000 cfs) (6)	Total number of data pairs (7)	LOWESS curve MSE[4] (8)	Characteristic curve MSE (9)
1	0.7	2	0	8	22	201	510,466	552,352
	0.2			8	13	181	299,883	
	0.7			13	22	20	2,555,554	
3	0.2			8	22	201	1,367,756	1,806,174
	0.2			8	13	181	1,168,187	
	0.7			13	22	20	2,802,423	
7	0.2			8	22	201	2,417,325	7,982,243
15	0.2			8	22	201	3,293,534	21,812,221
30	0.2			8	22	201	2,083,062	19,331,298

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 5 and 6.

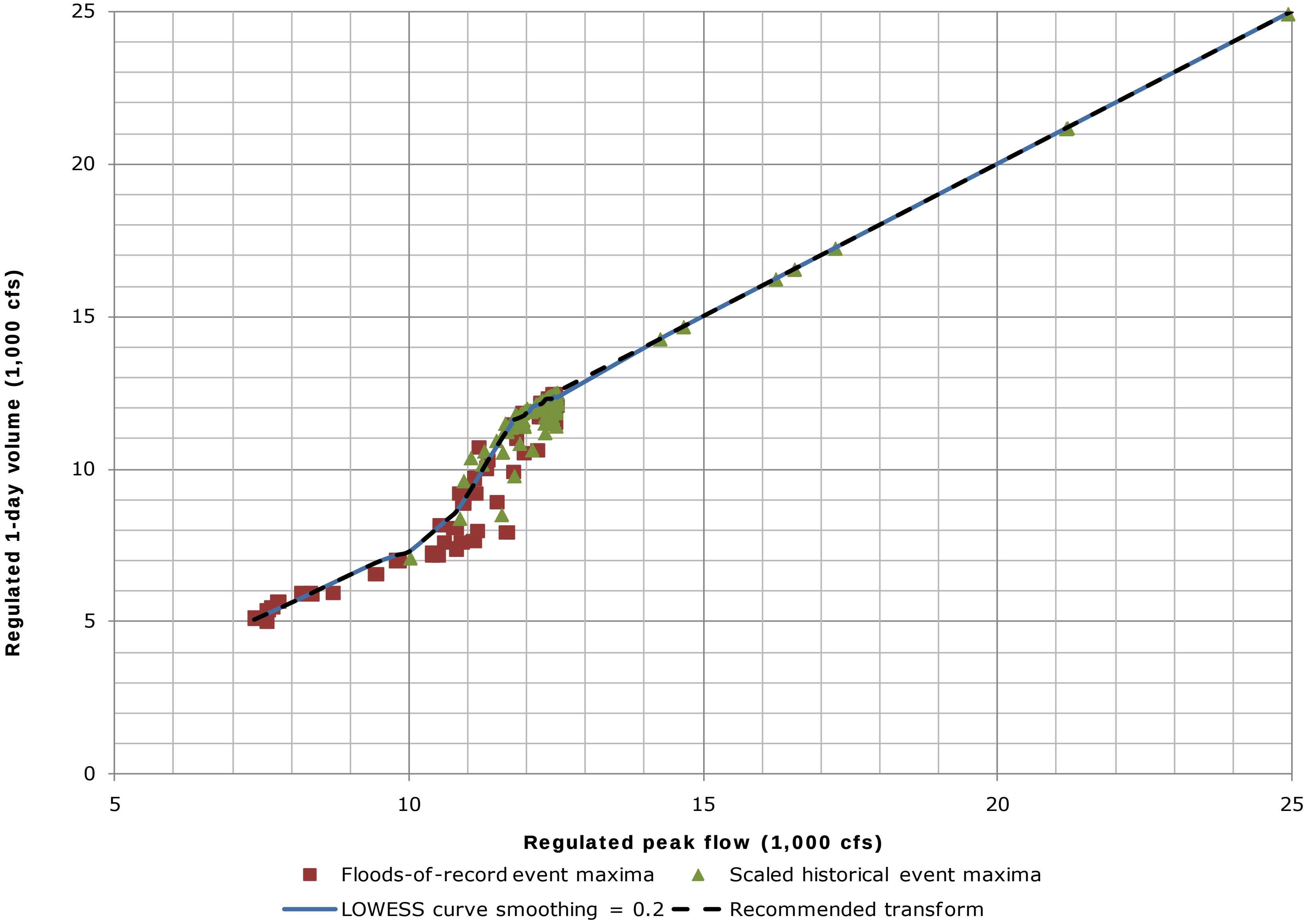


Figure 23. New Hogan Reservoir regulated characteristic curve: 1-day duration

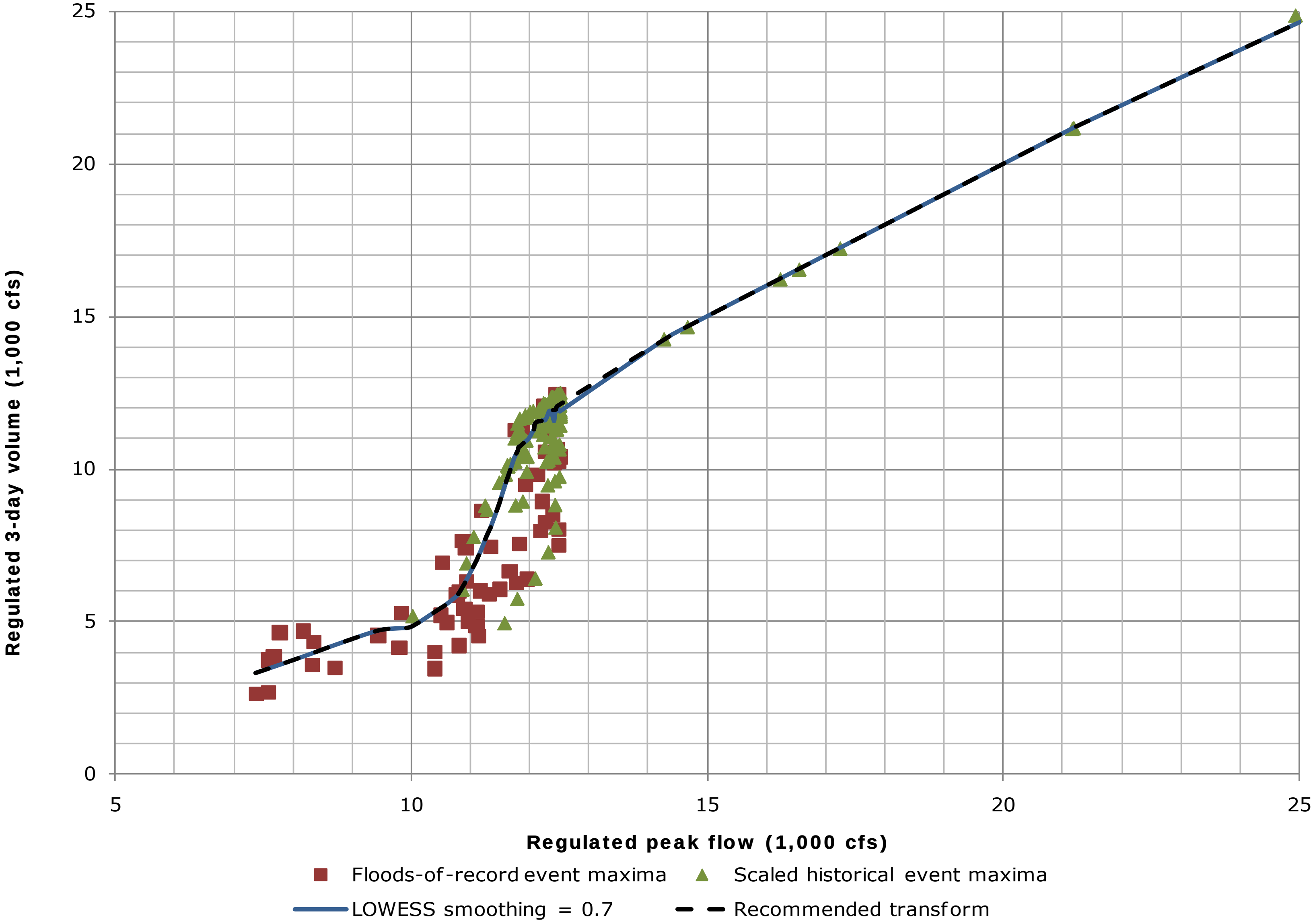


Figure 24. New Hogan Reservoir regulated characteristic curve: 3-day duration

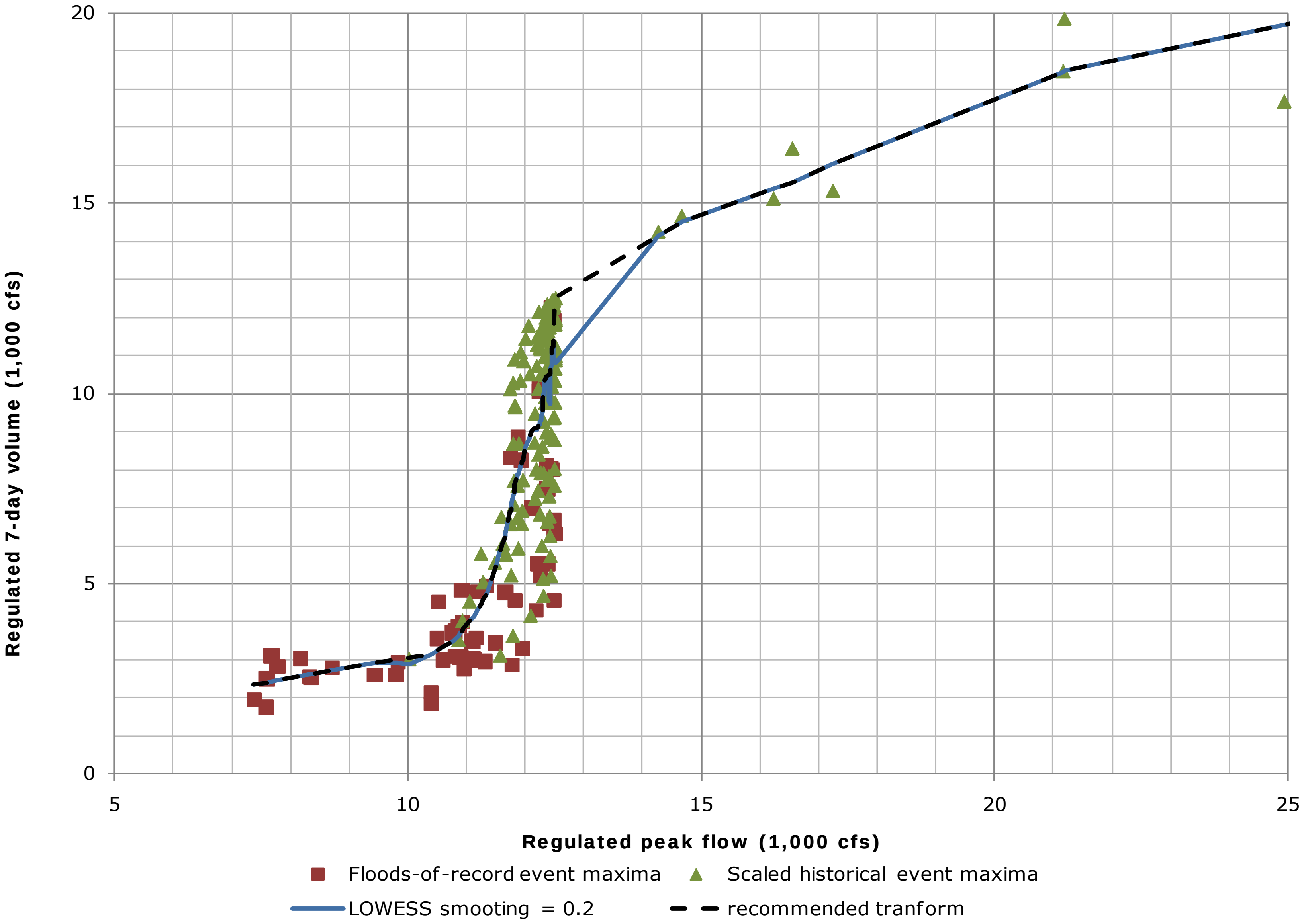


Figure 25. New Hogan Reservoir regulated characteristic curve: 7-day duration

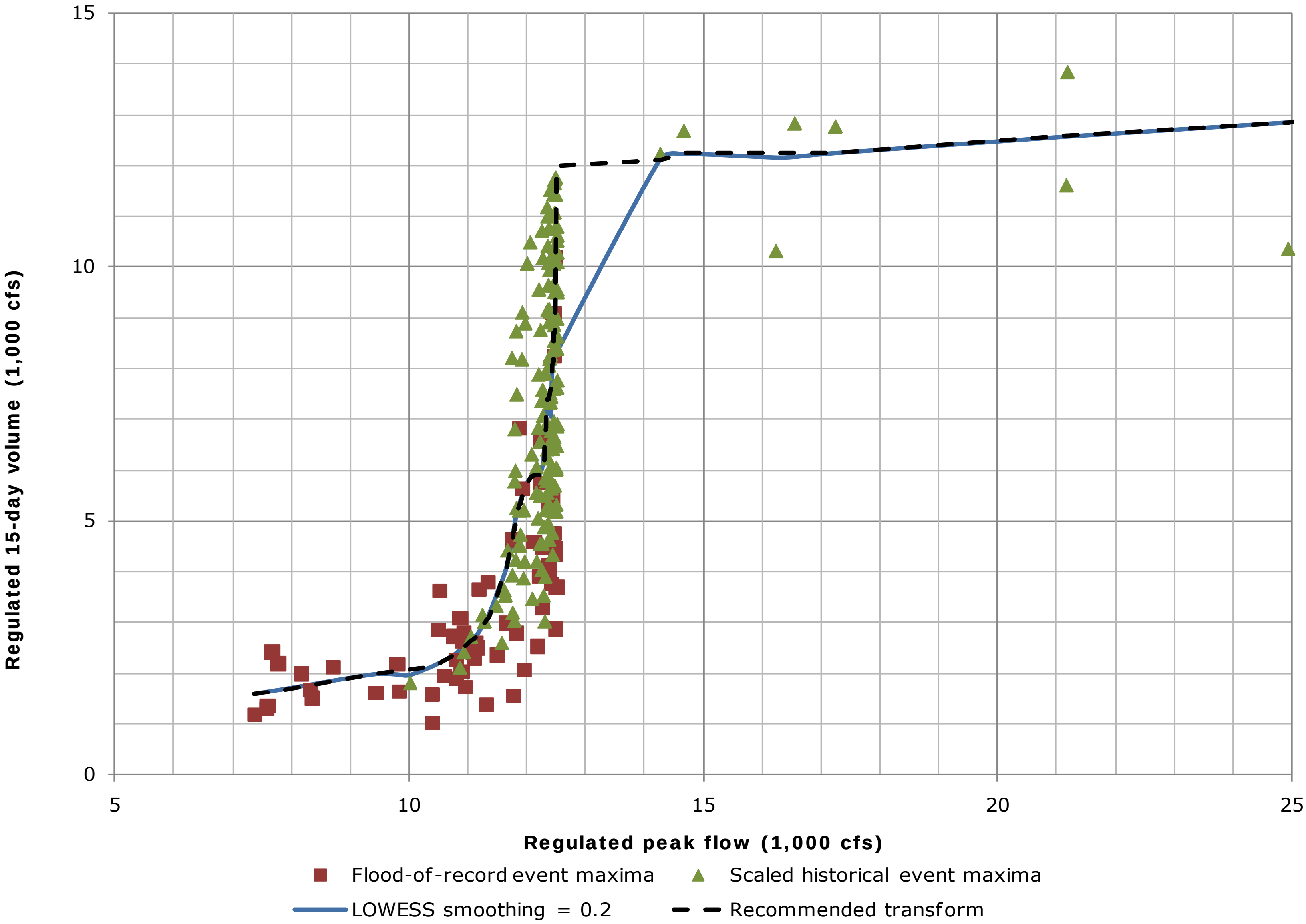


Figure 26. New Hogan Reservoir regulated characteristic curve: 15-day duration

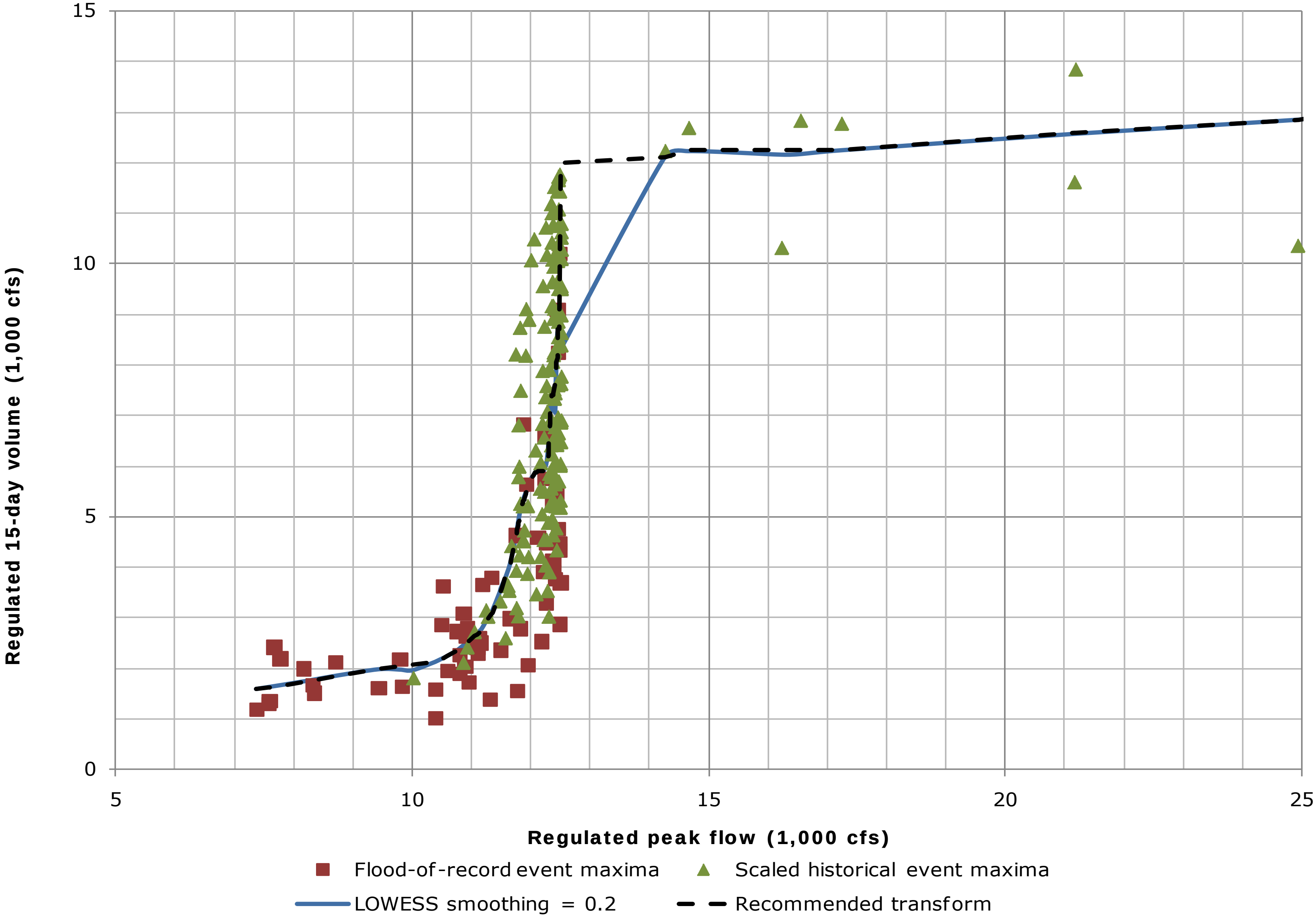


Figure 27. New Hogan Reservoir regulated characteristic curve: 30-day duration

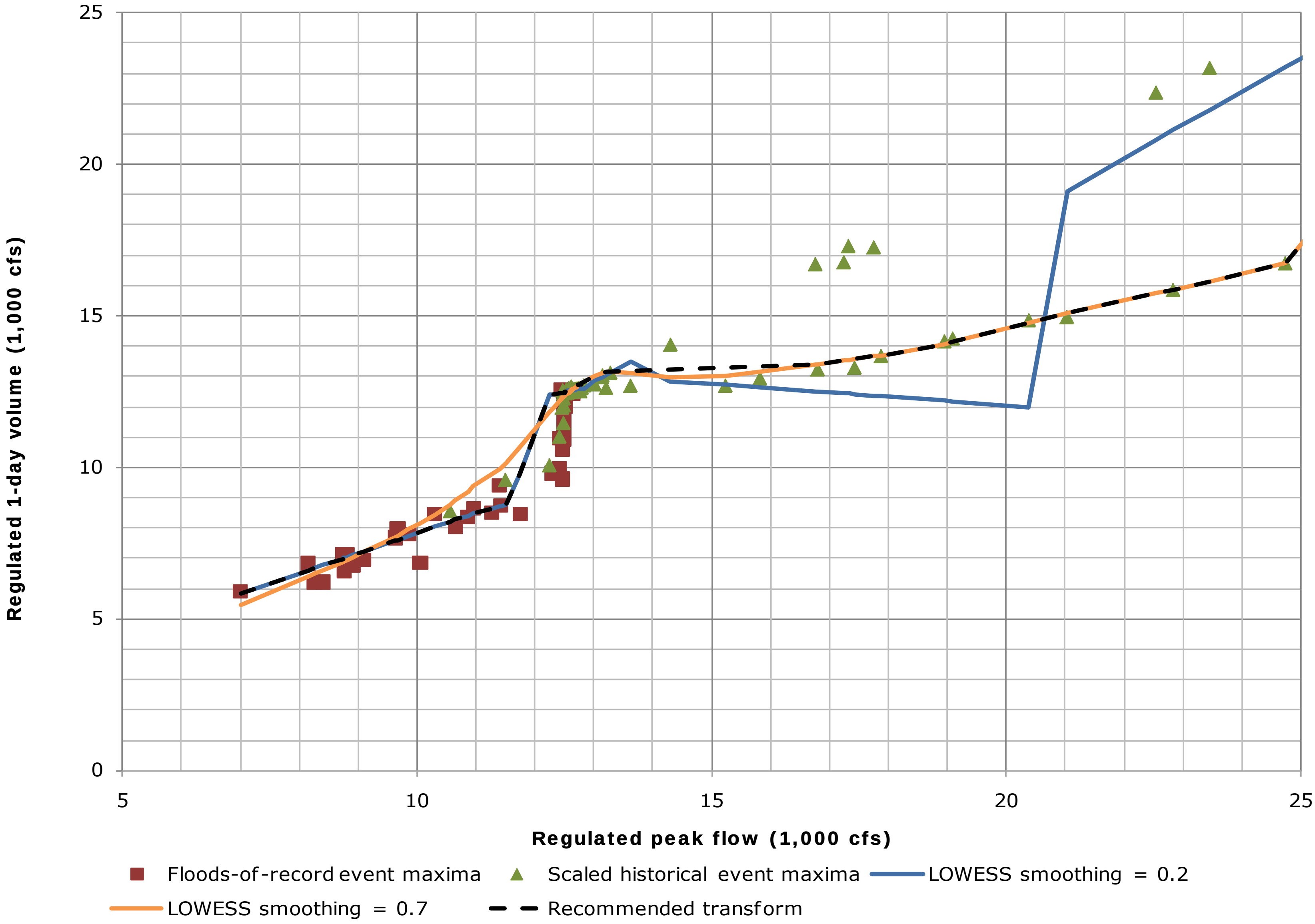


Figure 28. Calaveras River at Bellota regulated characteristic curve: 1-day duration

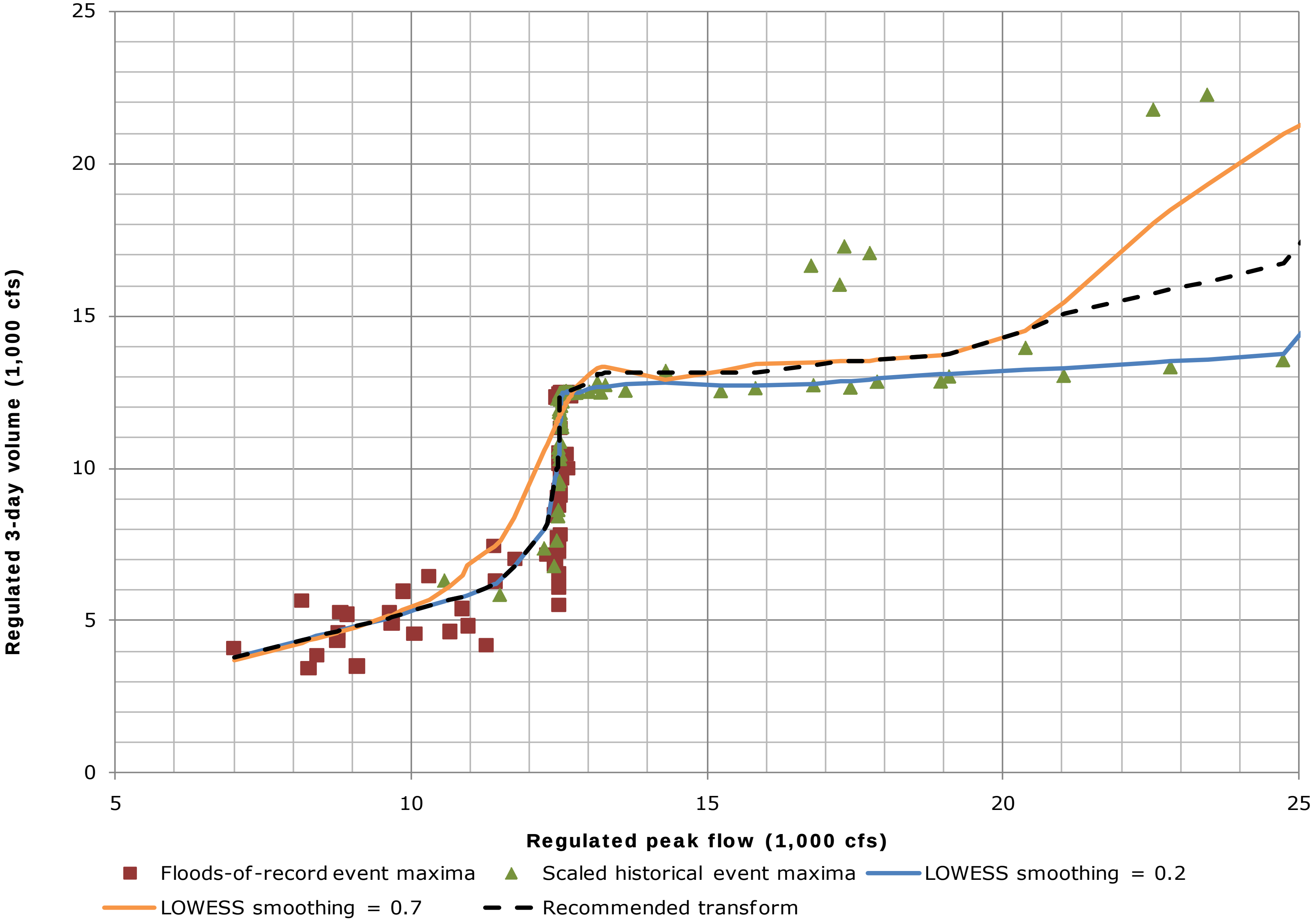


Figure 29. Calaveras River at Bellota regulated characteristic curve: 3-day duration

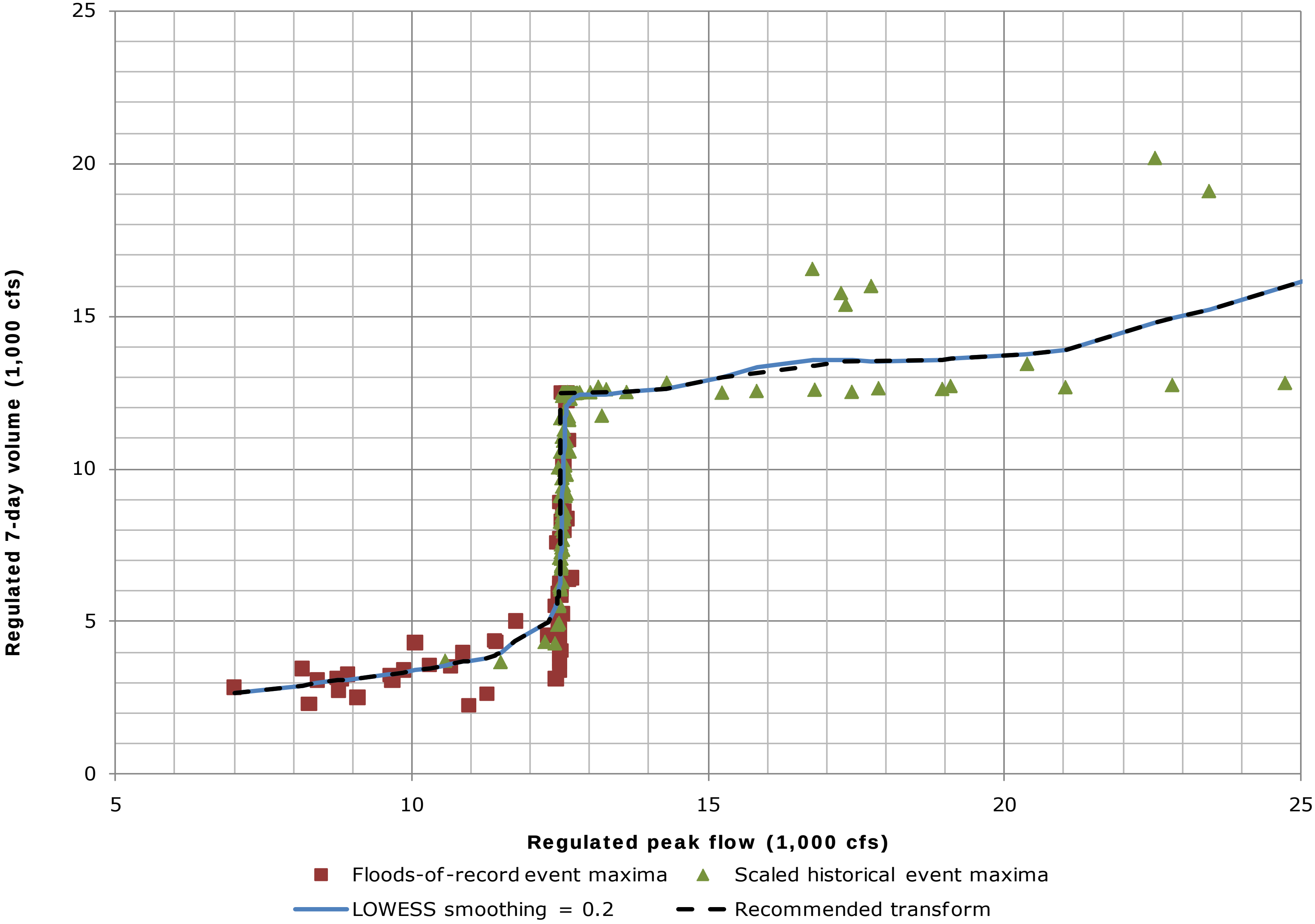


Figure 30. Calaveras River at Bellota regulated characteristic curve: 7-day duration

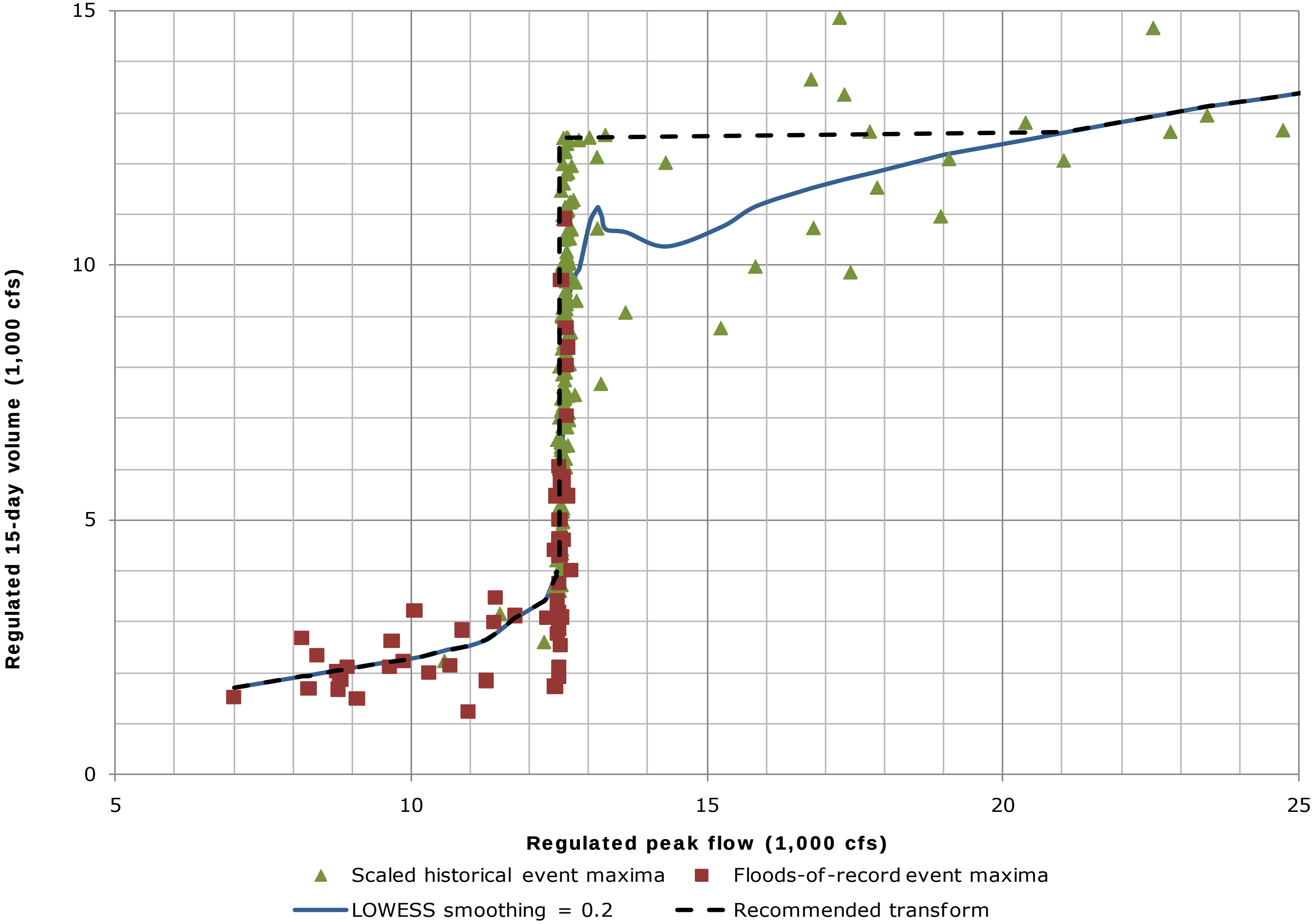


Figure 31. Calaveras River at Bellota regulated characteristic curve: 15-day duration

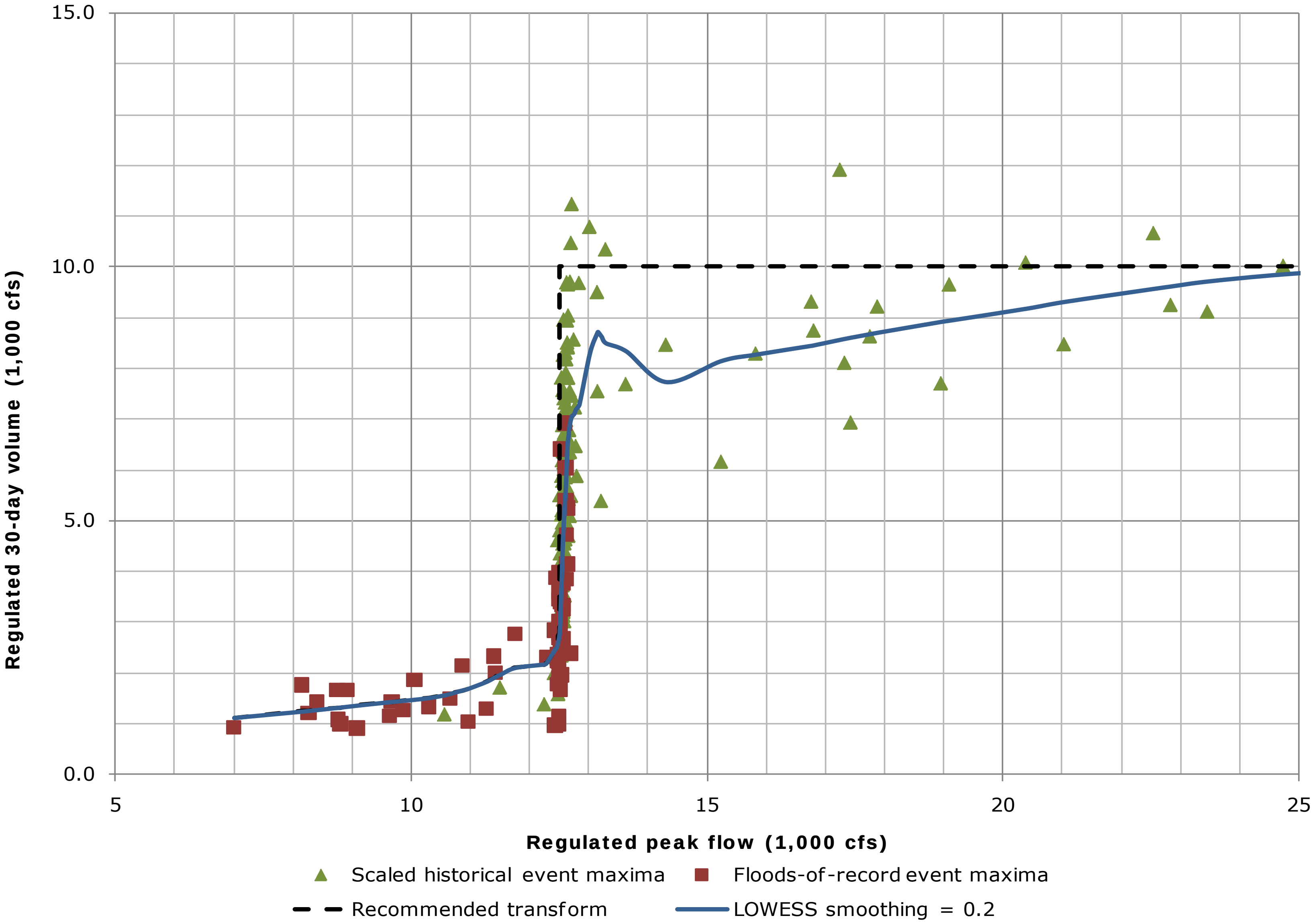


Figure 32. Calaveras River at Bellota regulated characteristic curve: 30-day duration

Attachment 7: Quality control certification

David Ford Consulting Engineers, Inc. completed Task 3, development of flow frequency curves, expected hydrographs, and documentation of procedures for contract W91238-09-D-0004—Lower San Joaquin River Feasibility Study, San Joaquin County, CA including Stockton City and nearby communities.

Notice is hereby given that all quality control activities of the technical memorandum prepared by the firm have been completed, appropriate to the level of risk and complexity inherent in the project, as defined in the Quality Control Plan. Compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This includes review of assumptions; methods, procedures, and material used in the analyses; the appropriateness of data used and level of data obtained; and reasonableness of the results, including whether the product is consistent with law and existing Corps policy.

REGISTERED PROFESSIONAL ENGINEER
DAVID THURMAN FORD
No. 33889
Exp. 6/30/2012
CIVIL
STATE OF CALIFORNIA

3/25/2011

David T. Ford, PhD, PE, D.WRE
President
David Ford Consulting Engineers, Inc.

(date)

Appendix 1- Attachment 2

Lower San Joaquin Feasibility Study Alternative Analysis for Calaveras River at New Hogan Dam

David Ford Consulting Engineers, Inc.

2015 J Street, Suite 200
Sacramento, CA 95811

Ph. 916.447.8779
Fx. 916.447.8780

MEMORANDUM

To: John High and Steve Holmstrom, PE

From: Nathan Pingel, PE; Teresa Bowen, PE; and Michael Konieczki, PE

Date: August 12, 2011

Subject: Contract W91238-09-D-0004-0004 modification 2: Lower San Joaquin River feasibility study, San Joaquin County, CA, including Stockton City and nearby communities

Deliverable for task 6 and option task 1: Use existing New Hogan Dam HEC-ResSim model to evaluate re-operation alternatives to achieve 200-yr protection downstream and investigate the impact of downstream channel improvements to achieve 200-year protection downstream

Situation

In support of the lower San Joaquin River feasibility study (LSJR FS), we completed a hydrologic analysis of the Calaveras River, specifically focusing on analysis points at New Hogan Reservoir and Bellota. The results of this analysis are described in our June 20, 2011, report, *Lower San Joaquin River feasibility study: Calaveras River frequency analysis and hydrographs.* In that report, we presented unregulated flow-frequency curves and unregulated to regulated flow transforms for the analysis points noted above. Using these 2 products, we also presented regulated peak flow-frequency curves at the analysis point locations. This work was completed for the Sacramento District of the US Army Corps of Engineers (Corps).

Table 1 summarizes the peak regulated flow at 2 locations along the Calaveras River—immediately below New Hogan Dam, and at the downstream reservoir operating point of Bellota—for 2 events: the p=0.005 event and the p=0.01 event.

The reservoir is operated to limit the flow at Bellota to 12,500 cfs, unless a larger release is required by the reservoir operation rules or the available flood storage in the dam is exhausted. The downstream peak flow at Bellota is a function both of reservoir releases and local uncontrolled flow from the watershed area between New Hogan and Bellota.

As part of the LSJR FS, the Corps and the local sponsor, the San Joaquin Area Flood Control Agency (SJAFCA), are evaluating alternative flood risk reduction measures that will provide greater flood protection. The focus of these alternatives is to protect downstream areas from flooding from events more common than the p=0.005 event.

Table 1. Peak regulated flow for selected annual exceedence probabilities[1]

Annual exceedence probability (1)	**Peak regulated flow below New Hogan Dam (cfs) (2)**	**Peak regulated flow at Bellota (cfs) (3)**
0.01	12,367	13,634
0.005	12,903	16,409

Notes:
1. Values are as reported in the June 2011 report *Lower San Joaquin River feasibility study: Calaveras River frequency analysis and hydrographs*

Tasks

Our task is to use the baseline hydrologic analysis as documented in our June 2011 report and evaluate 2 alternatives:

1. Modifications to New Hogan Reservoir to reduce p=0.005 peak flows downstream to 12,500 cfs.
2. Modifications to the downstream channel capacity to contain the p=0.005 peak flows. (Alternative channel capacities under consideration by the project team include increases from 12,500 cfs to 15,000 cfs, 18,000 cfs, or 21,000 cfs.)

This evaluation is from a hydrologic perspective only and to support initial alternative screening. This evaluation does not include the assessment of risk reduction, as measured with reduction in expected annual damage, nor does it include an explicit consideration of uncertainty in of the assessments of "level of protection" or ability of the system to pass or control an event of specified probability.

Actions

To evaluate the 2 alternatives above, we:

1. Prepared an exposition of the reservoir simulation results for selected events from our June 2011 report, which allowed us to elaborate specifically on whether downstream channel capacity was exceeded, and if so, why. Doing so allows us to focus on the predominant factors influencing flooding downstream of New Hogan:
    - The inflow to New Hogan Reservoir.
    - The local uncontrolled flow between New Hogan and Bellota.
    - The use of the flood storage in New Hogan Reservoir.
    - The rate-of-change reservoir operating rule and the emergency spillway release diagram (ESRD) minimum releases.

    The events selected for this exposition are those that have peak flows approximately equal to the p=0.005 flow at Bellota. These events are described in Attachment A.
2. Evaluated the coincident probabilities of New Hogan Reservoir inflows to probability of local uncontrolled flow. Like the exposition of the reservoir simulation results, the evaluation of coincident probabilities informs the assessment of alternative measures that could reduce the downstream

regulated peak flow. In addition, this evaluation provides guidance for critical storm centering for the rainfall-runoff portion of the overall LSJR FS.

For this evaluation, we completed a flow-frequency analysis on the local flow time series used in the baseline analysis. Using that limited-use local flow-frequency curve and the events described in step 1, we assessed the coincident probabilities between the reservoir inflow and the local flow hydrographs. [The flow-frequency curve developed for this step is intended only for this purpose and not intended to be adopted as a study product, thus referred to as a "limited-use local flow-frequency curve." The study product is being developed through rainfall-runoff model simulations of design storms.]

This analysis is described in Attachment B.

3. Developed and evaluated design events to assess further the sensitivity of reservoir storage and uncontrolled local flows to the peak regulated flow at Bellota. Design events (or hydrographs) are historical events scaled to a specific peak and/or volume(s) of specified probability.

   We developed design events focused at p=0.005 flow at New Hogan and Bellota. These design events are based on historical events and scaled using consistent methodology as in the baseline analysis. We also developed and simulated design (scaled) events for the p=0.01 and p=0.002 flows at both locations.

   This analysis is described in Attachment C.

4. Evaluated the impact of increased flood control storage in New Hogan Reservoir using selected events from our June 2011 report and the results from the actions noted above. Specifically, we focused here on whether increased flood control storage could reduce peak flows at Bellota. These selected events are the same as in step 1 above.

   The analysis plan is included in Attachment D and the analysis is described in Attachment E.

5. Evaluated the impact of increased channel capacity between New Hogan Reservoir and Bellota. This increased channel capacity allows for conveyance of both uncontrolled local flows and reservoir releases.

   This is described in Attachment F.

Findings

From the analysis described above and review of the baseline hydrologic analysis, we found:

- Peak regulated flows at Bellota are a result of both the uncontrolled local flow between New Hogan and Bellota and New Hogan Reservoir releases. New Hogan releases are determined by the prescribed flood control storage, the reservoir inflow, and the dictated reservoir operation rules. So, capacity exceedence at downstream locations may be caused by excessive local flow, excessive reservoir release, or both.
- The probability of the reservoir inflow and the coincident local flow varies by event. A predictable relationship does not exist. For some historical

events, the local flow is rarer than the reservoir inflows. And for others, the opposite is true.

- The p=0.005 3-day volume from the New Hogan frequency curve is less than the dedicated flood storage at New Hogan Reservoir.
- The p=0.005 4-day volume from the New Hogan frequency curve is greater than the dedicated flood storage at New Hogan Reservoir. For actual simulations of design (scaled) events, which include reservoir releases, the total required stored volume does not exceed the dedicated storage for the 1958, 1986, 1997, and 2006 design pattern events. [For the 1998 design pattern event, the stored volume exceeds the dedicated flood storage. However, to scale the 1998 event to the design criterion requires a scaling factor larger than that recommended in Corps' EM 1110-2-1415 (USACE 1993).]
- For the evaluation of selected events, in most cases, the local flow alone exceeded the downstream channel capacity. For the event where local flow did not exceed the channel capacity, the 1958 event scaled by 1.4, a minimum of 14,160 ac-ft of additional storage is needed to maintain a flow at Bellota below 12,500 cfs.

Results

Based on our findings, additional storage alone in New Hogan Reservoir will not reduce the p=0.005 event flow to less than or equal to 12,500 cfs at Bellota. Increased storage may reduce the regulated peak flow-frequency curve, but it will not lower it below the peak local flow-frequency curve for the watershed area between the dam and Bellota.

To "contain" the p=0.005 flow, increased channel capacity is required. As a minimum, for the current watershed condition, the increased channel capacity would need to be equal to or greater than the peak p=0.005 local flow from the watershed area between the dam and Bellota. An alternative to increased channel capacity would be to reduce the peak local flow-frequency curve.

The limited-use peak local flow-frequency curve presented herein is for this analysis only. As a part of the LSJR FS, a separate effort is being completed to develop a local flow-frequency curve using rainfall-runoff models and design storms. The results of that analysis were not available for use here. Once that analysis is completed and adopted, the impact of a revised peak local flow-frequency curve to the conclusions presented herein should be considered.

Guide to attachments

As described above, the attachments summarize the analysis completed to answer the questions posed. Below is the table of contents for these attachments:

References

Cohn, Tim. (2007). PeakfqSA, version 0.937 [Software]. <http://www.timcohn.com/TAC_Software/PeakfqSA/>.

Interagency Advisory Committee on Water Data (IACWD). (1982). *Guidelines for determining flood flow frequency, Bulletin 17B.* US Geological Survey, Reston, VA.

US Army Corps of Engineers (USACE). (2010). *Cosgrove Creek, California Section 205 flood risk management study: Existing conditions hydrology*, Sacramento District, Sacramento, CA.

USACE. (2007). *HEC-ResSim Reservoir System Simulation, version 3.0. CPD-82,* Institute for Water Resources, HEC, Davis, CA.

USACE. (2004). *Farmington Dam and Reservoir, Littlejohn Creek, California, Water control manual, Appendix IV to Master water control manual, San Joaquin River Basin, California*, Sacramento District, Sacramento, CA.

USACE. (2001). *Sacramento and San Joaquin river basins comprehensive study.* Sacramento District, Sacramento CA.

USACE. (1993). *EM 1110-2-1415, Hydrologic frequency analysis*, Washington, D.C.

USACE. (1983). *New Hogan Dam and Lake, Calaveras River, California, Water control manual, Appendix III to Master water control manual, San Joaquin River Basin, California,* Sacramento District, Sacramento, CA.

US Geological Survey (USGS). (2011). *Regional skew for California, and flood frequency for selected sites in the Sacramento–San Joaquin River Basin, based on data through water year 2006: U.S. Geological Survey Scientific Investigations Report 2010–5260*, USGS, Sacramento, CA.

USGS. (forthcoming). Regional Skews for 1-Day, 3-Day, 7-Day, 15-Day, and 30-Day Duration Discharge for the Central Valley Region of California, USGS, Sacramento, CA.

Attachment A. Exposition of selected reservoir simulations from baseline analysis

Overview

For the analysis in our June 2011 report, we routed 60 historical events and 190 scaled-versions of the historical events (19 events times 10 scale factors each) through the reservoir simulation model. Computer program HEC-ResSim was used to develop the New Hogan Reservoir model and to complete the simulations. In that report, the results of the simulations were summarized in the unregulated to regulated flow and regulated flow to regulated volume transform plots; each point in the figures represented a reservoir simulation of a historical or scaled historical event.

As a part of this current analysis, and to support ongoing discussions of the baseline analysis described in the June 2011 report, we include here an exposition of a subset of the reservoir simulations completed.

Selection of events

We selected 8 events used in the baseline hydrologic analysis that represent approximately a p=0.005 regulated peak flow at Bellota. (An event with a regulated peak flow at Bellota equal to the p=0.005 event does not necessarily correspond to an event with a New Hogan Reservoir inflow equal to the p=0.005 event.) The regulated peak flow at Bellota for the p=0.005 event is 16,407 cfs, per the June 2011 report. Selected events are shown in Table 2. Column 1 of Table 2 notes the selected historical event; the associated start and end dates are listed in columns 3 and 4. Column 2 notes if the event was a scaled version of this historical event or not; the value indicates the factor that was used to scale uniformly the historical event. For reference, column 5 notes the peak regulated flow at Bellota from the reservoir operation simulations and column 6 indicates the peak local flow used as input for the simulation. In the following section, the reservoir simulations are further described in graphical form.

For reference, Figure 1 shows the Bellota unregulated to regulated flow transform from the June 2011 analysis with these selected events labeled. For the development of that transform, the 2006 event was not included, but has been added to the figure for reference purposes.

Reservoir operation simulation for selected events

Reservoir simulation routings for each of the events listed in Table 2 are shown in Figure 2 through Figure 9. For each figure, we include a plot showing the water surface elevation at New Hogan, inflow, outflow, local flow between New Hogan and Bellota, unregulated flow at Bellota (flow that would have occurred with no upstream reservoir), and regulated flow at Bellota (local flow plus reservoir releases).

Critique of simulations and events

Table 3 summarizes the selected event simulations. In column 3 of Table 3 we note whether or not the downstream channel capacity of 12,500 cfs was exceeded. If it was, we note in column 4 the prominent factor from the simulation that caused that to occur. In column 5 we provide notes about mitigation alternative(s) (additional flood storage, revision to the ESRD, or lowering the flood pool) that may be considered to lessen the peak flow downstream. And, in column 6, we note what the resulting downstream peak flow for that event could be with those mitigation alternative(s) in place. This list of alternatives is for planning purposes only and is not the result of a full alternative analysis.

Table 2. Selected historical and scaled historical events

Event (1)	**Scale factor (2)**	**Start date (3)**	**End date (4)**	**Peak regulated flow at Bellota[1] (cfs) (5)**	**Peak local flow[1,2] (cfs) (6)**
1907	2.2	Mar 1, 1907	Apr 13, 1907	16,543	13,195
1958	1.4	Mar 10, 1958	Apr 29, 1958	16,759	3,070
1969	3.0	Jan 3, 1969	Mar 1, 1969	12,500[3]	4,777
1986	1.6	Jan 21, 1986	Mar 31, 1986	12,500[3]	9,359
1997	2.2	Dec 1, 1996	Feb 15, 1997	15,822	14,714
1998	1.6	Jan 1, 1998	Mar 15, 1998	15,906	15,098
1999	1.0	Feb 6, 1999	Feb 12, 1999	12,500[3]	5,620
2006	1.6	Mar 24, 2006	Apr 24, 2006	12,500[3]	11,698

Notes:

1. Peak regulated flow and peak local flow values are not necessarily coincident in time.
2. Local flow is the uncontrolled watershed contribution from New Hogan Dam to Bellota.
3. Reservoir releases adjusted to 12,500 cfs from HEC-ResSim computed releases to compensate for known routing issues in the computer program. For these simulations, sufficient flood storage is available for the event.

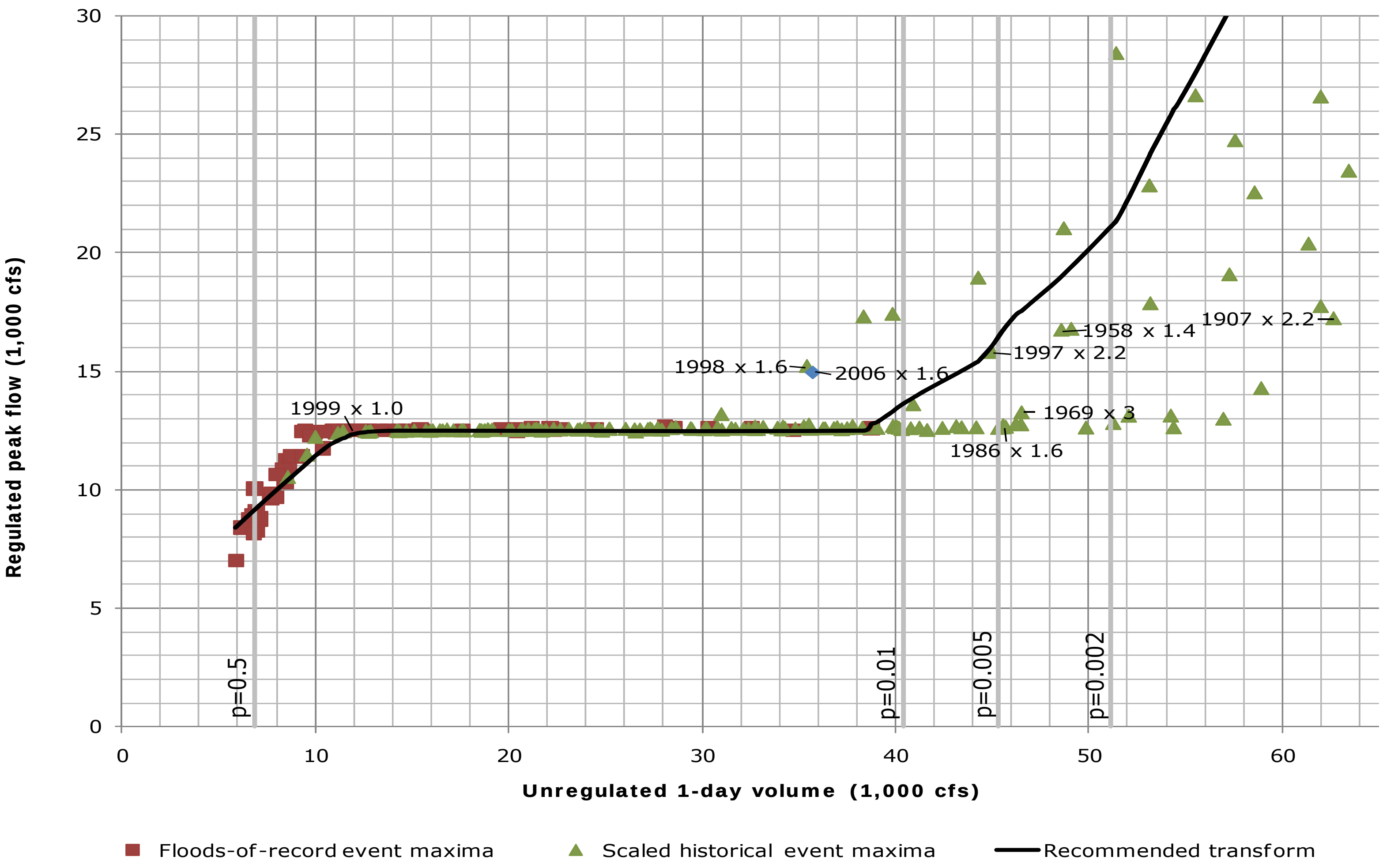


Figure 1. Unregulated to regulated flow transform from June 2011 baseline analysis: Calaveras River at Bellota with highlighted selected events. The 2006 event shown with a blue diamond was not used for flow transform development.

Table 3. Critique of controlling factor for simulations of selected historical and scaled historical events

Event (1)	Scale factor (2)	Channel capacity at Bellota exceeded? (3)	If channel capacity at Bellota is exceeded, why? (4)	Notes about possible New Hogan mitigation alternative(s) (5)	Peak flow at Bellota after modification (cfs) (6)
1907	2.2	Yes	Local flows	Additional flood storage will not keep flow at Bellota < 12,500 cfs	N/A
1958	1.4	Yes	ESRD release	1. Remove or revise ESRD 2. Lower flood pool to 661 ft[2]	12,500[1] 12,500[1]
1969	3.0	No	N/A	—	—
1986	1.6	No	N/A	—	—
1997	2.2	Yes	Local flows	Additional flood storage will not keep flow at Bellota<12,500 cfs	N/A
1998	1.6	Yes	Local flows	Additional flood storage will not keep flow at Bellota<12,500 cfs	N/A
1999	1.0	No	N/A	—	—
2006	1.6	No	N/A	—	12,500[1]

Notes:

1. Reservoir releases adjusted to 12,500 cfs from HEC-ResSim computed releases to compensate for known routing issues in the computer program. For these simulations, sufficient flood storage is available for the event.
2. A lowered flood pool to elevation 661 ft translates to additional flood storage of 14,157 ac-ft.

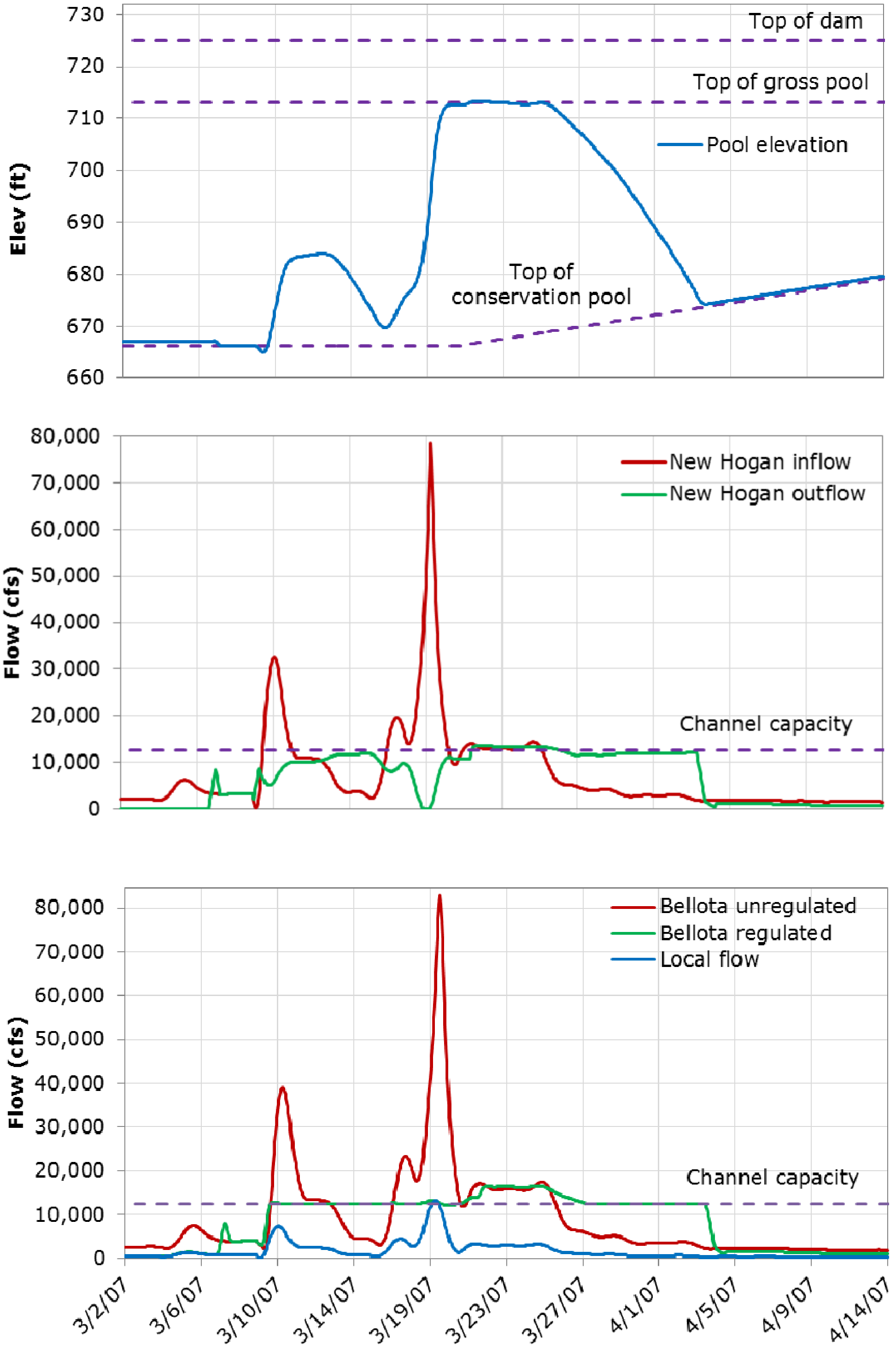


Figure 2. New Hogan routing of 1907 event scaled by 2.2

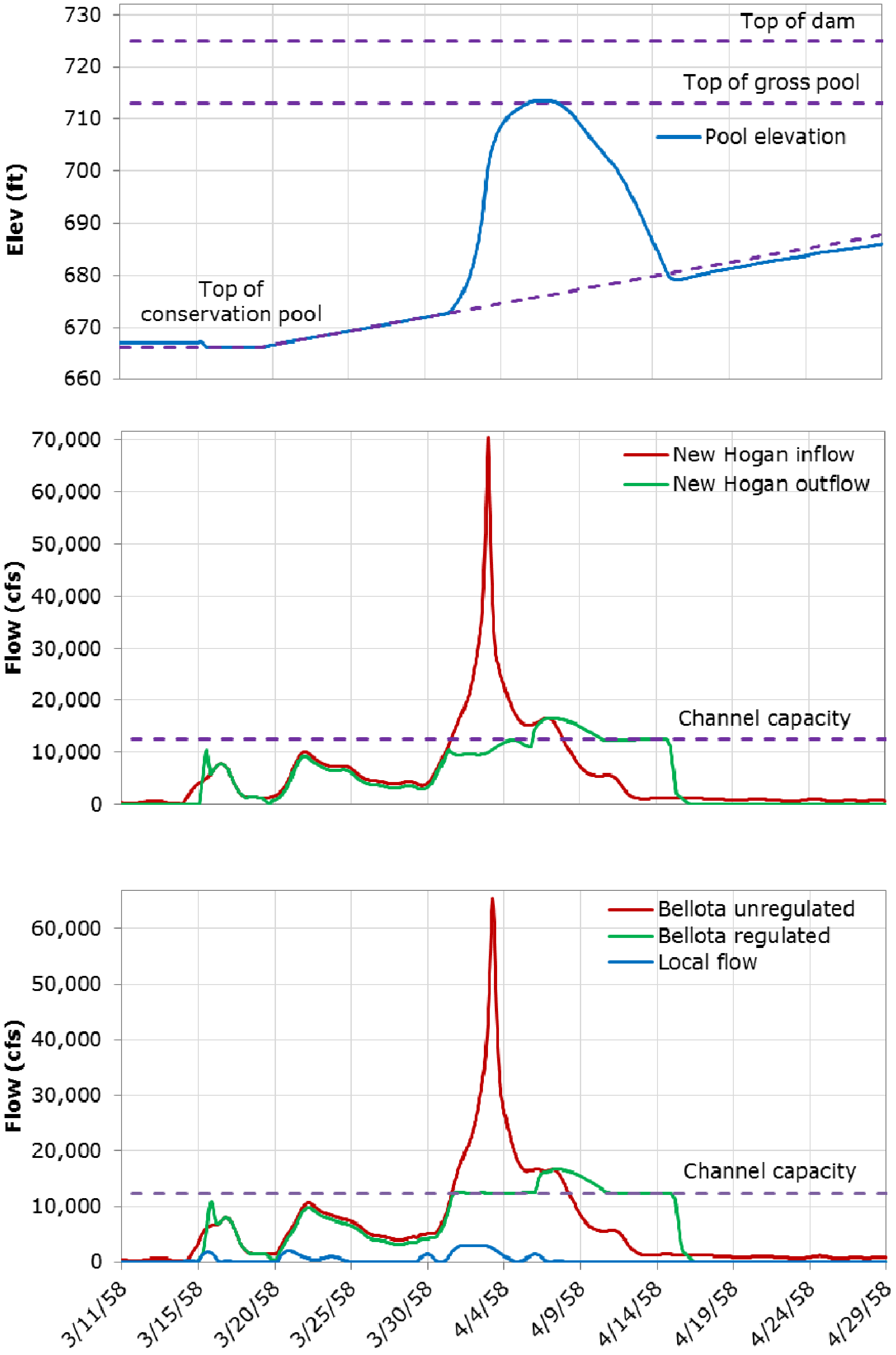


Figure 3. New Hogan Reservoir routing of 1958 event scaled by 1.4

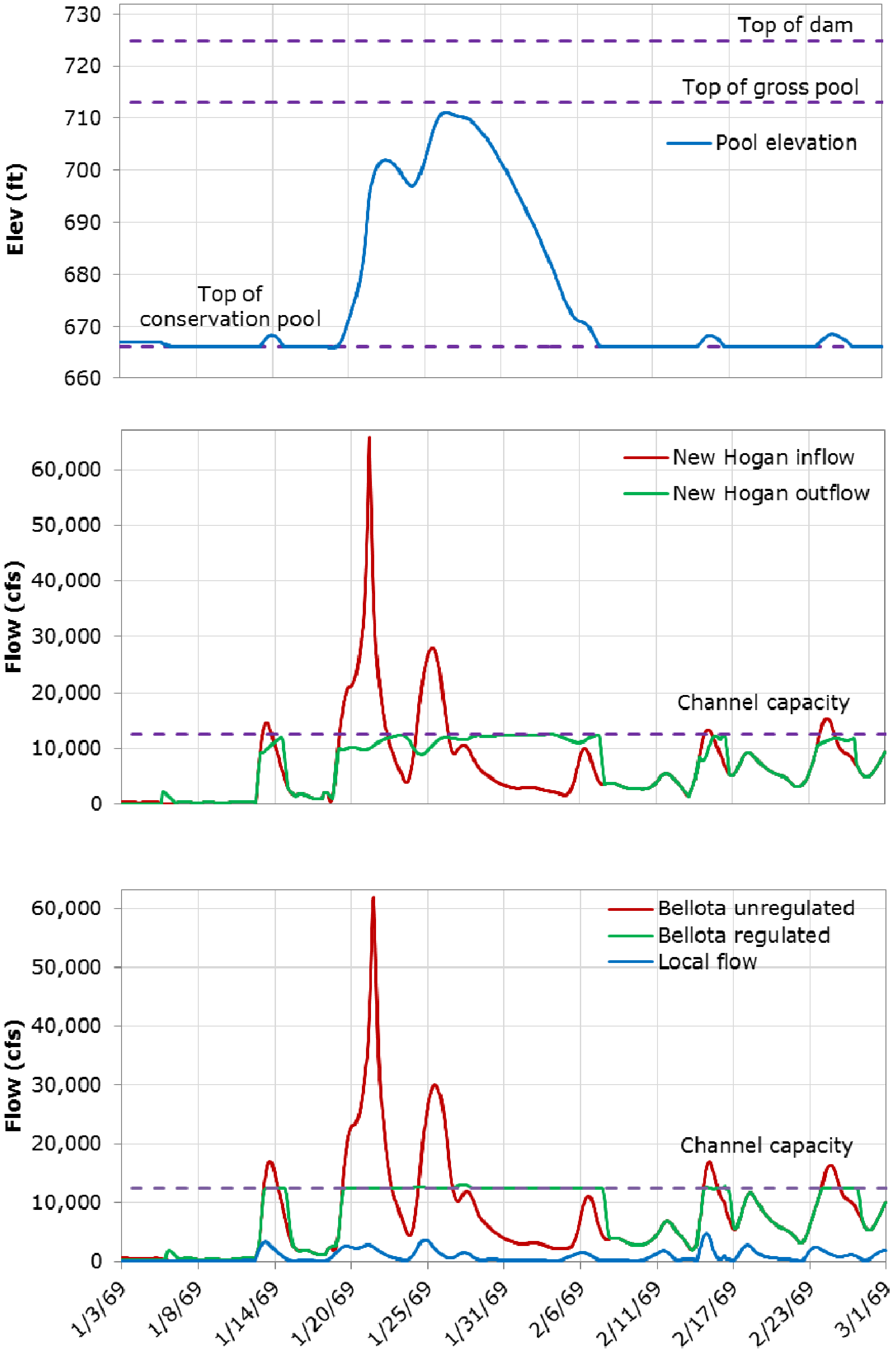


Figure 4. New Hogan Reservoir routing of 1969 event scaled by 3.0

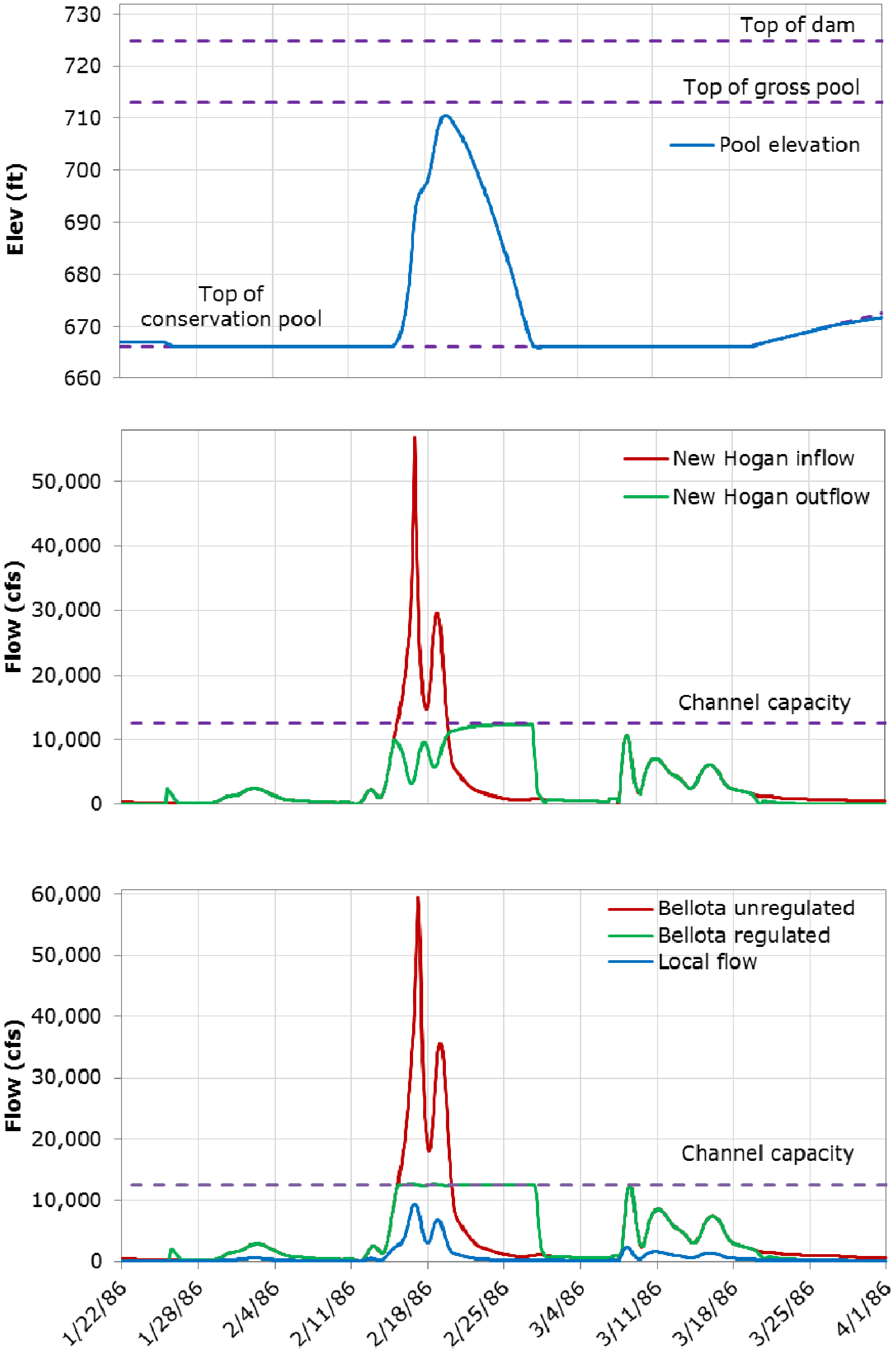


Figure 5. New Hogan Reservoir routing of 1986 event scaled by 1.6

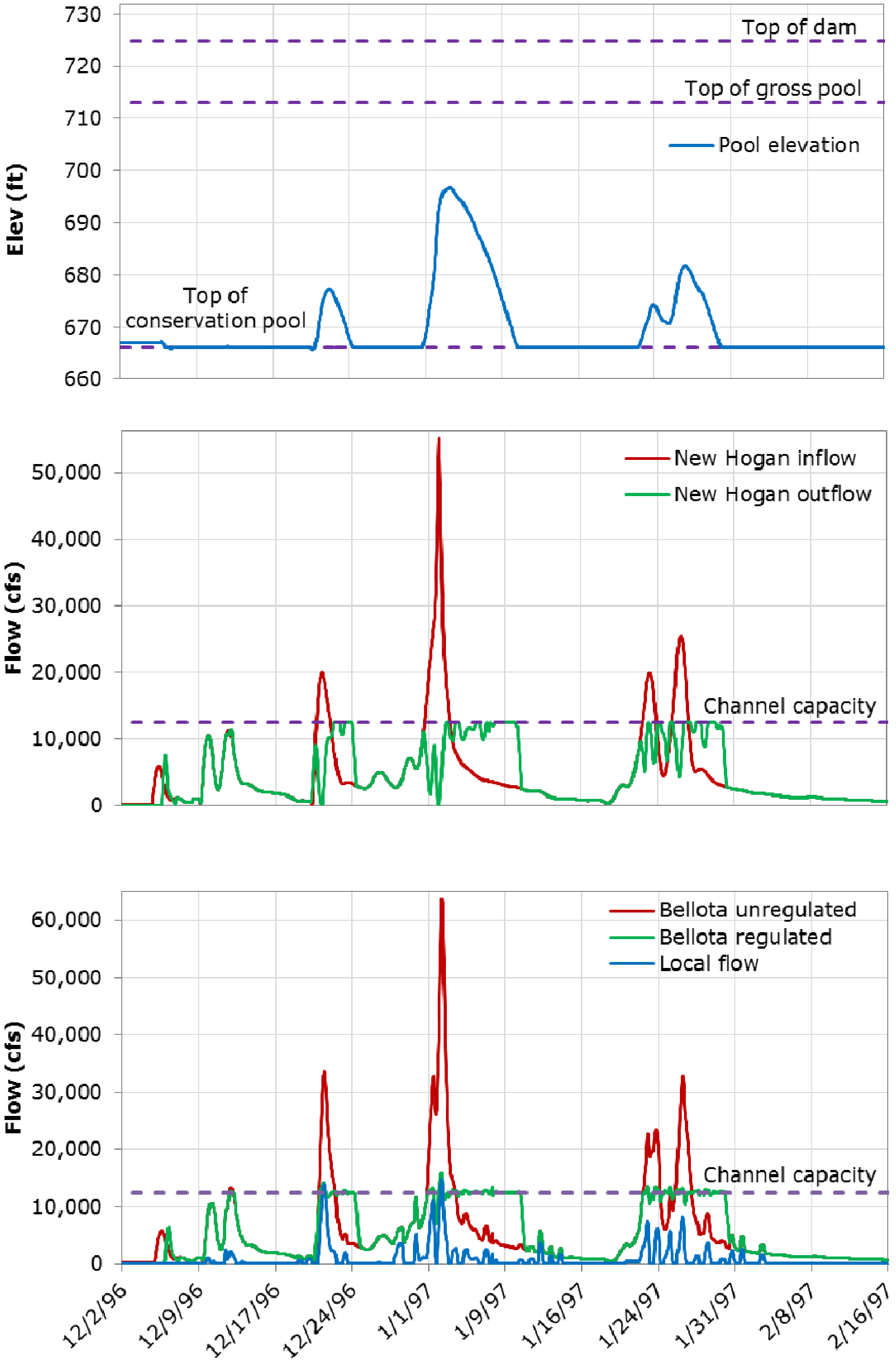


Figure 6. New Hogan Reservoir routing of 1997 event scaled by 2.2

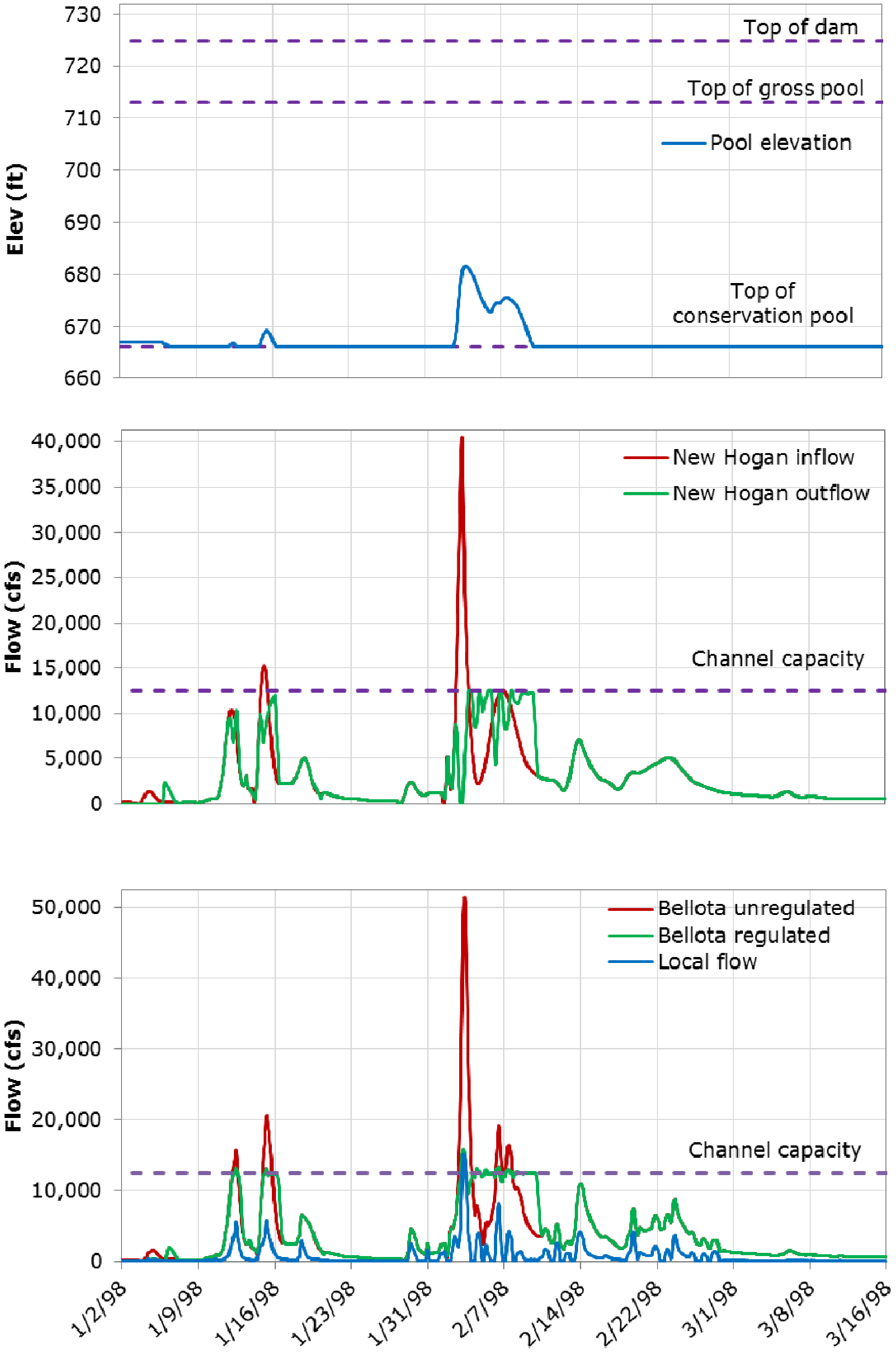


Figure 7. New Hogan Reservoir routing of 1998 event scaled by 1.6

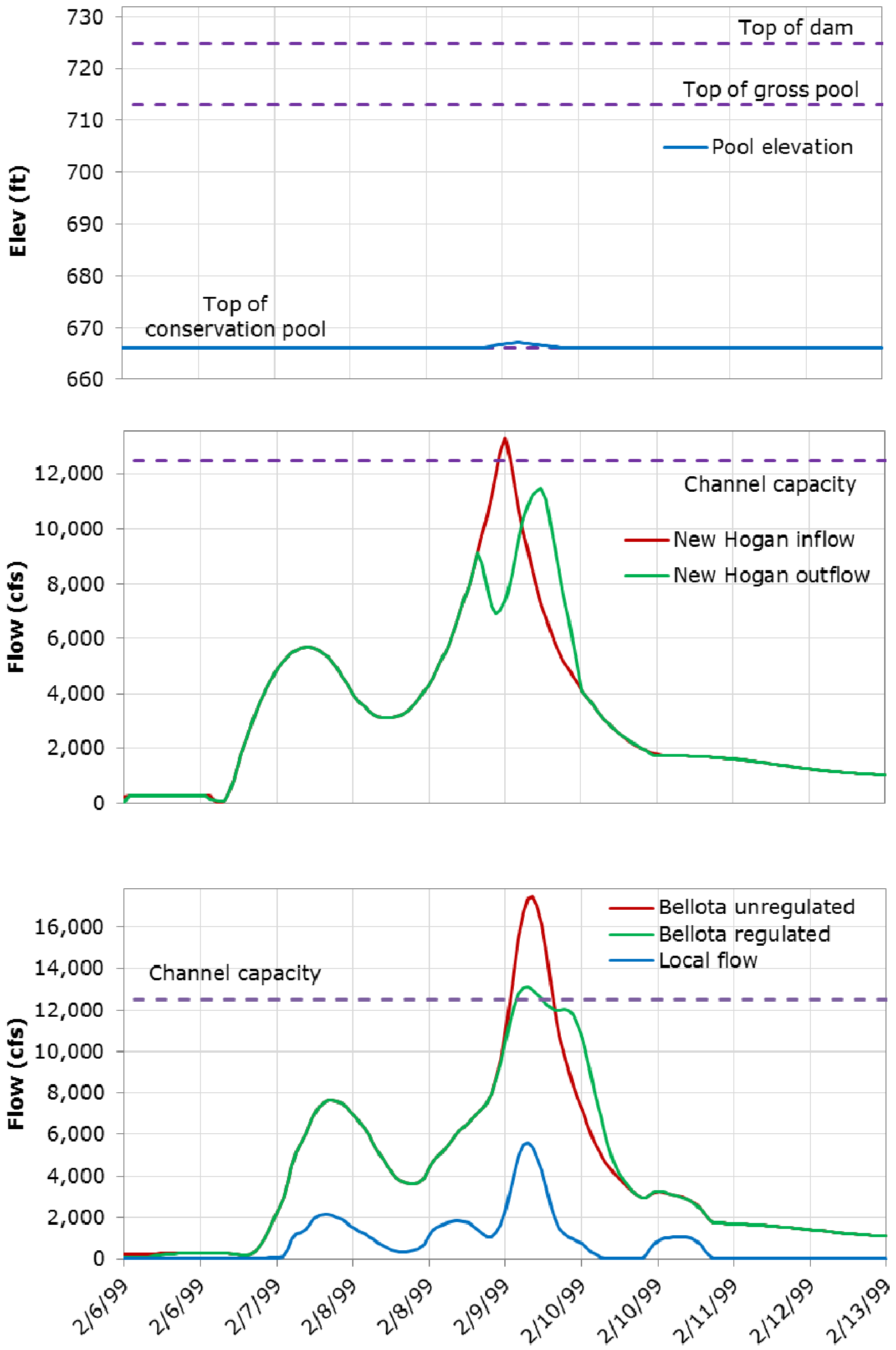


Figure 8. New Hogan Reservoir routing of 1999 event scaled by 1.0

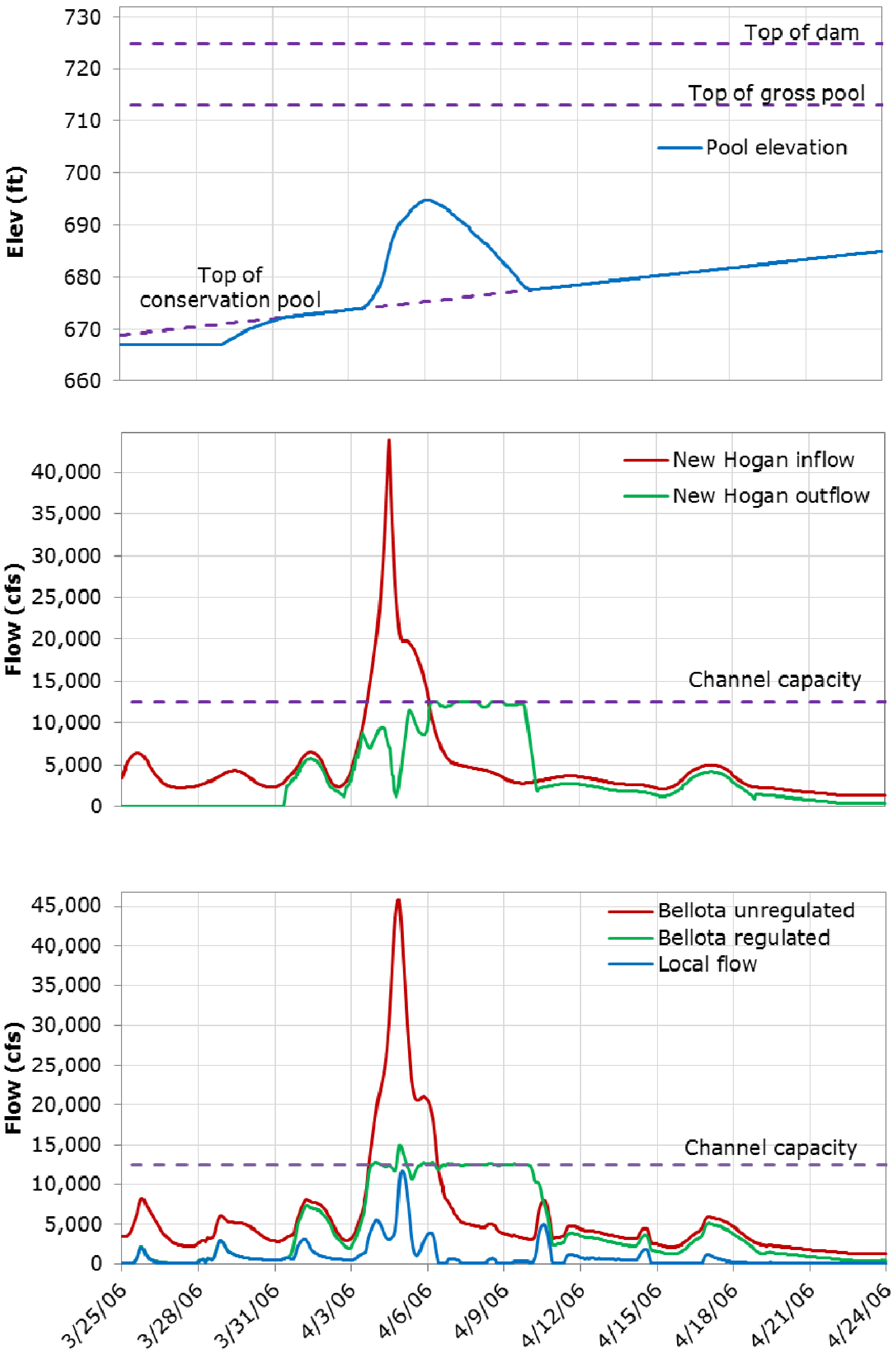


Figure 9. New Hogan Reservoir routing of 2006 event scaled by 1.6

Attachment B. Assessment of coincident reservoir inflow and local flow annual exceedence probabilities

Overview

The unregulated flow-frequency curve is used to predict the flow and volume for the rare events. To transform the unregulated flow and volume to regulated peak flow, we must rely on reservoir simulations of historical or design (scaled) events.

A challenge in this transformation in a regulated system is the distribution of volume above and below the regulating features, in this case a flood control reservoir. Given the variability of historical flood events, typically a predictable or fixed relationship of volume above and below the reservoir does not exist. Thus, this variability must be accounted for in development of the transform.

In the baseline analysis, as documented in the June 2011 report, we followed guidance from EM 1110-2-1415 (USACE 1993), page 3-26:

> *(3) Use of hypothetical-flood routings. Usually recorded values of flows are not large enough to define the upper end of the regulated frequency curve. In such cases, it is usually possible to use one or more large hypothetical floods (whose frequency can be estimated from the frequency curve of unregulated flows) to establish the corresponding magnitude of regulated flows. These floods can be multiples of the largest observed floods or of floods computed from rainfall; but it is best not to multiple any one flood by a factor greater than two or three. The floods are best selected or adjusted to represent about equal severity in terms of runoff frequency of peak and volumes for various durations. The routings should be made under reasonably conservative assumptions as to initial reservoir stages.*

Also of note in the EM regarding local flows is the following:

> *(5) Runoff from unregulated areas. In estimating the frequency of runoff at a location that is a considerable distance downstream from one or more reservoir projects, it must be recognized that none of the runoff from the intermediate areas between the reservoir(s) and the damage center will be regulated. This factor can be accounted for by constructing a frequency curve of the runoff from the intermediate area, and using this curve as an indicator of the lower limit for the curve of regulated flows. Streamflow routing and combing of both the flows from the unregulated area and those from the regulated area is the best procedure for deriving the regulated frequency curve.*

Here, we evaluate the coincidence of events, as related to local flows and reservoir inflows, of the same or similar probabilities for the historical and scaled historical events used in the baseline analysis. To do so, we first construct a local flow-frequency curve and then pair the probability of local flows and reservoir inflows from historical events. Then, we use these figures to assess the relationship with respect to annual exceedence probabilities of local flows and reservoir inflows. This comparison is made for both historical events and scaled historical events that are used in the unregulated to regulated flow transform development.

Local flow-frequency curve development

The peak local flow-frequency curve developed and presented here is for the purpose of assessing the coincidence, with respect to annual exceedence probabilities, of local flows and reservoir inflows. We developed these curves as a comparison tool only and is referred to as the "limited-use local flow-frequency curve." Currently, rainfall-runoff analyses with design precipitation events are being completed to support the development and adoption of a local flow-frequency curve on the Calaveras River for use in the LSJR FS.

The local flow area we are referring to here is the area downstream of New Hogan Reservoir and upstream of Bellota along the Calaveras River. The area is approximately 110 sq. mile and is illustrated in Figure 8 from the June 2011 report.

To develop the peak local flow flow-frequency curve, we:

1. Identified the local peak flow annual maximum series. For this, we used only the peak flows directly computed from observed data. For the peak series of annual maximums, this includes those events where local flows were developed using Option 1 (as defined in section "Unregulated flow time series" of the June 2011 report) and hourly flows (as defined in Table 5 of the June 2011 report). Thus, this includes 14 annual peaks from the 1996 through 2009 water years. The annual maximum series is listed in Table 4.

   [Peak local flows for the other historical events are from the data series smoothing as described in section "Regulated flow time series development," subsection "Smooth unregulated flow time series" of the June 2011 report. These synthetic peaks are not used here for frequency analysis.]

2. Consistent with Corps policy and the standard of practice, fit a Pearson III (LPIII) distribution to the logarithmic transforms of annual maximum series identified from directly calculated local flow data following procedures specified in *Bulletin 17B* (IACWD 1982). We fit the curve using the expected moment algorithm (EMA) enabled flow-frequency software PeakfqSA, version 0.937. This was developed by Tim Cohn of the USGS and is based on the USGS's flow-frequency software PeakFQ (Cohn 2007).For this statistical analysis, we developed and used regional skew values using the relationships developed by the USGS (USGS 2011).

The resulting curve is shown in Figure 10 and selected flow quantiles are tabulated in Table 5.

For this analysis, we used only the directly calculated local flows because those are the values appropriate for peak flow-frequency analysis. Use of the local flows values estimated using Option 2 or Option 3, as described in the June 2011 report, are not appropriate here because:

1. These flows were calculated on a daily basis (as detailed in Table 5 of the June 2011 report) and do not have observations of peak flows.
2. These flows are based on regression analysis where the values are scaled by a factor of approximately 4 or 5 (where the values are estimated as a

function of Cosgrove Creek or reservoir inflow.) The regression analyses used in the June 2011 report were in the context of estimating this local flow contribution to the total watershed runoff volume. Thus, the portion of flow estimated was small in comparison to the total.

As a check of the limited-use local flow-frequency curve, we:

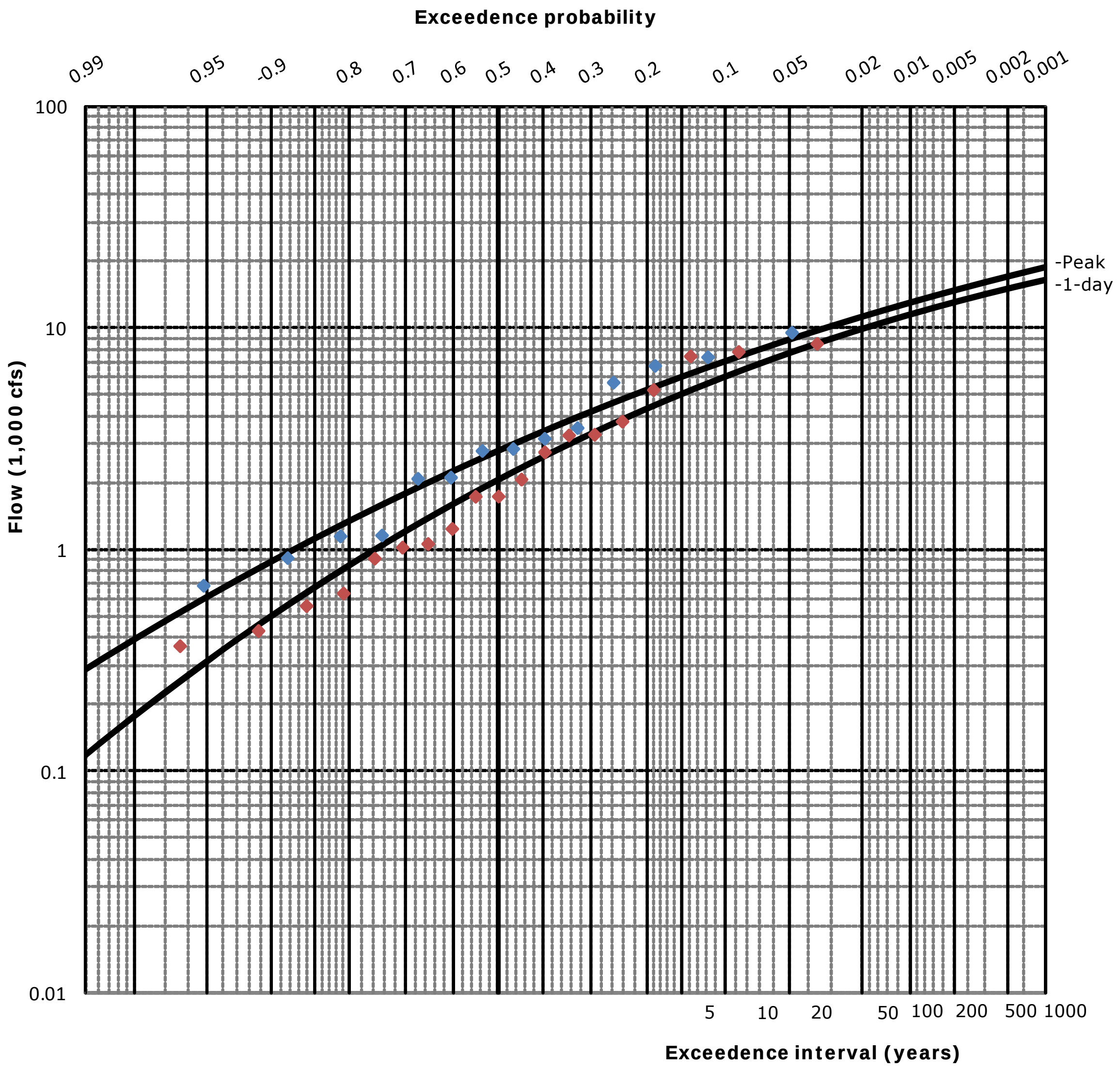
- Fit an unregulated volume-frequency curve to the 1-day local flow volumes following guidance provided by EM 1110-2-1415 (USACE 1993). Here, we used values calculated using Option 1 including those calculated on a daily basis (as defined in Table 5 of the June 2011 report), for a total of 19 annual maximums. This annual maximum series is listed in Table 4. For this statistical analysis, consistent with Corps policy and the standard of practice, fit a Pearson III (LPIII) distribution to the logarithmic transforms of annual maximum series again using PeakfqSA, version 0.937. We used a 1-day regional skew developed using the USGS relationships (USGS forthcoming). In addition, we treated the 19 1-day volumes as a systematic record, assuming no gaps or missing values. The resulting curve is shown in Figure 10 and selected flow quantiles are tabulated in Table 5. We compared the shape of the volume-frequency curve to the peak flow-frequency curve.
- Compared the results to the Cosgrove Creek peak flow-frequency curve from the recent Cosgrove Creek hydrology study (USACE 2010) multiplied by 3.2. This is the factor that is used to estimate the local flow as a function of Cosgrove Creek, Option 2 as described in the June 2011 report. We found the curves to be similar, but the scaled version of the Cosgrove Creek curve to be slightly higher. For the p=0.01 flow the scaled curve was 6% higher and for the p=0.005 flow the scaled curve was 4% higher than the peak curve developed here. (For reference, the Corps' Cosgrove Creek peak flow-frequency curve has the following properties: mean is 2.974, standard deviation is 0.3519, and adopted skew is -0.6.)
- Compared the peak flow and 1-day volume frequency curves on regional flow-per-square-mile estimates. For this, we used frequency curves from the Comp Study (USACE 2001) and latest study on Cosgrove Creek (USACE 2010) for all locations other than those on the Calaveras River and Littlejohn Creek. For these latter locations, we used the results from our June 2011 analyses. Specifically, we: (1) divided the peak, 1-day, and 3-day flow quantiles of 8 nearby watersheds by their associated watershed area, and (2) plotted these values as a function of watershed area. We found the quantiles associated with the local flow to be consistent with these other watersheds. This comparison is illustrated in Figure 11 for the p=0.01 flows and Figure 12 for the p=0.005 flows.
- Compared the peak flow-frequency curve to the results of the baseline analysis. The local flow peak flow-frequency curve should be less than the total flow regulated peak flow-frequency curve at Bellota, consistent with EM 1110-2-1415 guidance, and it is.

Table 4. Annual maximum local flow series used for frequency analysis

Water year (1)	**Peak local flow for area along Calaveras River between New Hogan Reservoir and Bellota, CA (cfs) (2)**	**1-day local flow for area along Calaveras River between New Hogan Reservoir and Bellota, CA (cfs) (3)**
1988	—	8,507
1989	—	—
1990	—	1,027
1991	—	7,823
1992	—	3,797
1993	—	7,471
1994	—	—
1995	—	—
1996	2,764	915
1997	6,688	3,312
1998	9,436	5,267
1999	5,620	2,762
2000	3,136	1,740
2001	2,069	1,066
2002	2,096	1,246
2003	681	432
2004	2,819	1,744
2005	3,505	2,081
2006	7,312	3,290
2007	1,149	369
2008	1,138	560
2009	908	638

Table 5. Calaveras River limited-use frequency curves: local flow between New Hogan Reservoir and Bellota, CA

Annual exceedence probability (1)	**1/annual exceedence probability (2)**	**Peak flow (cfs) (3)**	**1-day volume (cfs) (4)**
0.500	2	2,817	2,067
0.200	5	5,310	4,324
0.100	10	7,134	6,015
0.050	20	8,942	7,688
0.020	50	11,318	9,855
0.010	100	13,103	11,449
0.005	200	14,874	12,995
0.002	500	17,188	14,957



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.421	0.355	-0.492
1-day	3.270	0.427	-0.644

Notes:

- Median plotting positions.
- Drainage area: 110 sq. miles.
- Record lengths:
  Peak flows: 14 years.
  1-day volumes: 19 years.
- Regional skew values developed by USGS.

Figure 10. Limited-use peak local flow-frequency curve for areas along Calaveras River from New Hogan Reservoir to Bellota, CA

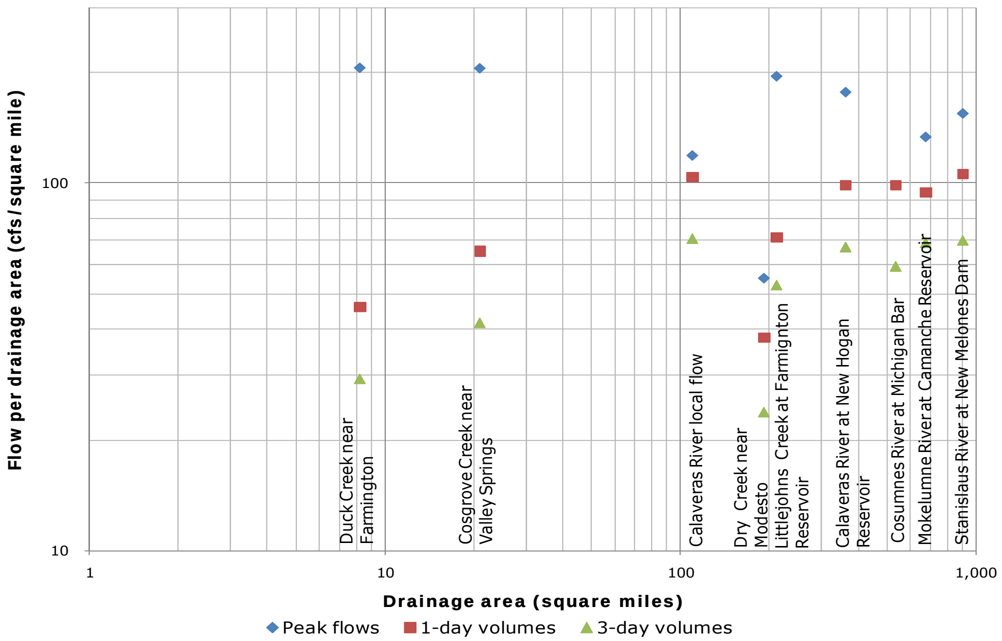


Figure 11. Comparison of regional flow per square mile ratios for the p=0.01 event; values from June 2011 report for Calaveras River and Littlejohn Creek, Comp Study (USACE 2001), and Cosgrove Creek study (USACE 2010)

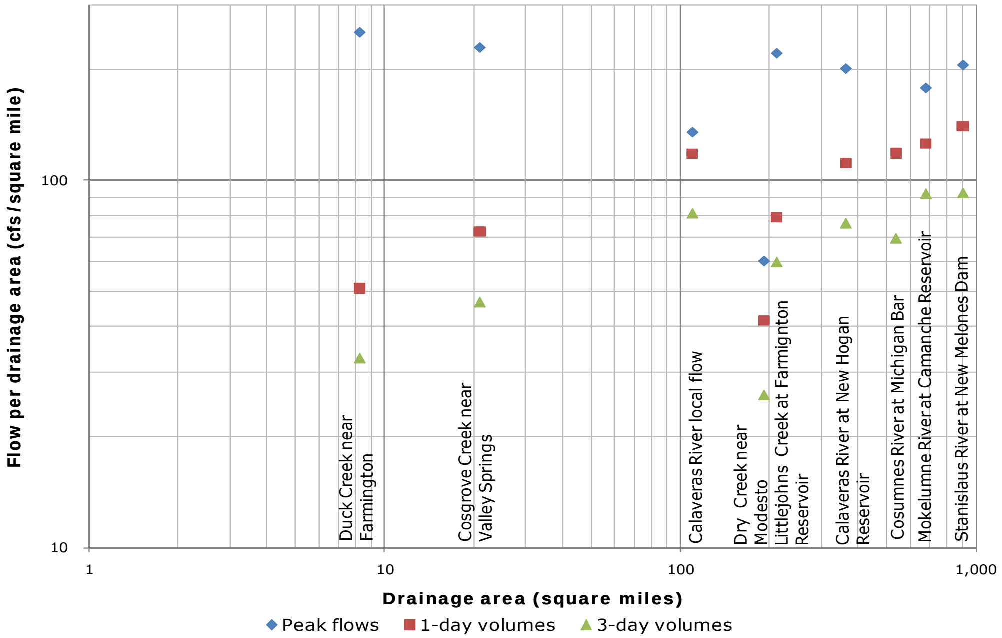


Figure 12. Comparison of regional flow per square mile ratios for the p=0.005 event; values from June 2011 report for Calaveras River and Littlejohn Creek, Comp Study (USACE 2001), and Cosgrove Creek study (USACE 2010)

Coincident event probabilities between local flow and reservoir inflow

Using the local flow-frequency curve in Figure 10 and the New Hogan reservoir inflow-frequency curve from the June 2011 report, we calculated and compared the annual exceedence probability (AEP) of the local flow and the coincident unregulated reservoir inflow for the selected events. We completed these comparisons considering the following combinations of flows, volumes, and events:

- Peak local flow and peak reservoir inflow for all 104 historical events. This is the entire period of record used for the Calaveras River frequency curves shown in the June 2011 report and is shown in Figure 13. As described in the June 2011 report, various computational options were used to estimate the local flow series for the historical events based on the availability of gage data. In the figure, we note by historical event which computational option was used for estimating that event's local flow.
- Peak local flow and 3-day reservoir inflow for 104 historical events. This is shown in Figure 14 and is similar to the comparison above, but here the 3-day reservoir value is used rather than the peak inflow. The values in the figure compare similarly between Figure 13 and Figure 14.
- Peak local flow and peak reservoir inflow for 8 scaled events. Here we focus on a combination of historical and scaled historical events, the same events as those listed in Table 2. This is shown in Figure 15.
- Peak local flow and 3-day reservoir inflow for 8 scaled events. This is shown in Figure 16 and is similar to the comparison above, but here the 3-day reservoir value is used rather than the peak inflow.
- Peak local flow and peak reservoir inflow for the 190 scaled historical events used to develop the flow transforms detailed in the baseline analysis. This is shown in Figure 17. Again, as described in the June 2011 report, various computational options were used to estimate the local flow series for the historical events based on the availability of gage data. In the figure, we note by historical event (which affects the scaled version of the historical event) which computational option was used for estimate the event's local flow.
- Peak local flow and 3-day reservoir inflow for the 190 scaled historical events used to develop the flow transforms detailed in the baseline analysis. This is shown in Figure 18 and is similar to the comparison above, but here the 3-day reservoir value is used rather than the peak inflow.

This analysis of coincidence AEP flows illustrates that there is not a consistent relation between the local flow and the reservoir inflow AEP values. In these figures, the area below the gray dashed "1 to 1" line indicates a region where the local flow AEP is greater than the reservoir inflow AEP. The area above the line indicates a region where the local flow AEP is less than the reservoir inflow AEP. On average for both the historical events and the scaled historical events, the local flow AEP tends to be greater than the reservoir inflow AEP. For example, when the reservoir inflow is a p=0.01 (100-yr) flow, the coincident local flow may be a p=0.10 (10-yr) flow. This is best illustrated in Figure 14 and Figure 18.

In Figure 17 and Figure 18, the events plotted appear to exhibit several linear and curved trends. The trends are scaled versions of a given historical event. Recall that historical events are scaled uniformly by specified factors, as described in the June 2011 report. The trends seen in the figures are a function of the relationship between the frequency curves used to calculate the probabilities of the peak local flows and the coincident event reservoir inflows. In Figure 19 and Figure 20, we highlight these trends with a select number of the historical events that were scaled and simulated to develop the unregulated to regulated flow transforms.

[For the baseline analysis, the local flow and reservoir inflow series were derived based on daily values. A data smoothing algorithm was then used to create synthetically an hourly series. The exception to this is where hourly data were available to derive the hourly local flow series directly. In this case, the derived hourly series was used directly. This flow development process is described in the June 2011 report. Here, the peak values from the baseline series are used.]

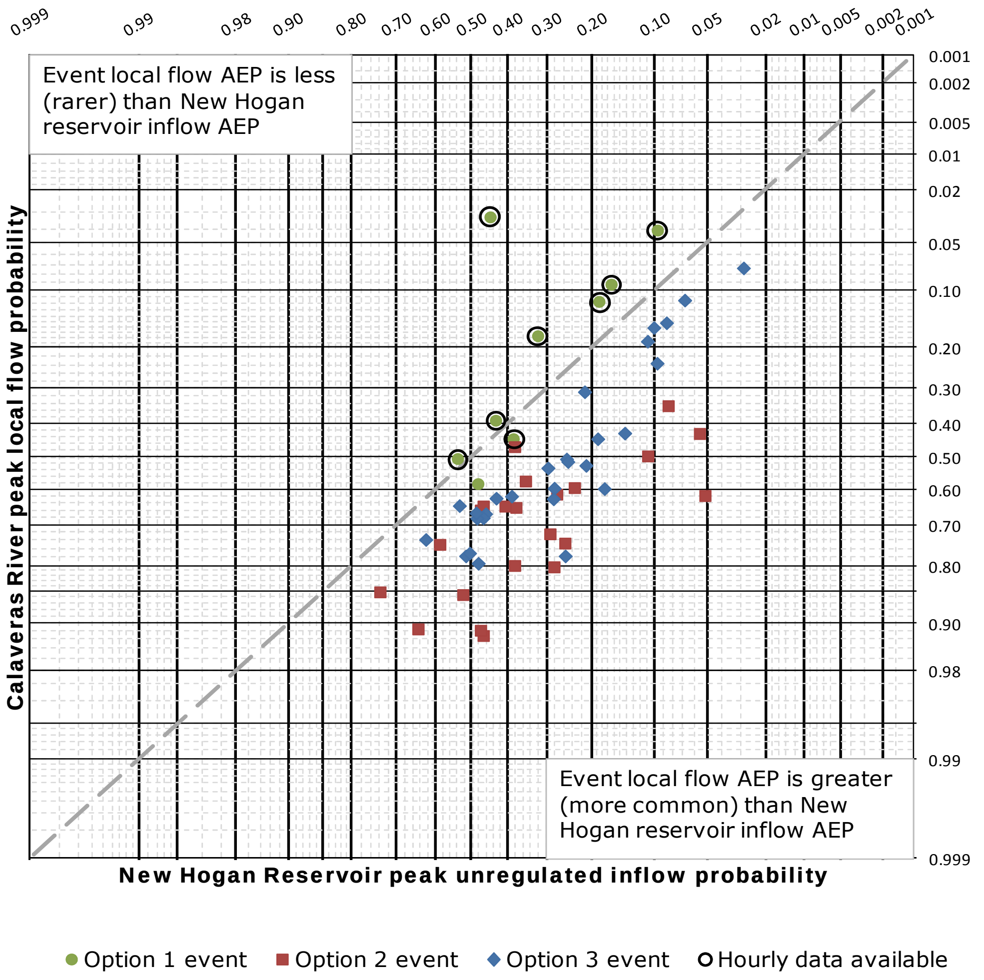


Figure 13. New Hogan Reservoir peak inflow and Calaveras River peak local flow coincident event probabilities: historical events

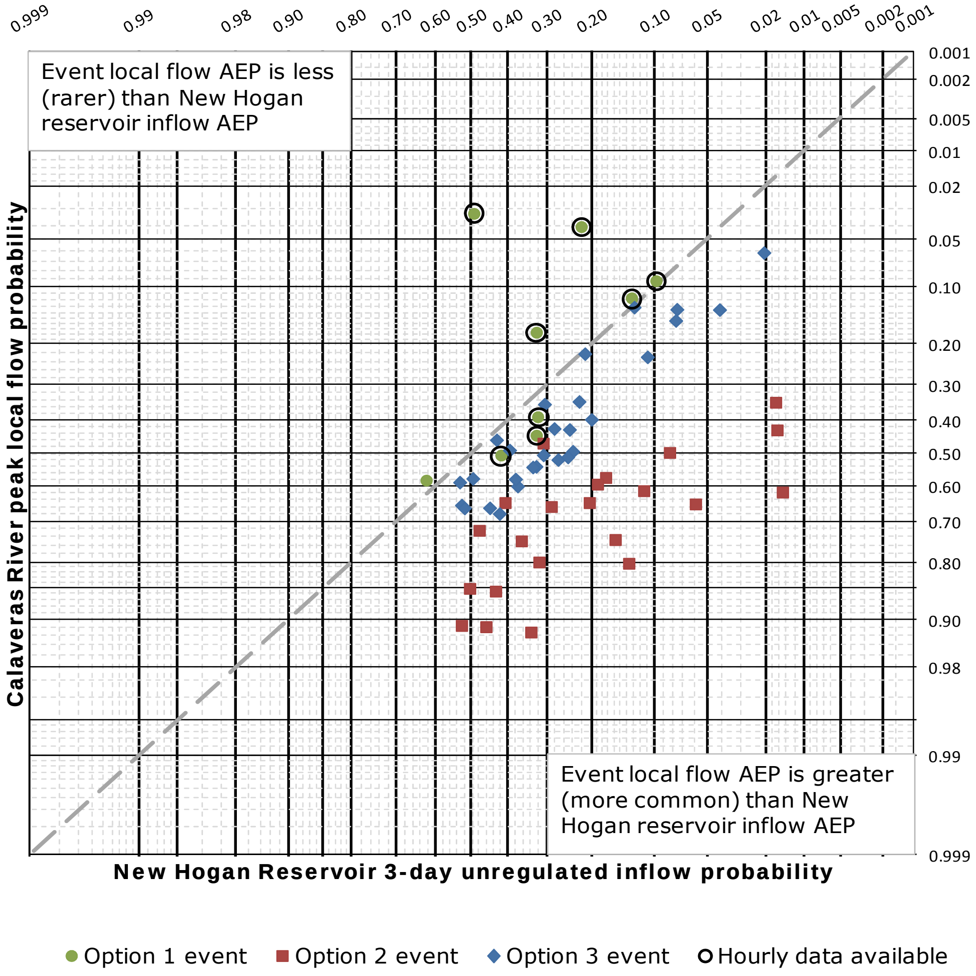


Figure 14. New Hogan Reservoir 3-day inflow volume and Calaveras River peak local flow coincident event probabilities: historical events

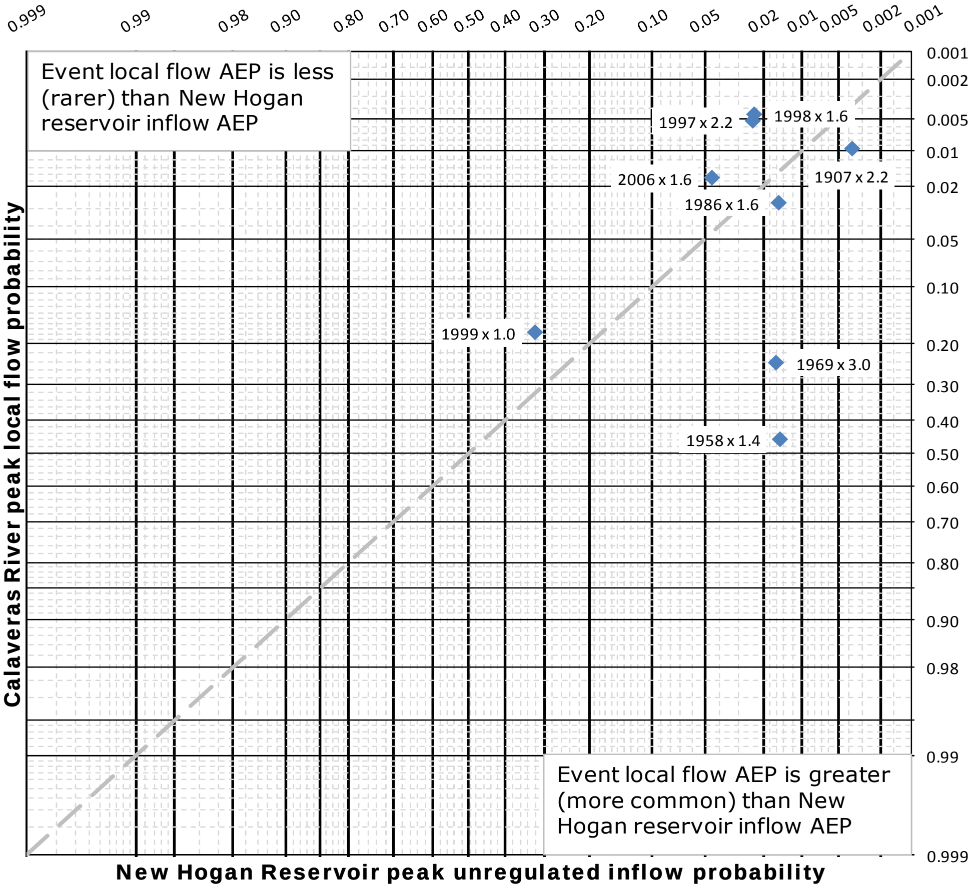


Figure 15. New Hogan Reservoir peak inflow and Calaveras River peak local flow coincident event probabilities: selected scaled events

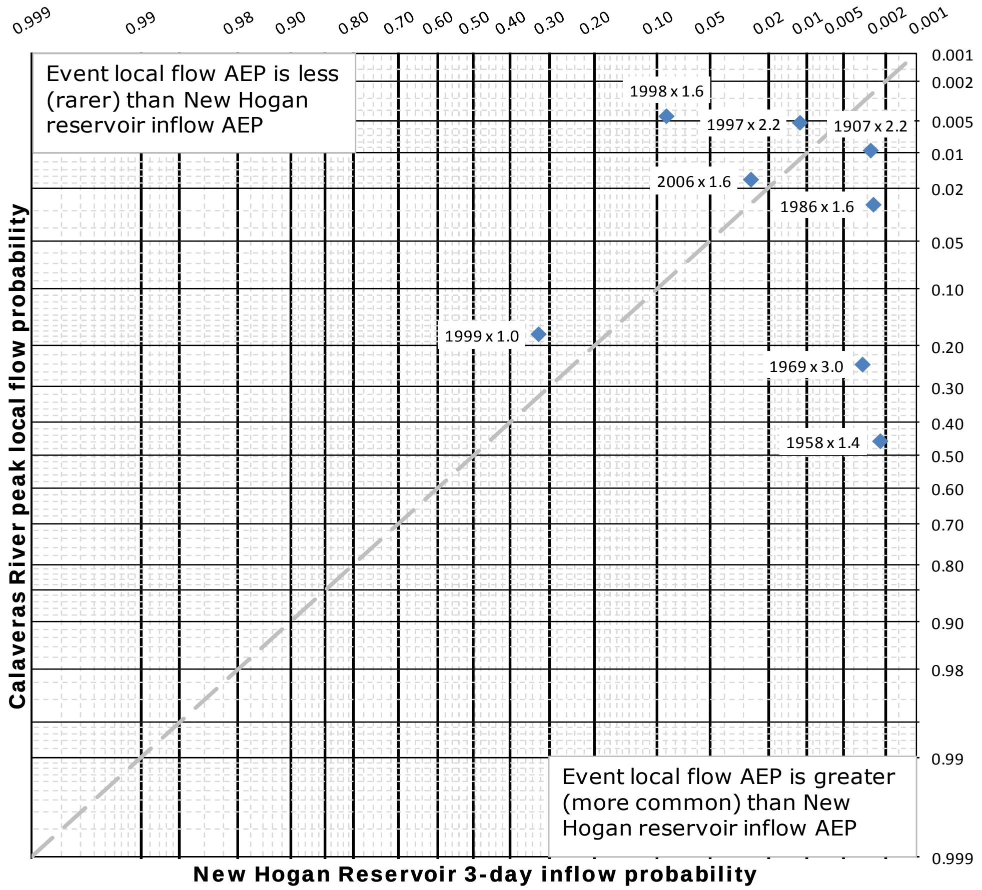


Figure 16. New Hogan Reservoir 3-day inflow volume and Calaveras River peak local flow coincident event probabilities: selected scaled events

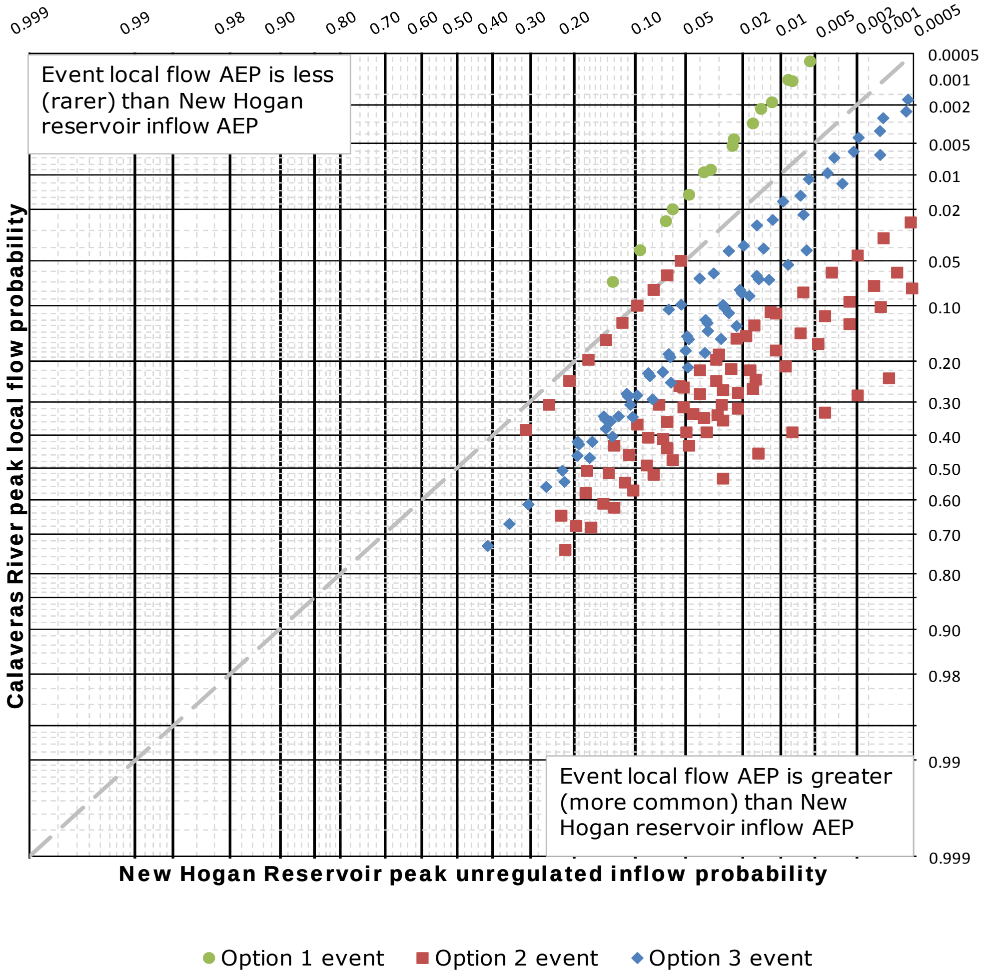


Figure 17. New Hogan Reservoir peak inflow and Calaveras River peak local flow coincident event probabilities: scaled events used to develop baseline flow transform

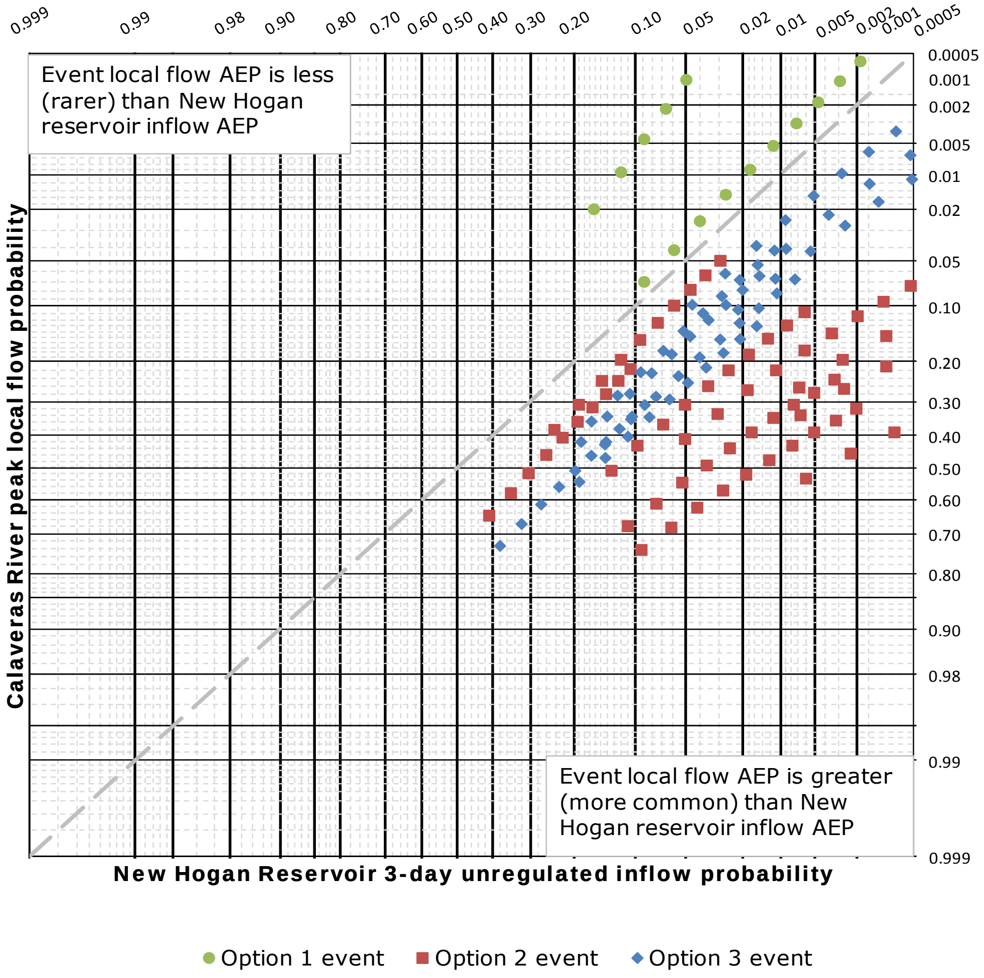


Figure 18. New Hogan Reservoir 3-day inflow volume and Calaveras River peak local flow coincident event probabilities: scaled events used to develop baseline flow transform

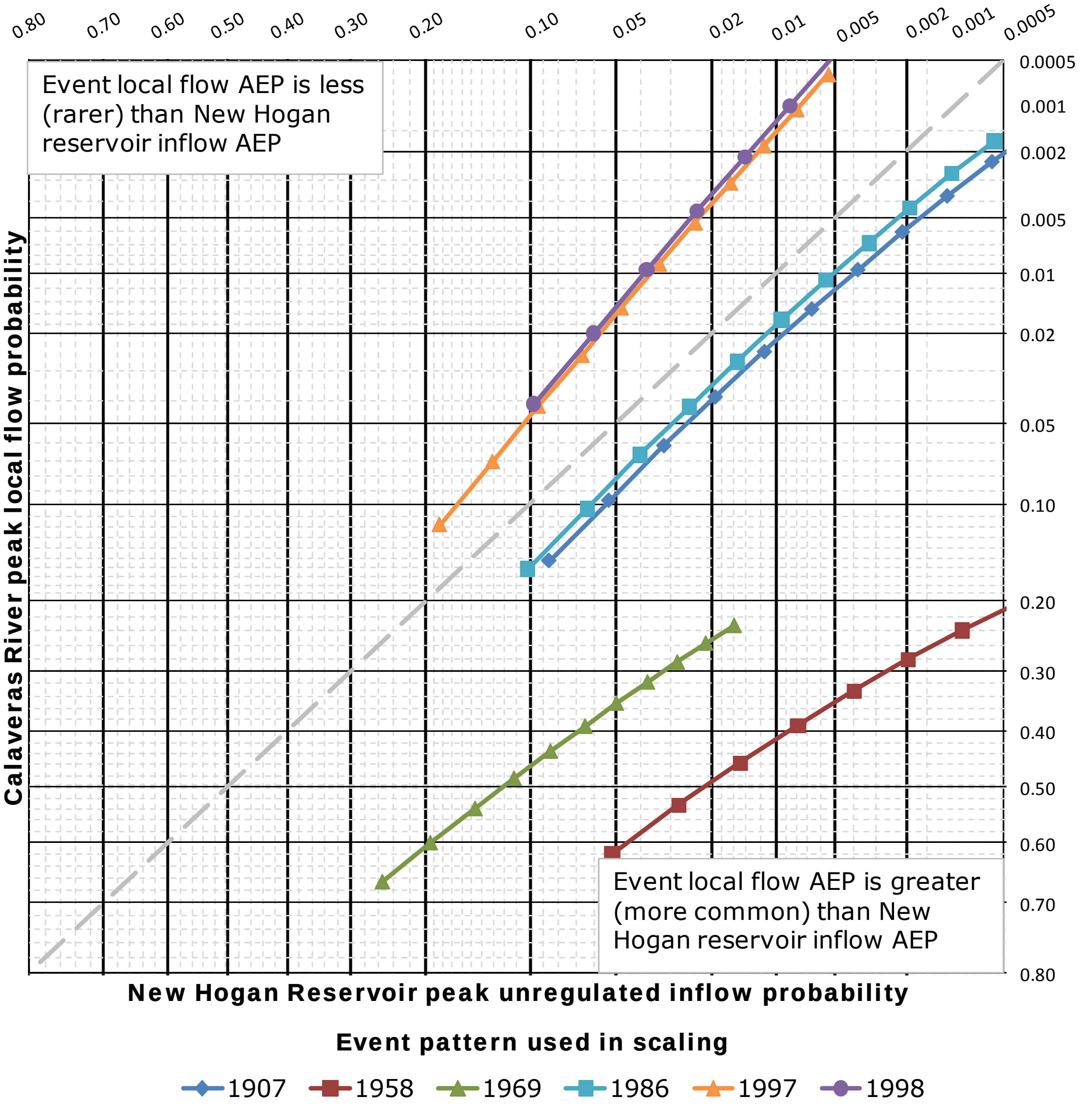


Figure 19. New Hogan Reservoir peak inflow and Calaveras River peak local flow coincident event probabilities: traces of select scaled events used to develop baseline flow transform

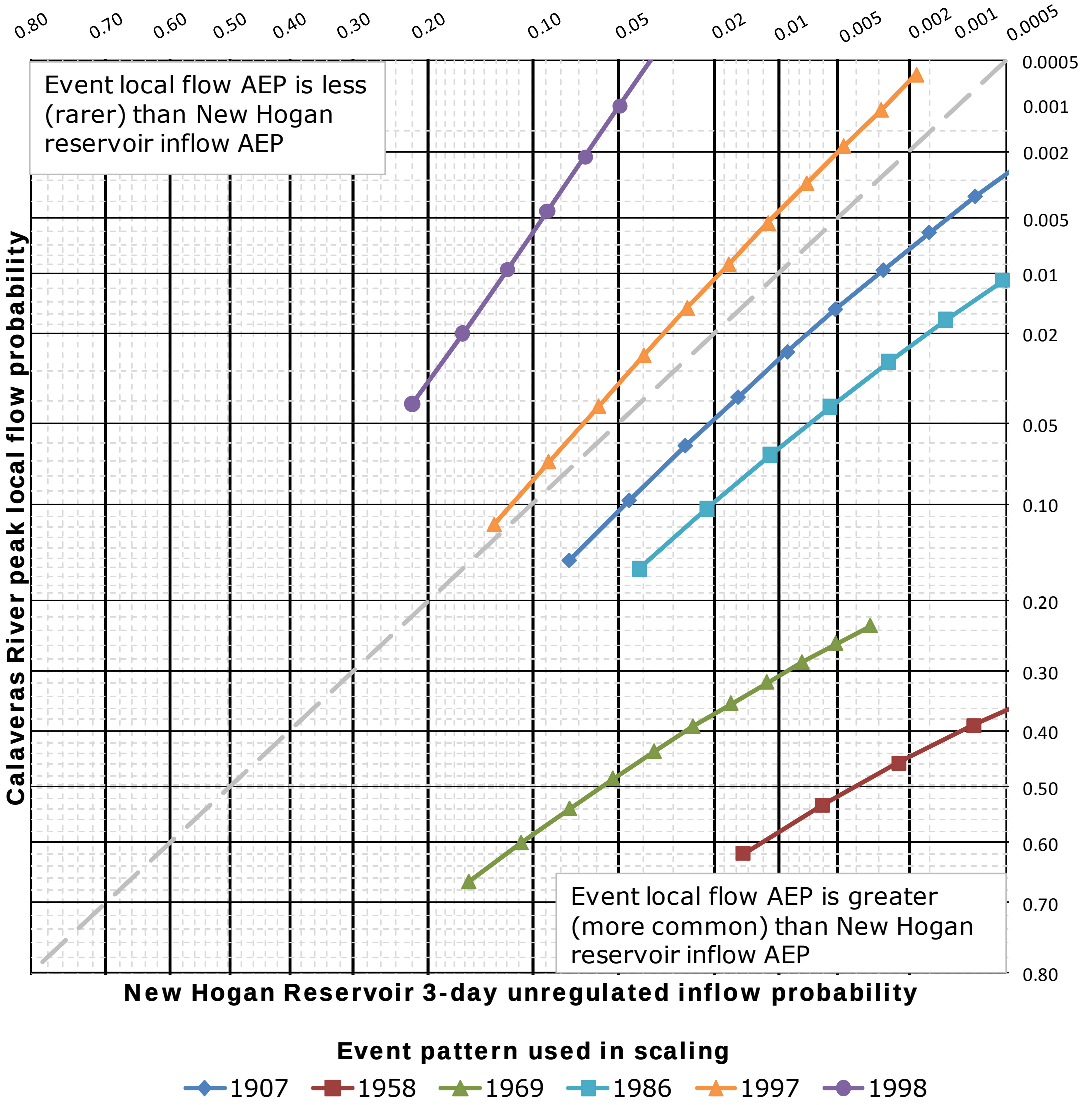


Figure 20. New Hogan Reservoir 3-day inflow volume and Calaveras River peak local flow coincident event probabilities: traces of select scaled events used to develop baseline flow transform

Attachment C. Reservoir simulation of design (scaled) events

Overview

To evaluate the sensitivity of reservoir flood storage and the effect of the uncontrolled local flows to the peak regulated flow-frequency curve at Bellota, we developed and evaluated an array of design (scaled) events. These design events (or hydrographs) are historical events scaled to a specific peak and/or volume(s) of specified probability. Here, we developed design events focused at p=0.005 flow at New Hogan and Bellota using consistent methodology as the baseline analysis described in the June 2011 report. We also developed and simulated design (scaled) events for the p=0.01 and p=0.002 flows at both locations.

Design hydrograph development and reservoir simulation

We developed design hydrographs at both New Hogan Reservoir and at Bellota. Thus, the design hydrographs for New Hogan Reservoir use the New Hogan unregulated flow-duration-frequency curve and the design hydrographs for Bellota use the Bellota unregulated flow-duration-frequency curve as documented in the June 2011 report.

To develop the design hydrographs for New Hogan Reservoir, we:

1. Selected historical events to serve as the template of the design hydrograph. Each historical event contains information including the temporal distribution (hydrograph shape and timing) and the spatial distribution (balance of flow above and below the reservoir). Here we used the 1958, 1986, 1997, 1998, and 2006 events.
2. Specified an AEP. We started with an AEP of 0.01.
3. Evaluated the AEPs of flow-duration properties of the selected historical hydrographs, using the New Hogan Reservoir flow-duration-frequency curve from the June 2011 report.
4. Scaled each selected event to a specified flow duration value. We started with duration equal to 3 days. For the scaling, both the reservoir inflow and the local flow are scaled uniformly. The uniform scale factor is computed as the specific flow-duration value from the unregulated flow-frequency curve matching the selected AEP divided by the corresponding flow-duration value from the historical event.
5. Simulated operation of the event with HEC-ResSim.
6. Recorded peak releases and downstream flows for each simulation.
7. Selected a different design duration and repeated steps 4 through 6. We repeated this process for durations of 4, 5, and 6 days in addition to the duration of 3 days.
8. Selected another AEP and repeated steps 3 though 7. We repeated this process for AEPs of 0.005 and 0.002 in addition to 0.01.

We repeated the process above for an analysis focused on flows at Bellota. Thus, the same steps were used but the Bellota unregulated flow-frequency curve from the June 2011 report was used instead of the New Hogan Reservoir inflow curve.

Following the process above, we developed and simulated 180 events. This includes 6 historical events, 3 design AEPs, 5 design durations, and 2 locations (unregulated flow-frequency curve). The results of these simulations are shown in Table 7 through Table 18.

Reservoir simulation results and synthesis

Below, by annual exceedence probability, we summarize our findings from the design (scaled) event simulations. Selective plots of the simulation of the design events, specifically those using the New Hogan frequency curve, are included. Additional plots of simulations using the Bellota frequency curve are on the CD delivered to the Corps.

For reference, we include Table 6 which describes the routing of the historical event used as a pattern for the design (scaled) events described herein.

Table 6. For reference, routing of historical events (no scaling)

Event (1)	Peak regulated flow at Bellota (cfs) (2)	Peak local flow at Bellota (cfs) (3)	New Hogan peak inflow (cfs) (4)	New Hogan peak release (cfs) (5)
1958	12,533	2,193	50,300	12,457
1986	12,500	5,850	35,500	12,244
1997	13,192	6,688	25,100	12,500
1998	13,422	9,436	25,300	12,500
2006	13,286	7,312	27,400	12,500

Baseline evaluation of p=0.01 design events

Table 7 includes simulation results for all durations for the p=0.01 design events scaled using the New Hogan frequency curve. Table 8 includes simulation results for all durations for the p=0.01 design event scaled using the Bellota frequency curve. Figure 21 through Figure 25 show reservoir routings of the 3-day design duration for 5 patterned events, scaled to p=0.01 flows using the New Hogan frequency curve. Although not included here, plots for all durations using both the New Hogan and Bellota frequency curves are on the CD delivered to the Corps.

The plots show that channel capacity of 12,500 cfs at Bellota is not exceeded for the 1958 and 1986 p=0.01 design events (scaled using either the New Hogan or Bellota frequency curve).

Channel capacity at Bellota is exceeded for all p=0.01 1997, 1998, and 2006 events (scaled using either the New Hogan or Bellota frequency curve). The channel capacity is exceeded because local flows at Bellota are greater than channel capacity.

The p=0.01 design (scaled) events for the 3-day duration are summarized in Table 9 and Table 10.

Table 7. p=0.01 design events scaled using the New Hogan frequency curve

Event pattern (1)	Duration (days) (2)	Scale factor (3)	Peak regulated flow at Bellota (cfs) (4)	Peak local flow (cfs) (5)	New Hogan peak inflow (cfs) (6)	New Hogan peak release (cfs) (7)
1958	3	1.09	12,500[1]	2,390	54,827	12,461
	4	1.06	12,500[1]	2,325	53,318	12,460
	5	1.04	12,500[1]	2,281	52,312	12,459
	6	1.01	12,500[1]	2,215	50,803	12,458
	7	0.99	12,500[1]	2,171	49,797	12,457
1986	3	1.43	12,500[1]	8,366	50,765	12,371
	4	1.32	12,500[1]	7,722	46,860	12,361
	5	1.32	12,500[1]	7,722	46,860	12,361
	6	1.36	12,500[1]	7,956	48,280	12,369
	7	1.41	12,500[1]	8,249	50,055	12,372
1997	3	2.25	13,192	15,073[2]	56,475	12,500[1]
	4	2.34	16,483	15,676[2]	58,734	12,500[1]
	5	2.43	16,961	16,279[2]	60,993	12,500[1]
	6	2.47	17,175	16,547[2]	61,997	12,500[1]
	7	2.53	17,497	16,948[2]	63,503	12,500[1]
1998	3	3.01	28,511	28,402[2]	76,153	14,723
	4	2.87	27,220	27,081[2]	72,611	12,500[1]
	5	2.55	24,280	24,062[2]	64,515	12,500[1]
	6	2.40	22,833	22,646[2]	60,720	12,500[1]
	7	2.39	22,741	22,552[2]	60,467	12,500[1]
2006	3	1.96	15,974	14,332[2]	53,704	12,500[1]
	4	2.05	16,233	14,990[2]	56,170	12,500[1]
	5	2.13	16,616	15,575[2]	58,362	12,500[1]
	6	2.21	18,089	16,160[2]	60,554	12,500[1]
	7	2.24	18,211	16,379[2]	61,376	12,500[1]

Notes:

1. Reservoir release adjusted by hand to 12,500 cfs to compensate for routing problem in HEC-ResSim. There is sufficient storage to contain event.
2. Local flow is greater than 12,500 cfs.

Table 8. p=0.01 design events scaled using the Bellota frequency curve

Event pattern (1)	Duration (days) (2)	Scale factor (3)	Peak regulated flow at Bellota (cfs) (4)	Peak local flow (cfs) (5)	New Hogan peak inflow (cfs) (6)	New Hogan peak release (cfs) (7)
1958	3	1.18	12,500[1]	2,588	59,354	12,466
	4	1.15	12,500[1]	2,522	57,845	12,465
	5	1.12	12,500[1]	2,456	56,336	12,466
	6	1.10	12,500[1]	2,413	55,330	12,466
	7	1.08	12,500[1]	2,369	54,324	12,465
1986	3	1.35	12,500[1]	7,898	47,925	12,368
	4	1.25	12,500[1]	7,313	44,375	12,343
	5	1.24	12,500[1]	7,254	44,020	12,337
	6	1.28	12,500[1]	7,488	45,440	12,351
	7	1.33	12,500[1]	7,781	47,215	12,364
1997	3	2.14	15,339	14,313[2]	53,714	12,500[1]
	4	2.25	16,040	15,049[2]	56,475	12,500[1]
	5	2.32	16,377	15,517[2]	58,232	12,500[1]
	6	2.37	16,643	15,851[2]	59,487	12,500[1]
	7	2.43	16,961	16,253[2]	60,993	12,500[1]
1998	3	2.63	25,014	24,817[2]	66,539	12,500[1]
	4	2.52	24,004	23,779[2]	63,756	12,500[1]
	5	2.29	21,836	21,608[2]	57,937	12,500[1]
	6	2.20	21,033	20,759[2]	55,660	12,500[1]
	7	2.20	21,033	20,759[2]	55,660	12,500[1]
2006	3	1.87	15,539	13,673[2]	51,238	12,500[1]
	4	1.97	16,024	14,404[2]	53,978	12,500[1]
	5	2.07	16,328	15,135[2]	56,718	12,500[1]
	6	2.12	16,568	15,500[2]	58,088	12,500[1]
	7	2.16	16,761	15,793[2]	59,184	12,500[1]

Notes:
1. Reservoir release rounded by hand to 12,500 cfs to compensate for routing problem in HEC-ResSim. There is sufficient storage to contain event.
2. Local flow is greater than 12,500 cfs.

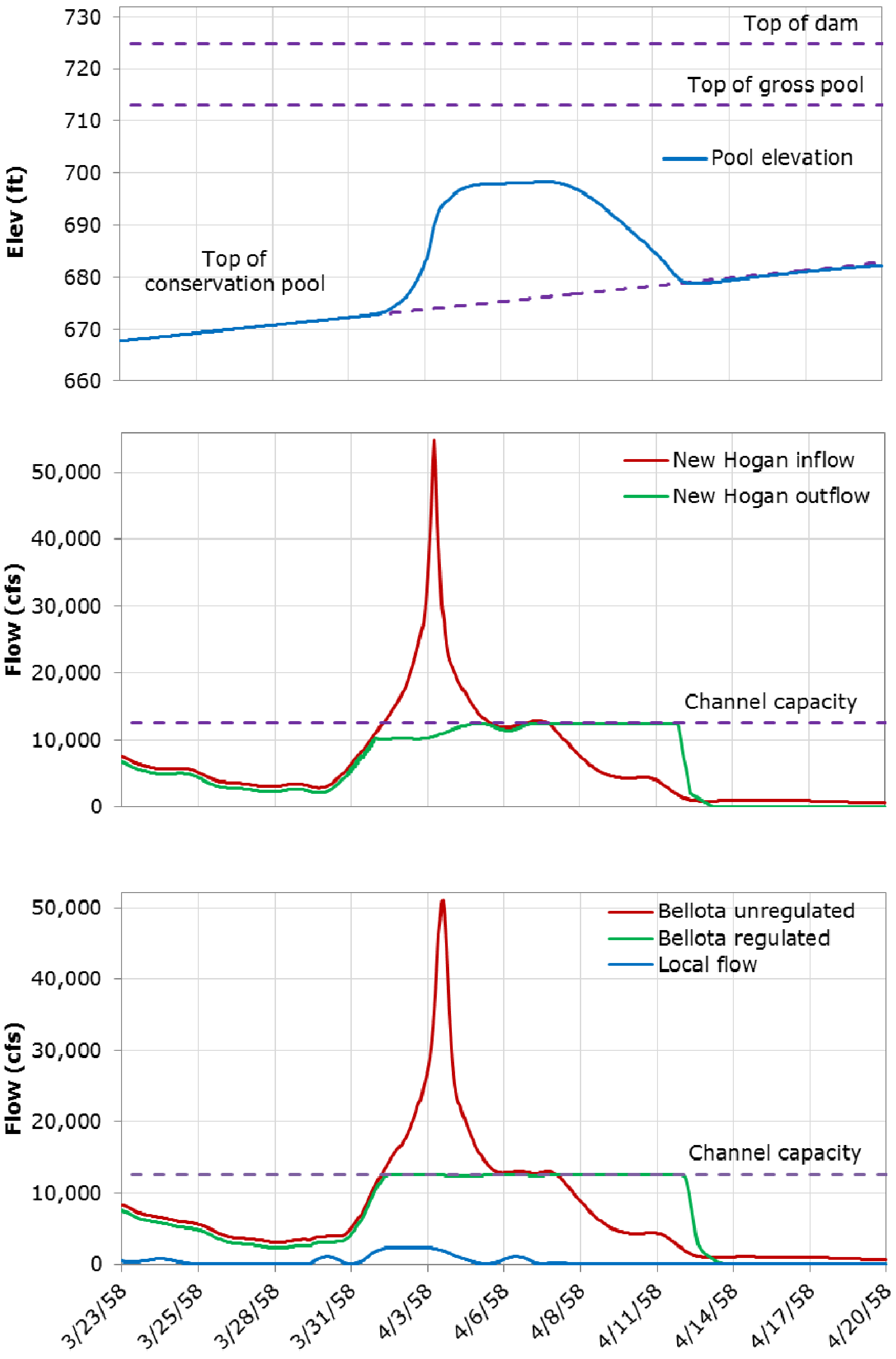


Figure 21. Reservoir routings of the 1958 event scaled using the New Hogan frequency curve to the p=0.01 3-day flow

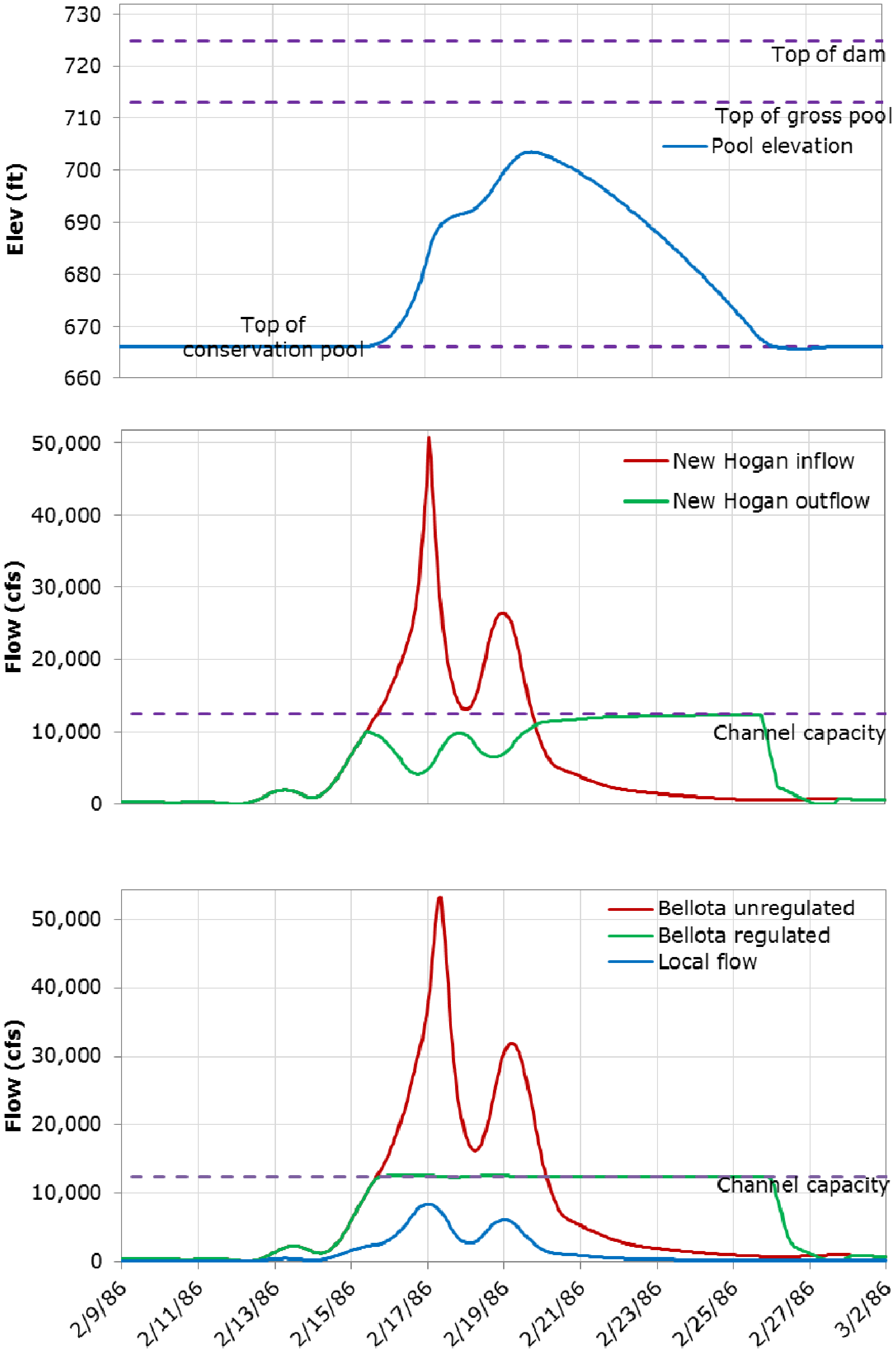


Figure 22. Reservoir routings of the 1986 event scaled using the New Hogan frequency curve to the p=0.01 3-day flow

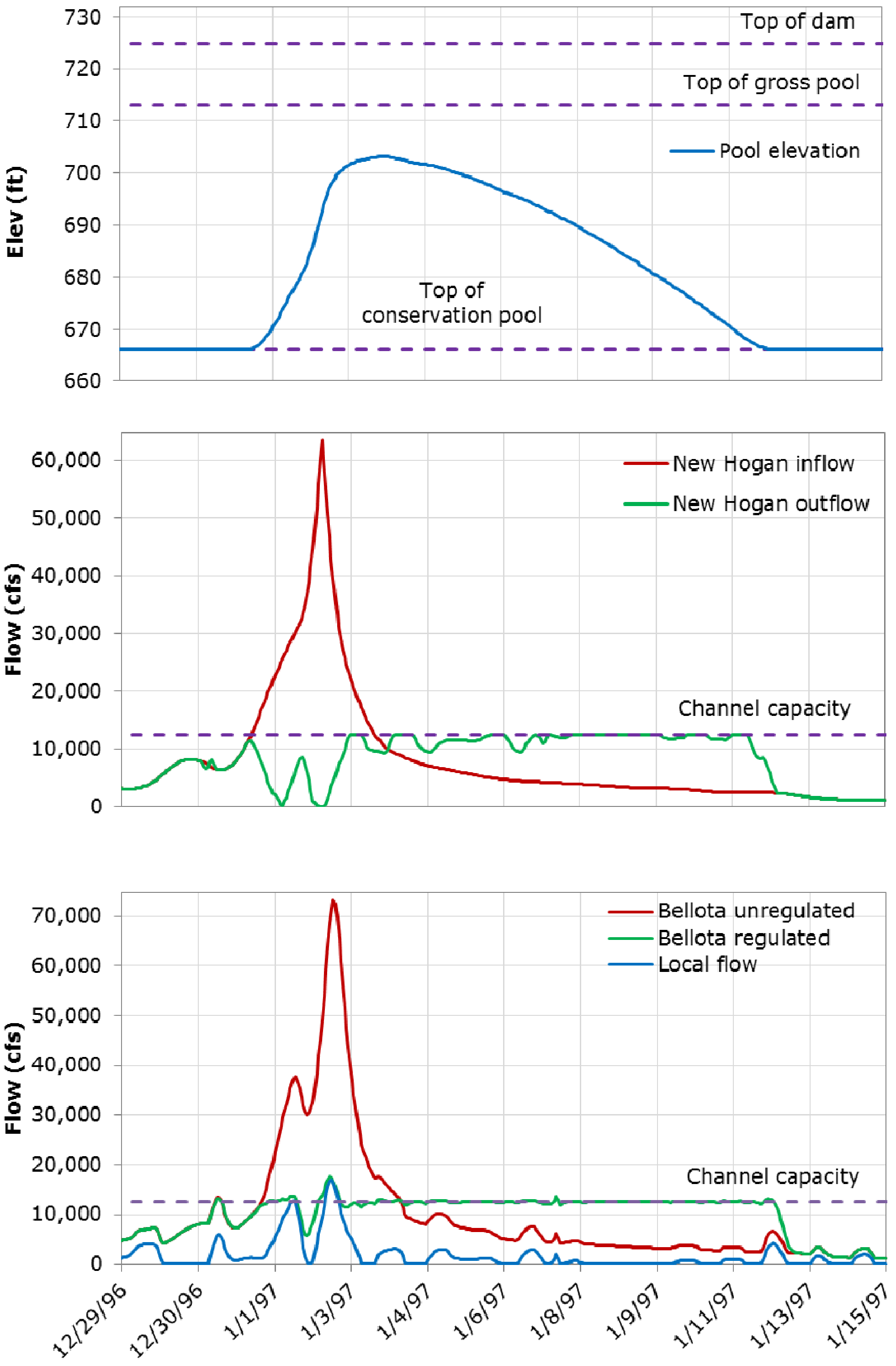


Figure 23. Reservoir routings of the 1997 event scaled using the New Hogan frequency curve to the p=0.01 3-day flow

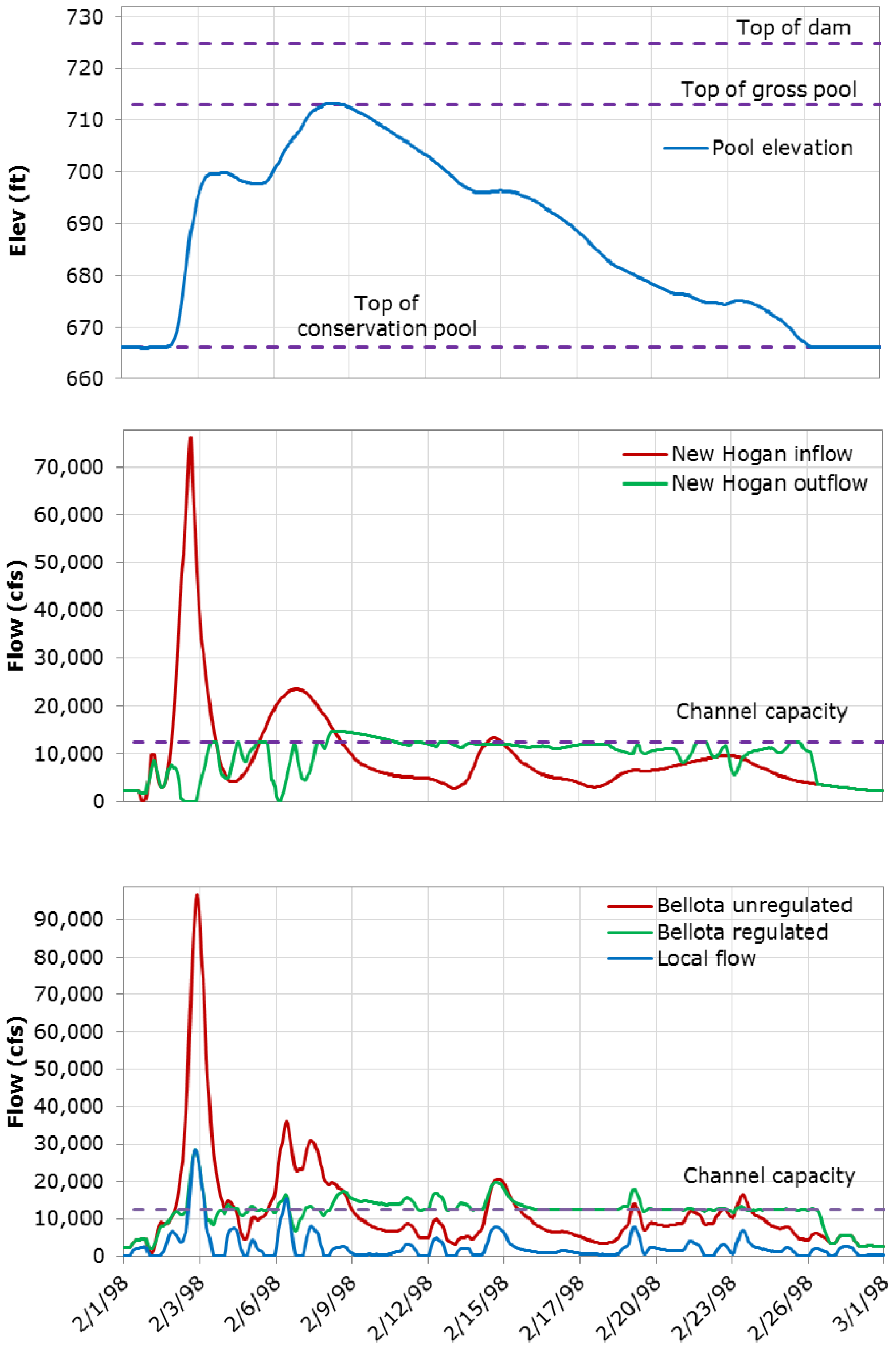


Figure 24. Reservoir routings of the 1998 event scaled using the New Hogan frequency curve to the p=0.01 3-day flow

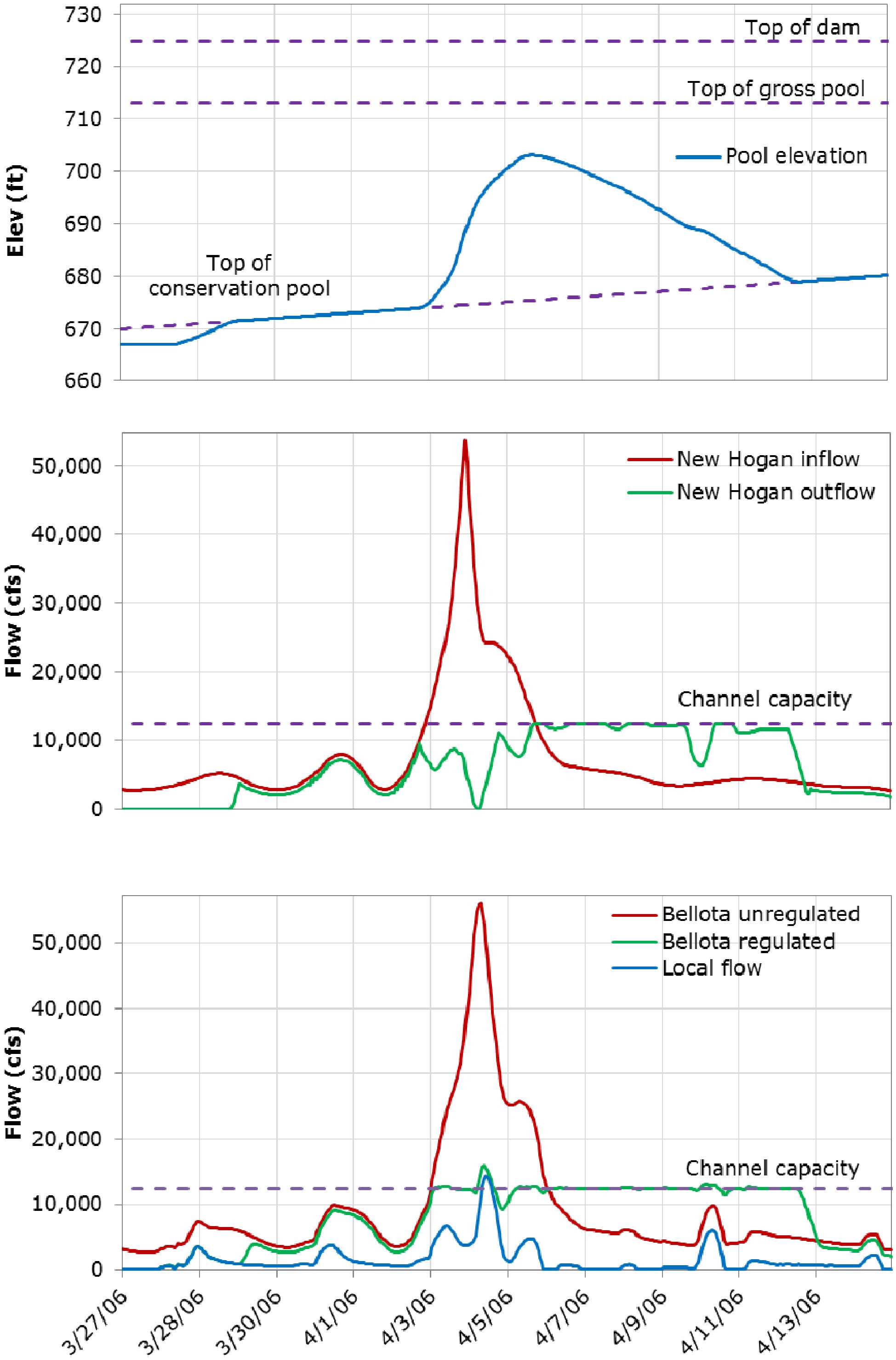


Figure 25. Reservoir routings of the 2006 event scaled using the New Hogan frequency curve to the p=0.01 3-day flow

Table 9. Summary of simulation results for all events analyzed; all flows scaled to the 3-day p=0.01 flow using the New Hogan frequency curve

Pattern event (1)	Channel capacity at Bellota exceeded? (2)	Notes (3)
1958	No	—
1986	No	—
1997	Yes	Peak local flow is greater than channel capacity for all durations.
1998	Yes	Peak local flow is greater than channel capacity for all durations.
2006	Yes	Peak local flow is greater than channel capacity for all durations.

Table 10. Summary of simulation results for all events analyzed; all flows scaled to the 3-day p=0.01 using the Bellota frequency curve

Pattern event (1)	Channel capacity at Bellota exceeded? (2)	Notes (3)
1958	No	—
1986	No	—
1997	Yes	Peak local flow is greater than channel capacity for all durations.
1998	Yes	Peak local flow is greater than channel capacity for all durations.
2006	Yes	Peak local flow is greater than channel capacity for all durations.

Baseline evaluation of p=0.005 design events

Table 11 includes simulation results for all durations for the p=0.005 design events scaled using the New Hogan frequency curve. Table 12 includes simulation results for all durations for the p=0.005 design event scaled using the Bellota frequency curve. Figure 26 through Figure 30 show reservoir routings for the 3-day design duration for 5 historical pattern events, scaled to p=0.005 flows using the New Hogan frequency curve. Although not included here, plots for all durations scaled using both New Hogan and Bellota frequency curves are on the CD delivered to the Corps.

The plots show the channel capacity of 12,500 cfs at Bellota is not exceeded for the 1958 and 1986 p=0.005 design events (scaled using either the New Hogan or Bellota frequency curve).

Channel capacity at Bellota is exceeded for all 1997, 1998, and 2006 events (scaled using either the New Hogan or Bellota frequency curve). The channel capacity is exceeded because local flows at Bellota are greater than channel capacity.

ESRD emergency releases are made in the 1998 3- and 4-day events and 2006 6- and 7-day events. ESRD releases are minimum releases that are required to be made to protect the integrity of the dam. The ESRD release is determined by the reservoir inflow and pool elevation. For all simulations which invoked an ESRD release, the release made was greater than channel capacity (12,500 cfs).

The p=0.005 design (scaled) events for the 3-day duration are summarized in Table 13 and Table 14.

Table 11. p=0.005 design events scaled using the New Hogan frequency curve

Event pattern (1)	Duration (days) (2)	Scale factor (3)	Peak regulated flow at Bellota (cfs) (4)	Peak local flow (cfs) (5)	New Hogan peak inflow (cfs) (6)	New Hogan peak release (cfs) (7)
1958	3	1.24	12,500[1]	2,719	62,372	12,466
	4	1.21	12,500[1]	2,654	60,863	12,466
	5	1.19	12,500[1]	2,610	59,857	12,465
	6	1.16	12,500[1]	2,544	58,348	12,465
	7	1.14	12,500[1]	2,500	57,342	12,464
1986	3	1.62	12,500[1]	9,477	57,510	12,352
	4	1.51	12,500[1]	8,834	53,605	12,362
	5	1.51	12,500[1]	8,834	53,605	12,362
	6	1.56	12,500[1]	9,126	55,380	12,357
	7	1.63	12,500[1]	9,536	57,865	12,351
1997	3	2.56	17,659	17,149[2]	64,256	12,500
	4	2.68	18,494	17,953[2]	67,268	12,500
	5	2.78	18,976	18,623[2]	69,778	12,500
	6	2.84	19,509	19,025[2]	71,284	12,500
	7	2.91	19,998	19,494[2]	73,041	12,500
1998	3	3.43	33,892	32,365[2]	86,779	25,651[3]
	4	3.28	31,030	30,950[2]	82,984	22,543[3]
	5	2.92	27,681	27,553[2]	73,876	12,500
	6	2.76	26,208	26,043[2]	69,828	12,500
	7	2.75	26,116	25,949[2]	69,575	12,500
2006	3	2.23	18,171	16,306[2]	61,102	12,500
	4	2.34	18,671	17,110[2]	64,116	12,500
	5	2.44	19,081	17,841[2]	66,856	12,500
	6	2.54	20,829	18,572[2]	69,596	17,872[3]
	7	2.58	23,515	18,865[2]	70,692	19,512[3]

Notes:
1. Reservoir release adjusted by hand to compensate for routing problem in HEC-ResSim. There is sufficient storage to contain event.
2. Local flow is greater than 12,500 cfs.
3. ESRD release.

Table 12. p=0.005 design events scaled using the Bellota frequency curve

Event pattern (1)	Duration (days) (2)	Scale factor (3)	Peak regulated flow at Bellota (cfs) (4)	Peak local flow (cfs) (5)	New Hogan peak inflow (cfs) (6)	New Hogan peak release (cfs) (7)
1958	3	1.33	12,500[1]	2,917	66,899	12,466
	4	1.30	12,500[1]	2,851	65,390	12,465
	5	1.28	12,500[1]	2,807	64,384	12,466
	6	1.25	12,500[1]	2,742	62,875	12,466
	7	1.23	12,500[1]	2,698	61,869	12,465
1986	3	1.52	12,500[1]	8,892	53,960	12,361
	4	1.41	12,500[1]	8,249	50,055	12,372
	5	1.41	12,500[1]	8,249	50,055	12,372
	6	1.46	12,500[1]	8,541	51,830	12,367
	7	1.52	12,500[1]	8,892	53,960	12,361
1997	3	2.42	16,908	16,186[2]	60,742	12,500[1]
	4	2.55	17,605	17,055[2]	64,005	12,500[1]
	5	2.63	18,106	17,590[2]	66,013	12,500[1]
	6	2.70	18,601	18,058[2]	67,770	12,500[1]
	7	2.77	18,914	18,527[2]	69,527	12,500[1]
1998	3	2.98	28,234	28,119[2]	75,394	12,923[3]
	4	2.86	27,127	26,987[2]	72,358	12,500[1]
	5	2.61	24,831	24,628[2]	66,033	12,500[1]
	6	2.51	23,912	23,684[2]	63,503	12,500[1]
	7	2.51	23,912	23,684[2]	63,503	12,500[1]
2006	3	2.11	16,520	15,427[2]	57,814	12,500[1]
	4	2.23	18,171	16,305[2]	61,102	12,500[1]
	5	2.36	18,659	17,255[2]	64,664	12,500[1]
	6	2.41	18,970	17,621[2]	66,034	12,500[1]
	7	2.46	19,140	17,986[2]	67,404	13,309[3]

Notes:

1. Reservoir release adjusted by hand to improve HEC-ResSim routing. There is sufficient storage to contain event.
2. Local flow is greater than 12,500 cfs.
3. ESRD release.

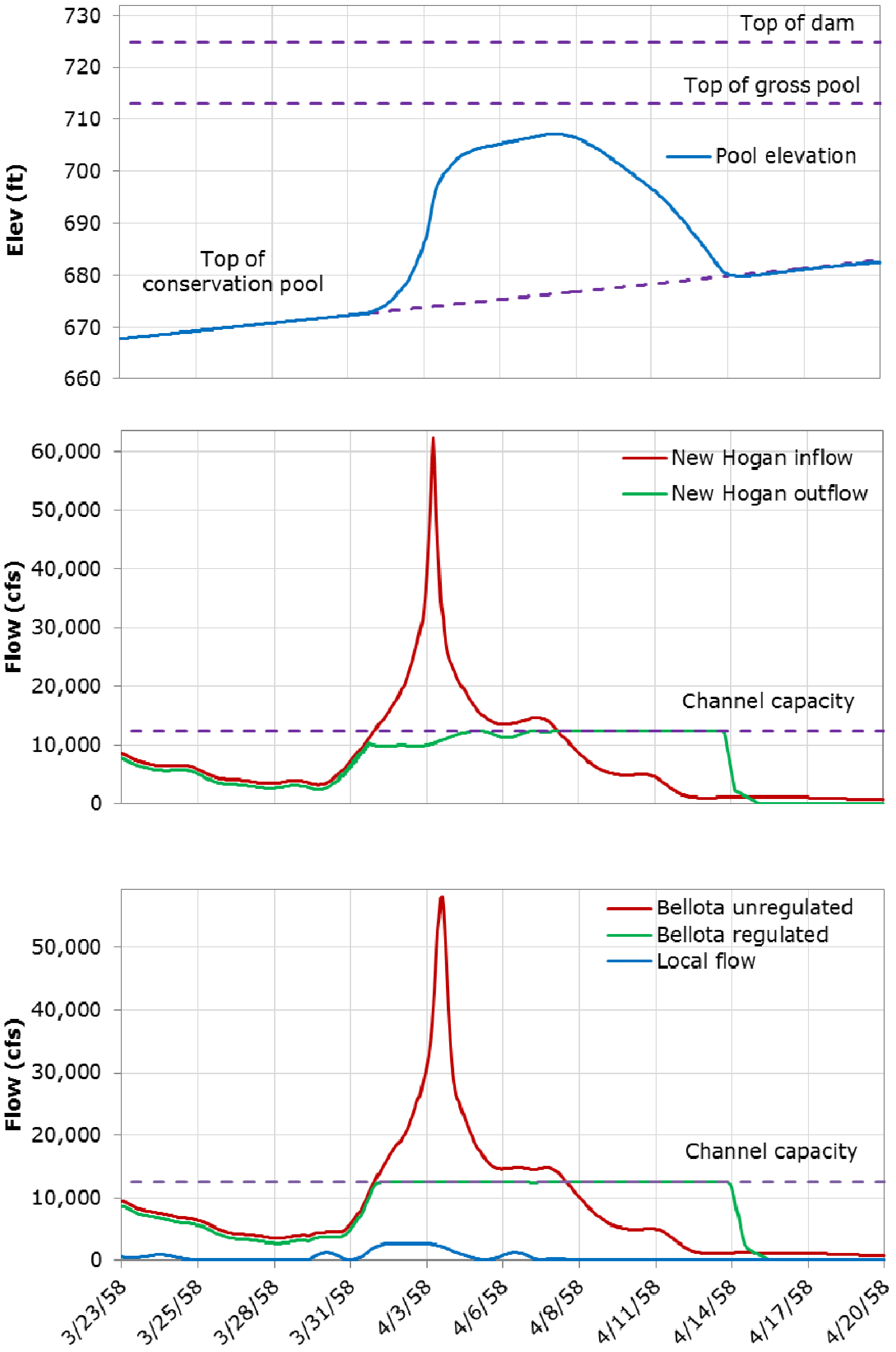


Figure 26. Reservoir routings of the 1958 event scaled using the New Hogan frequency curve to the p=0.005 3-day flow

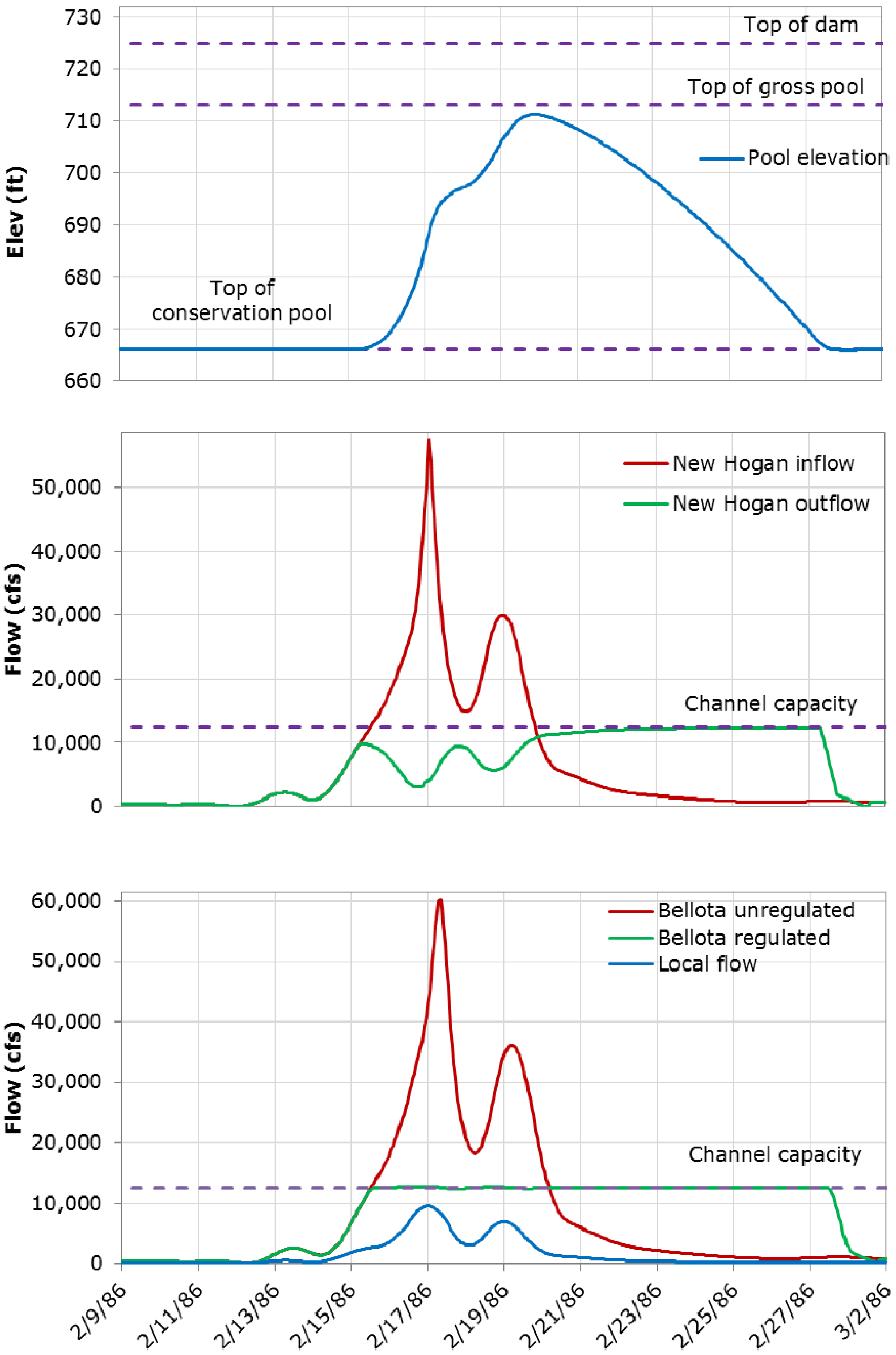


Figure 27. Reservoir routings of the 1986 event scaled using the New Hogan frequency curve to the p=0.005 3-day flow

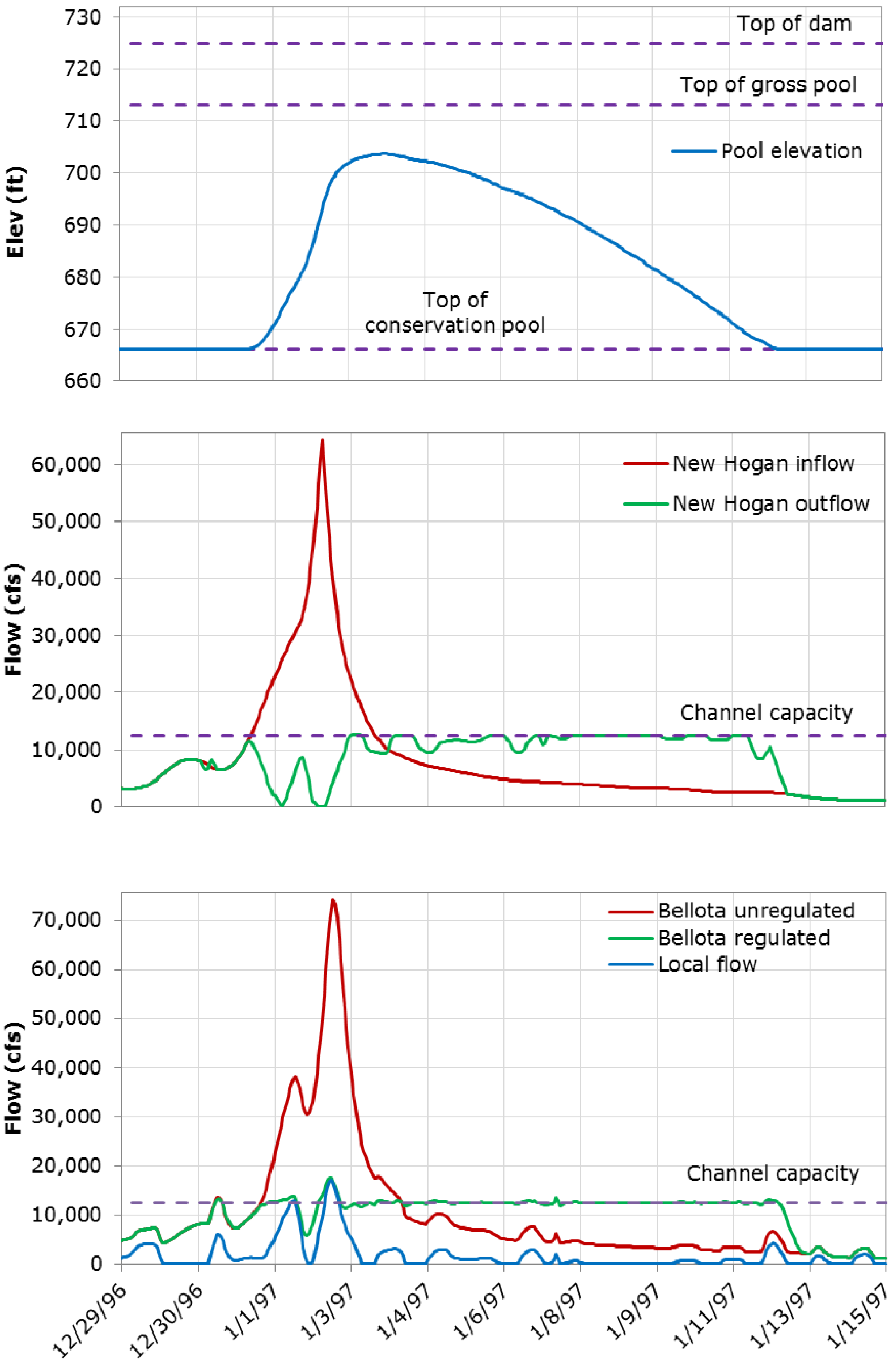


Figure 28. Reservoir routings of the 1997 event scaled using the New Hogan frequency curve to the p=0.005 3-day flow

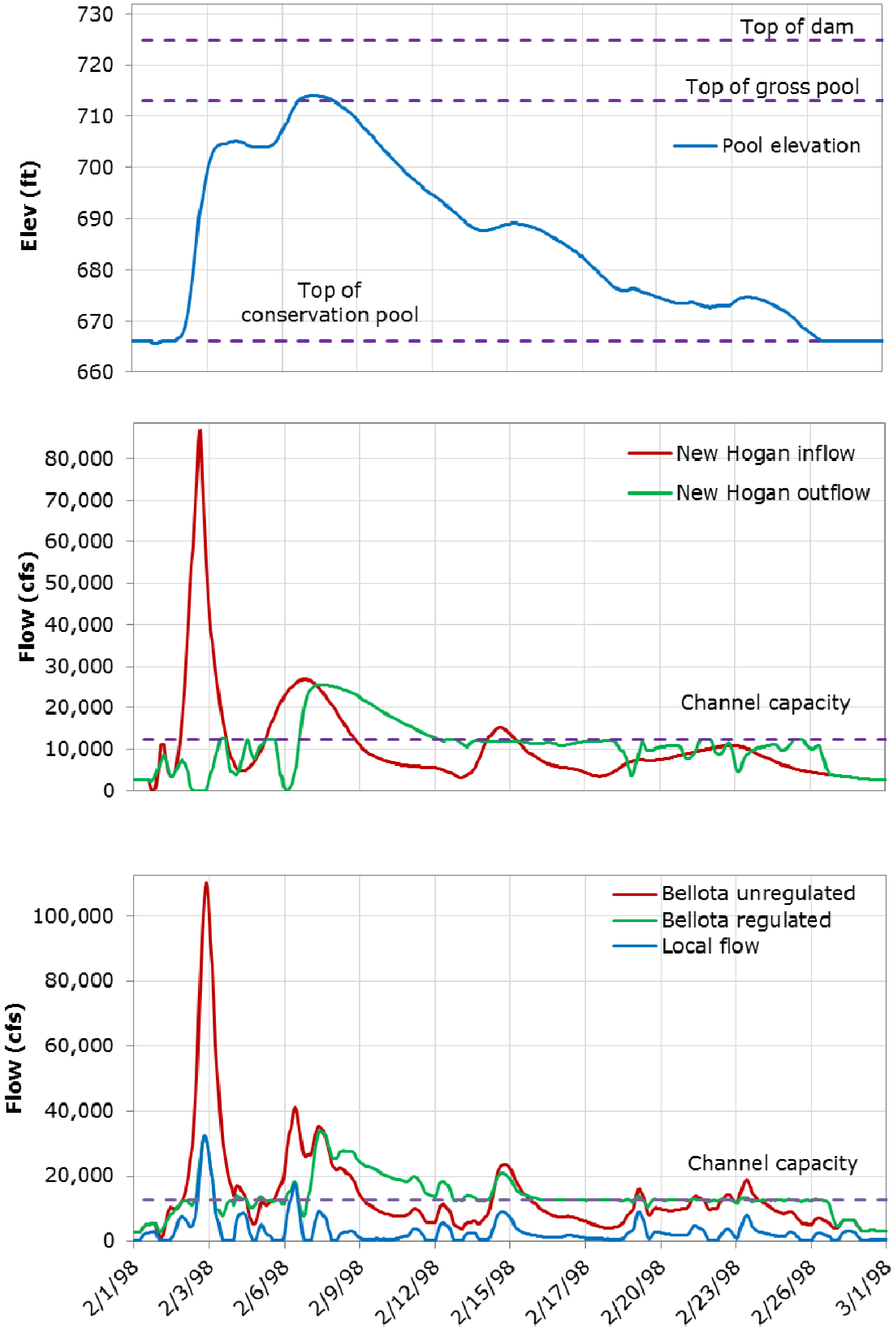


Figure 29. Reservoir routings of the 1998 event scaled using the New Hogan frequency curve to the p=0.005 3-day flow

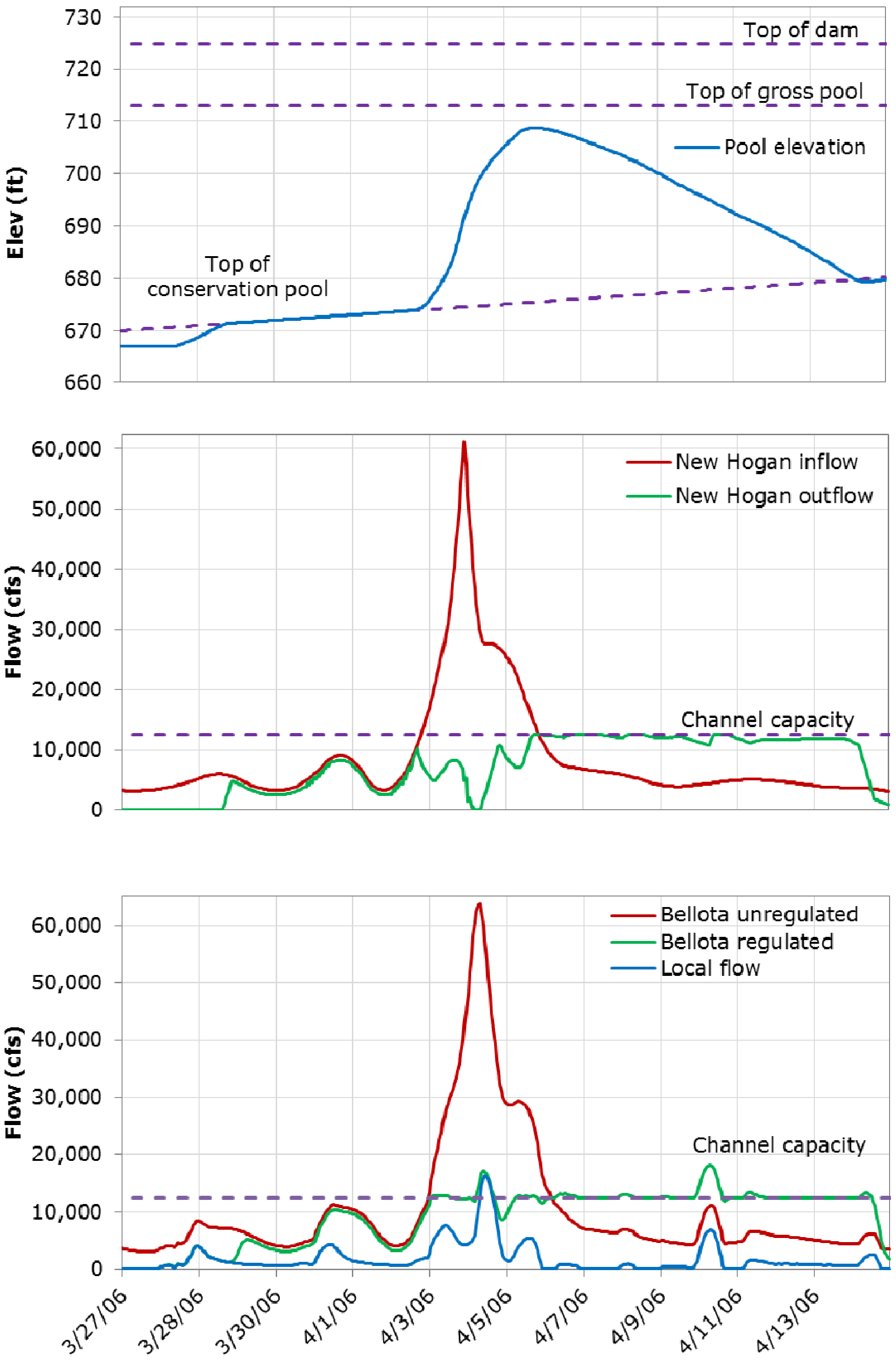


Figure 30. Reservoir routings of the 2006 event scaled using the New Hogan frequency curve to the p=0.005 3-day flow

Table 13. Summary of simulation results for all events analyzed; all flows scaled to the 3-day p=0.005 flow using the New Hogan frequency curve

Pattern event (1)	Channel capacity at Bellota exceeded? (2)	Notes (3)
1958	No	—
1986	No	—
1997	No	—
1998	Yes	Channel capacity at Bellota is exceeded due to local flows for all durations. In addition, ESRD releases are made at the 3-day and 4-day durations.
2006	Yes	Channel capacity at Bellota is exceeded due to local flows. In addition, ESRD releases are made at the 6-day and 7-day durations.

Table 14. Summary of simulation results for all events analyzed; all flows scaled to the p=0.005 flow using the Bellota frequency curve

Pattern event (1)	Channel capacity at Bellota exceeded? (2)	Notes (3)
1958	No	—
1986	No	—
1997	Yes	Channel capacity at Bellota is exceeded due to local flows for all durations.
1998	Yes	Channel capacity at Bellota is exceeded due to local flows for all durations. In addition, ESRD releases are made at the 3-day duration.
2006	Yes	Channel capacity at Bellota is exceeded due to local flows. In addition, ESRD releases are made at the 7-day duration.

Baseline evaluation of p=0.002 design events

Table 15 includes simulation results for all durations for the p=0.002 design events scaled using the New Hogan frequency curve. Table 16 includes simulation results for all durations for the p=0.002 design events scaled using the Bellota frequency curve. Figure 31 through Figure 35 show reservoir routings for the 3-day design duration for 5 historical pattern events, scaled to p=0.002 flows using the New Hogan frequency curve. Although not included here, plots for all durations using the New Hogan frequency curve and the Bellota frequency curve are on the CD delivered to the Corps.

The plots show that channel capacity at Bellota for each of the 5 design hydrographs scaled to the p=0.002 flow using the New Hogan frequency curve is exceeded for all design events except for 3: the 7-day 1958 event scaled to the New Hogan frequency curve, and the 1986 4-day and 5-day events scaled to the Bellota frequency curve. Local flows alone are greater than channel capacity for all 1997, 1998, and 2006 design events. ESRD releases are made for most p=0.002 design events which contribute to downstream flooding.

The p=0.002 design (scaled) events for the 3-day duration are summarized in Table 17 and Table 18.

Table 15. p=0.002 design events scaled using the New Hogan frequency curve

Event pattern (1)	Duration (days) (2)	Scale factor (3)	Peak regulated flow at Bellota (cfs) (4)	Peak local flow (cfs) (5)	New Hogan peak inflow (cfs) (6)	New Hogan peak release (cfs) (7)
1958	3	1.42	16,999	3,114	71,426	16,789[3]
	4	1.40	16,759	3,070	70,420	16,552[3]
	5	1.39	16,628	3,048	69,917	16,422[3]
	6	1.36	15,230	2,983	68,408	15,069[3]
	7	1.33	12,500[1]	2,917	66,899	12,466
1986	3	1.87	35,436	10,940	66,385	31,171[3]
	4	1.75	24,134	10,238	62,125	21,256[3]
	5	1.75	24,134	10,238	62,125	21,256[3]
	6	1.82	30,740	10,647	64,610	27,272[3]
	7	1.90	37,637	11,115	67,450	32,835[3]
1997	3	2.95	20,218	19,762[2]	74,045	12,500
	4	3.09	21,099	20,700[2]	77,559	12,500
	5	3.23	22,642	21,638[2]	81,073	18,817[3]
	6	3.31	24,752	22,174[2]	83,081	20,985[3]
	7	3.40	26,763	22,777[2]	85,340	23,115[3]
1998	3	3.94	41,261	37,178[2]	99,682	30,731[3]
	4	3.80	39,713	35,857[2]	96,140	29,559[3]
	5	3.39	32,726	31,988[2]	85,767	24,706[3]
	6	3.21	30,376	30,290[2]	81,213	21,041[3]
	7	3.21	30,376	30,290[2]	81,213	21,041[3]
2006	3	2.56	22,110	18,719[2]	70,144	18,655[3]
	4	2.70	29,668	19,742[2]	73,980	23,752[3]
	5	2.84	36,275	20,766[2]	77,816	29,521[3]
	6	2.96	39,697	21,644[2]	81,104	32,586[3]
	7	3.01	41,060	22,009[2]	82,474	33,868[3]

Notes:

1. Reservoir release adjusted by hand to improve HEC-ResSim routing. There is sufficient storage to contain event.
2. Local flow is greater than 12,500 cfs.
3. ESRD release.

Table 16. p=0.002 design events scaled using the Bellota frequency curve

Pattern event (1)	Duration (days) (2)	Scale factor (3)	Peak regulated flow at Bellota (cfs) (4)	Peak local flow (cfs) (5)	New Hogan peak inflow (cfs) (6)	New Hogan peak release (cfs) (7)
1958	3	1.52	21,409	3,334	76,456	20,638[3]
	4	1.49	19,897	3,268	74,947	18,799[3]
	5	1.47	18,738	3,224	73,941	17,458[3]
	6	1.45	17,620	3,180	72,935	17,018[3]
	7	1.42	16,999	3,114	71,426	16,789[3]
1986	3	1.74	23,159	10,179	61,770	20,414[3]
	4	1.62	12,500[1]	9,477	57,510	12,352
	5	1.63	12,500[1]	9,536	57,865	12,351
	6	1.69	17,338	9,887	59,995	15,171[3]
	7	1.76	25,065	10,296	62,480	22,080[3]
1997	3	2.76	18,852	18,460[2]	69,276	12,500[1]
	4	2.93	20,109	19,597[2]	73,543	12,500[1]
	5	3.03	20,892	20,266[2]	76,053	12,500[1]
	6	3.12	21,194	20,867[2]	78,312	12,500[1]
	7	3.21	21,745	21,469[2]	80,571	17,854[3]
1998	3	3.40	33,019	32,082[2]	86,020	24,976[3]
	4	3.29	31,123	31,044[2]	83,237	22,770[3]
	5	3.00	28,419	28,308[2]	75,900	14,166[3]
	6	2.90	27,496	27,364[2]	73,370	12,500[1]
	7	2.91	27,589	27,459[2]	73,623	12,500[1]
2006	3	2.41	18,970	17,621[2]	66,034	12,500[1]
	4	2.56	22,110	18,717[2]	70,144	18,655[3]
	5	2.72	30,654	19,887[2]	74,528	24,499[3]
	6	2.79	34,307	20,399[2]	76,446	27,842[3]
	7	2.85	36,596	20,838[2]	78,090	29,784[3]

Notes:
1. Reservoir release adjusted by hand to improve HEC-ResSim routing. There is sufficient storage to contain event.
2. Local flow is greater than 12,500 cfs.
3. ESRD release.

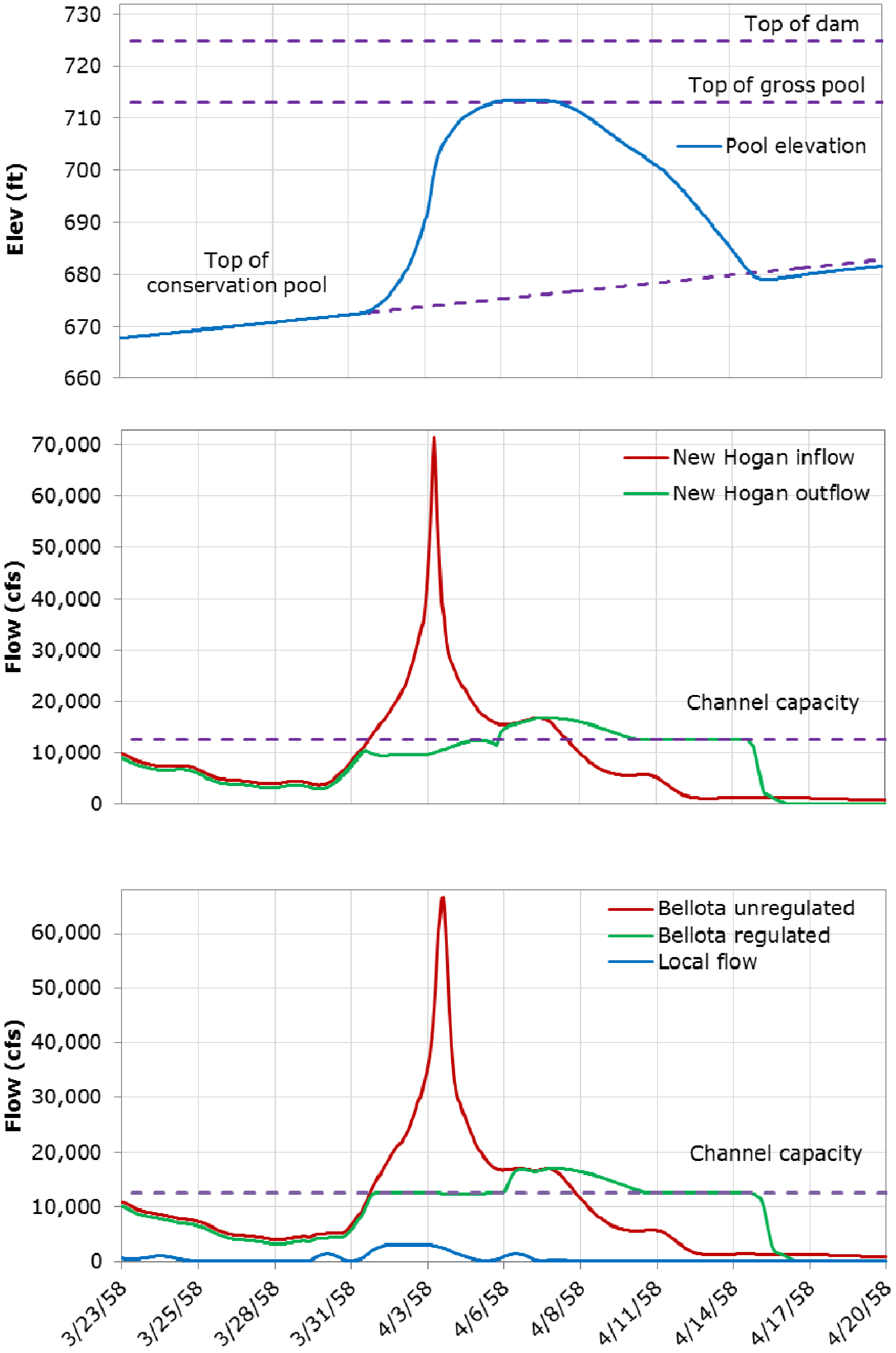


Figure 31. Reservoir routings of the 1958 event scaled using the New Hogan frequency curve to the p=0.002 3-day flow

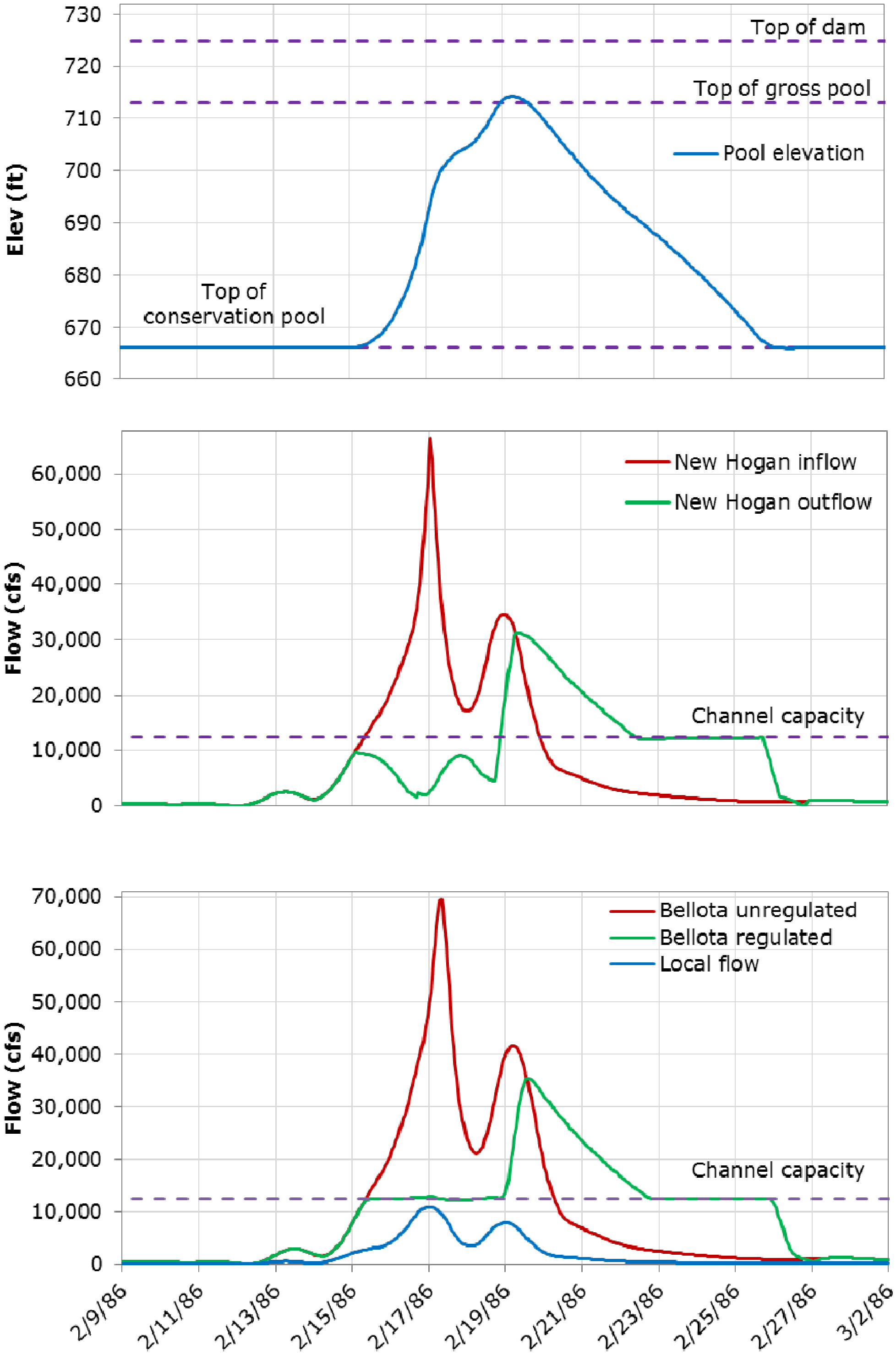


Figure 32. Reservoir routings of the 1986 event scaled using the New Hogan frequency curve to the p=0.002 3-day flow

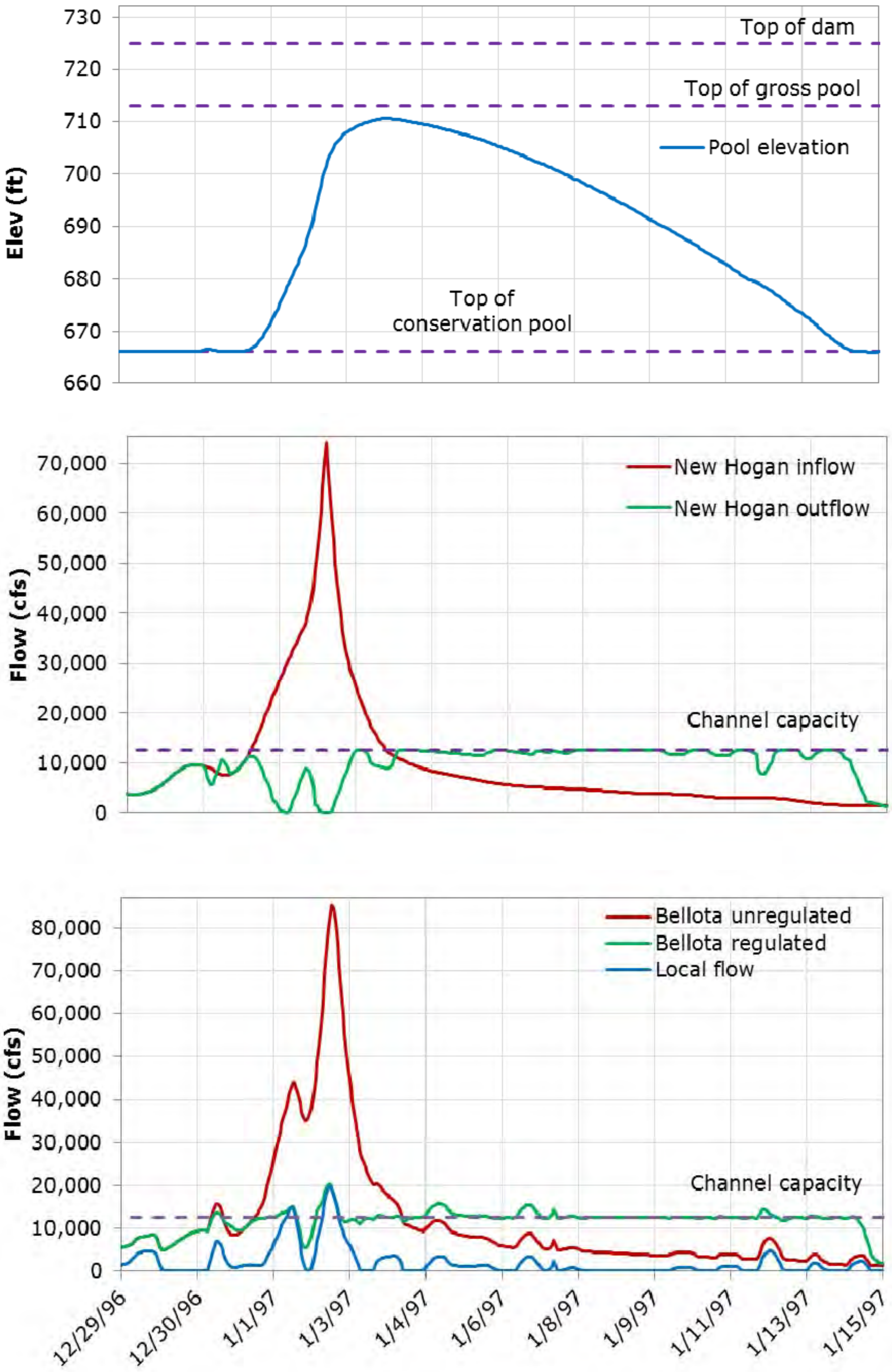


Figure 33. Reservoir routings of the 1997 event scaled using the New Hogan frequency curve to the p=0.002 3-day flow

Figure 34. Reservoir routings of the 1998 event scaled using the New Hogan frequency curve to the p=0.002 3-day flow

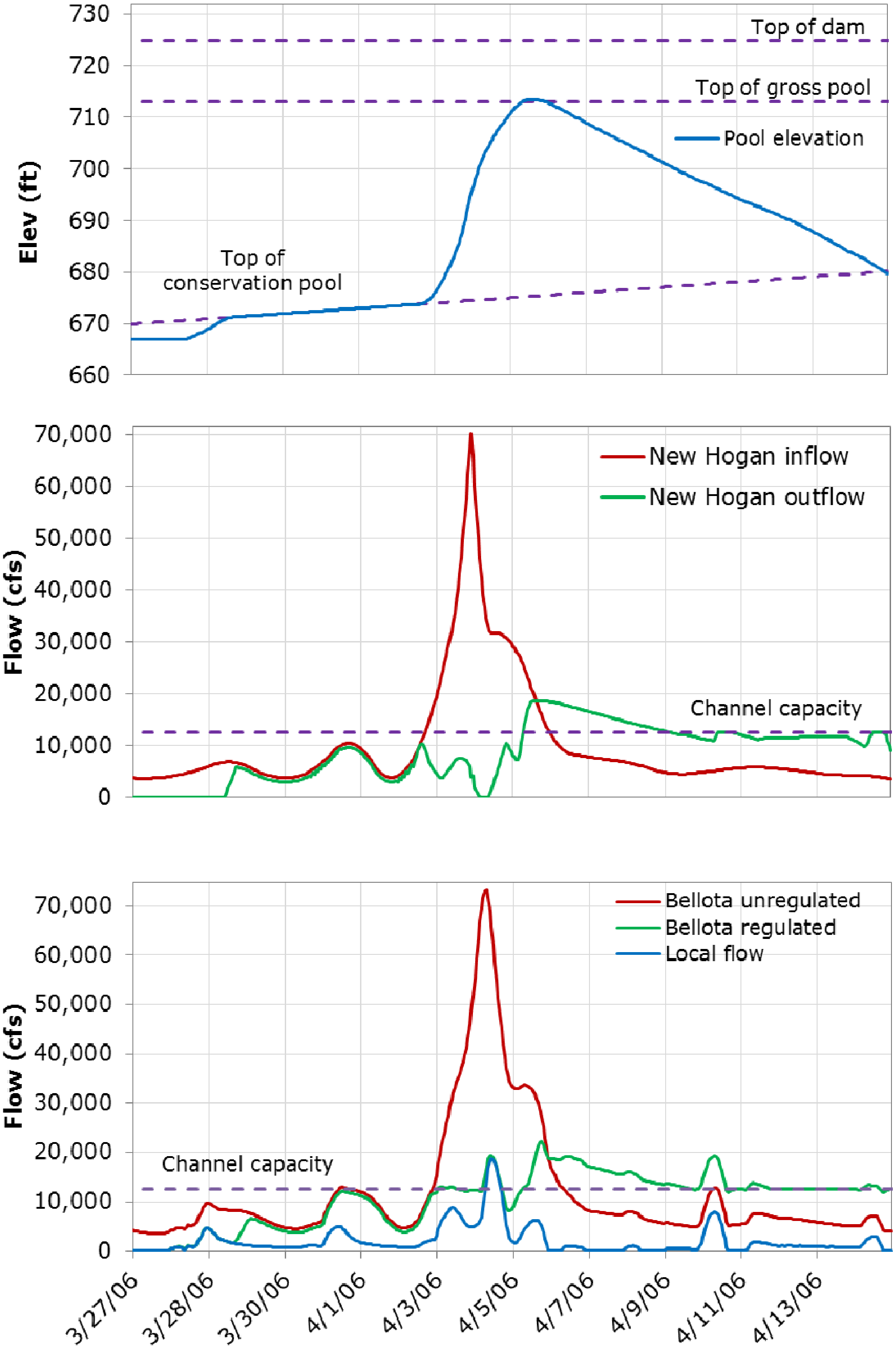


Figure 35. Reservoir routings of the 2006 event scaled using the New Hogan frequency curve to the p=0.002 3-day flow

Table 17. Summary of New Hogan operation design events; all flows scaled to p=0.002 flows using the New Hogan frequency curve

Pattern event (1)	**Channel capacity at Bellota exceeded? (2)**	**Notes (3)**
1958	3-,4-,5-,6-day: Yes 7-day: No	ESRD releases are made at all durations except for the 7-day.
1986	Yes	ESRD releases are made at all durations.
1997	Yes	Peak local flow exceeds channel capacity at all durations. In addition, ESRD releases are made at 3-day, 6-day, and 7-day durations.
1998	Yes	Peak local flow exceeds channel capacity at all durations. In addition, ESRD releases are made at all durations.
2006	Yes	Peak local flow exceeds channel capacity at all durations. In addition, ESRD releases are made at all durations.

Table 18. Summary of regulated flows at Bellota for design events scaled to p=0.002 flows using the Bellota frequency curve

Pattern event (1)	**Channel capacity at Bellota exceeded? (2)**	**Notes (3)**
1958	Yes	ESRD releases are made at all durations.
1986	3-,6-,7-day: Yes 4-,5-day: No	ESRD releases are made at the 3-day, 6-day, and 7-day durations.
1997	Yes	Peak local flow exceeds channel capacity at all durations. In addition, ESRD releases are made at the 7-day duration.
1998	Yes	Peak local flow exceeds channel capacity at all durations, In addition, ESRD releases are made at the 3-day, 4-day and 5-day durations.
2006	Yes	Peak local flow exceeds channel capacity at all durations. In addition, ESRD releases are made at the 4-day, 5-day, 6-day, and 7-day durations.

Attachment D. Memorandum of study plan

The following alternative analysis plan was provided to Corps staff June 17, 2011.

New Hogan Reservoir re-operation sensitivity analysis summary

Task 6 and Option Task 1 of our current scope of work calls for completion of initial sensitivity analysis regarding New Hogan Reservoir re-operation alternatives for containing p=0.005 flows at Bellota. The current channel capacity is reported to be 12,500 cfs.

Our scope of work describes the required analysis and the specific questions that we will answer. The scope of work does call for providing a technical memorandum to identify which historical or scaled historical events we will use for the analysis.

Also included in the scope of work is an assessment of how the regulated flow for the selected events would change with an increased channel capacity at Bellota.

In this memorandum, we:

- Describe the reservoir simulation model we will use for the analysis.
- Propose selected historical and scaled historical events for both the re-operation simulations. These are the same events that will be used for Task 6 and Option Task 1.

Reservoir simulation model: HEC-ResSim

For the reservoir simulations, we will use computer program HEC-ResSim. Specifically, we will use the model of New Hogan used in the Lower San Joaquin River (LSJR) feasibility study provided to the Corps in June 2011. This study used Version 3.1 Build 101.

A known computational bug exists in this version regarding the reservoir operation for downstream constraints when Muskingum routing is used. The downstream channel capacity may be exceeded due to this bug, even when there is sufficient storage in the flood pool to contain the event. If we notice this problem in any of the simulations, we will make a note of our findings and inform the SPK technical lead. Further, we will evaluate the simulation results when the channel capacities are exceeded as to if additional flood storage is available in New Hogan Reservoir or not. We will not complete hand routings or use "release overrides" to correct the computer program simulations.

Regarding the application of HEC-ResSim for this analysis, we will:

- For the increased reservoir storage analysis, configure the model to increase the storage by lowering the flood pool. We will simulate selected events through a series of trials to determine the minimum amount of flood storage required to meet the downstream channel capacity of 12,500 cfs at Bellota.

- For all simulations, we will ensure that the current New Hogan Dam outlet works do not limit the release capacity from the dam. (If release capacity is an issue, we will note this.)
- For all simulations, keep the rate of change and ESRD operation rules in the model.
- For all simulations, when the downstream objective flow is exceeded, we will evaluate the simulation and identify the limiting rule or constraint and note this.

Selected historical and scaled historical events

The time series inputs for this analysis will be the same as those used for our June 2011 baseline analysis. This includes both the reservoir inflow and the corresponding local flow between New Hogan and Bellota.

For the event selection, we used the following considerations for selecting events:

- Regulated peak flow close to the p=0.005 peak flow at Bellota from the June 2011 study, which is 16,407 cfs.
- Preference given to events with low scale factors.
- Preference given to events that have local flows developed based on hourly observed values.

Further, in finalizing the selection, we chose at least 3 events for which the local flow at Bellota is less than the channel capacity of 12,500 cfs, and chose events that had showed a ranged of shapes (temporal distribution.)

The 7 events that best matched this above criteria are those shown in Table 19. We will select a minimum of 5 events from this table for use in the analysis. If needed based on the simulations and any errors we find in the reservoir simulations, we may use the remaining events in Table 19.

Proposed increased channel capacity

For Option Task 1, we will simulate the selected historical or scaled historical events for 1 alternative channel capacity at Bellota. Alternative capacities proposed by Dave Peterson, Peterson, Brustad, Inc. in a memorandum to us dated November 29, 2010 are 15,000 cfs, 18,000 cfs, and 21,000 cfs. The increased channel capacity we will use is to be decided upon by the project team. For each simulation, we will report the change in peak release from the reservoir and the peak regulated flow at Bellota.

Table 19. Candidate historical and scaled historical events for analysis

Event (1)	**Scale factor (2)**	**Hourly local flows? (3)**
Criteria	1.0	Yes
1997	2.2	Yes
1958	1.4	No
1986	1.6	Yes
1907	2.2	No
1998	1.6	Yes
1999	1.0	Yes
2006	1.0	Yes

Attachment E. Evaluation of New Hogan re-operation alternative with selected events

Overview

The June 2011 results, as shown in Table 1, show that the 12,500 cfs channel capacity at Bellota is exceeded for the p=0.01 and p=0.005 events. One of the flood risk reduction measures being considered by the study team is the increased flood storage in New Hogan Reservoir. Specifically, the question is how much additional flood storage capacity is needed to contain the p=0.005 peak flow at Bellota within the existing channel capacity.

Volume analysis

Before completing reservoir routings of design (pattern) events, we completed a volume analysis based on the reservoir inflow-frequency curves and the available flood storage in New Hogan Reservoir.

Using the unregulated flow-frequency curve in our June 2011 report, included as Figure 9 in that report, we tabulated the volume for various flow quantiles and durations. Table 20 lists average flows for the p=0.01 1-, 3-, 4-, 5-, and 7-day durations from the frequency curve in column 2. In column 3, we convert the values from column 2 from an average flow for a specified duration to a total volume for the same duration. Table 21 is a similar table, but uses the p=0.005 flows from the frequency curve.

Table 20. Volume analysis for the p=0.01 event using the June 2011 inflow-frequency curve

Duration (days) (1)	Average flow for specified duration and AEP (cfs) (2)	Total volume for specified duration (ac-ft) (3)
1	36,000	71,500
3	24,400	145,000
4	21,100	167,500
5	18,900	187,300
7	16,000	221,700

Table 21. Volume analysis for the p=0.005 event using the June 2011 inflow-frequency curve

Duration (days) (1)	Average flow for specified duration and AEP (cfs) (2)	Total volume for specified duration (ac-ft) (3)
1	40,700	80,700
3	27,700	165,000
4	24,100	191,300
5	21,600	214,600
7	18,400	255,100

The current flood control storage in New Hogan Reservoir is 165,000 ac-ft between November 30 and March 20 per the water control manual (USACE 2004).

Comparing the runoff volumes in column 3 of Table 20**Error! Reference source not found.** and Table 21, we find:

- For durations of approximately 4 days or less for the p=0.01 flows, the entire runoff volume can be stored within the designated flood storage.
- For the durations of approximately 3 days or less for the p=0.005 flows, the entire runoff volume can be stored within the designated flood storage.

Thus, this implies that such an event should be able to be contained within the reservoir with no release. However, in reality, events are longer than 3 to 4 days. Further, the downstream local flows do not fill the entire channel capacity for that long either, thus the reservoir does not need to stop all releases.

Reservoir simulations and alternative analysis

Using the events from Table 2, we evaluated the impact of increased storage. As shown in Table 3, the only event where additional storage would help reduce flooding at Bellota is the 1958 event scaled by 1.4. Simulation results for this event show that with current storage, the flood pool was full, and emergency releases were made which contributed to downstream flooding at Bellota. Only this event was used for this analysis as additional storage would not help reduce downstream peak flows for the other events listed in the table.

Increased storage was simulated by shifting storage from the conservation pool by lowering the flood pool elevation (as opposed to increasing flood storage by raising the dam), such that the p=0.005 flow is within the current channel capacity (reservoir operation control) at Bellota. Thus, given the local flow contribution between New Hogan Dam and Bellota, the New Hogan Dam release for the p=0.005 event would be less than 12,500 cfs, the channel capacity at Bellota. We found the minimum additional storage through an iterative process. For the simulations, we used the same HEC-ResSim model as used in the June 2011 analysis.

Table 22 lists the trial simulations for the 1958 event and Figure 36 shows a plot of simulation results for the existing condition and with the minimal amount of additional storage needed so the channel capacity at Bellota is not exceeded.

Findings

Consistent with the findings from the design (scaled) event simulations described in Attachment C, the local flows tend to be the dominant factor for peak flows exceeding downstream channel capacity. Further, the volume analysis described here shows that the existing flood storage in New Hogan Reservoir is greater than the 3-day p=0.005 flow. However, for the 1958 event scaled by 1.4, an additional 14,000 acre-ft of flood storage would help to control downstream flows to within channel capacity.

Table 22. Trial simulations for 1958 event scaled by 1.4

Simulation (1)	**Elevation of bottom of flood pool (ft) (2)**	**Capacity at bottom of flood pool (ac-ft) (3)**	**Additional flood storage (ac-ft) (4)**	**Peak flow at Bellota (cfs) (5)**	**Peak pool elevation (ft) (6)**
Existing condition	666.16	152,105	None	16,759	713.4
Trial 1	660	135,292	16,813	12,500	712.7
Trial 2	661	137,948	14,157	12,500	713.2

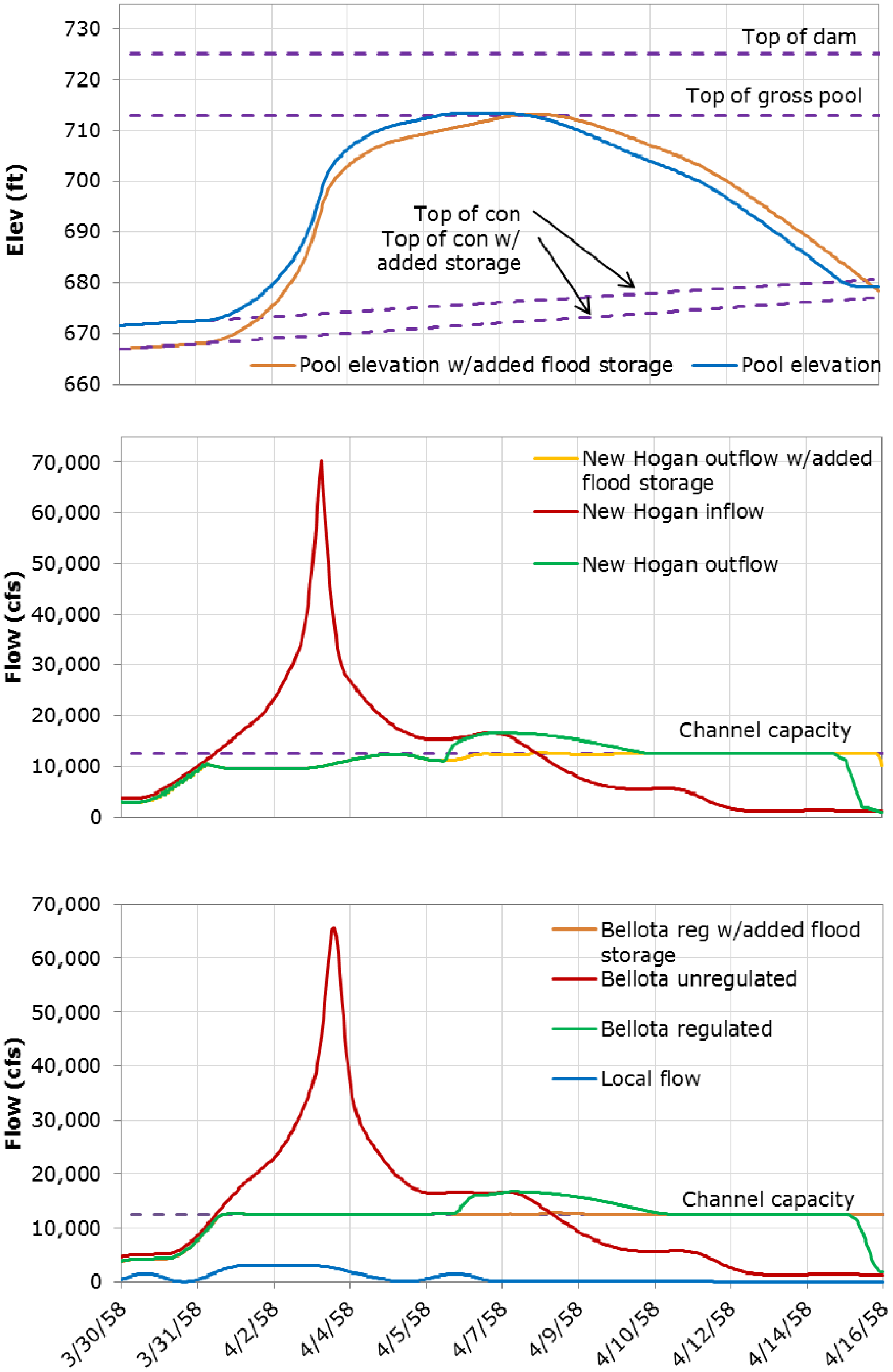


Figure 36. 1958 event scaled to p=0.005, existing condition and with 14,157 ac-ft additional flood storage (created by lowering the top of conservation pool)

Attachment F. Evaluation of channel capacity alternative with selected events

Overview

The June 2011 results, as shown in Table 1, show that the 12,500 cfs channel capacity at Bellota is exceeded for the p=0.01 and p=0.005 events. One of the flood risk reduction measures being considered by the study team is the increased channel capacity. Specifically, the question is how much additional downstream capacity is needed to contain the p=0.005 peak flow.

Analysis

Based on the analysis completed and documented in the previous attachments, we have found that the local flows are the dominant factor in the regulated peak flow-frequency curve at Bellota. For the design (scaled) events using the Bellota unregulated flow-frequency curve, from the June 2011 report, the channel capacity at Bellota was exceeded for the 1997, 1998, and 2006 patterned p=0.005 design (scaled) events. These are shown in Table 12. The channel capacity exceedence for these is due to the local flows, not because of the loss of reservoir flood storage.

To evaluate further these selected design events, we re-simulated the events forcing release to 0 cfs during the period the channel capacity was previously exceeded. These re-simulations are shown in Figure 37, Figure 38, and Figure 39 for the 1997, 1998, and 2006 patterned events respectively. The figures show that the 3-day p=0.005 events can be contained with the current storage in New Hogan and that the flooding at Bellota is due to local flows only. Therefore, the need for increased channel capacity at Bellota is dependent on the local flow-frequency curve.

Findings

Given that for the p=0.005 design (scaled) events, the local flows between New Hogan Reservoir and Bellota drive the peak flow at Bellota, an accepted local flow-frequency curve must be developed and evaluated. Consistent with the guidance in EM 1110-2-1415, as included in Attachment C, the local flow-frequency curve "…is an indicator of the lower limit for the curve of regulated flow."

The limited-use local flow-frequency curve developed herein and included in Attachment B is based on a limited record. Before adoption and acceptance for this purpose, additional analysis is recommended. Further, as part of the LSJR FS, a separate effort is being completed to develop a local flow-frequency curve using rainfall-runoff models and design storms that could also be considered for use for this purpose.

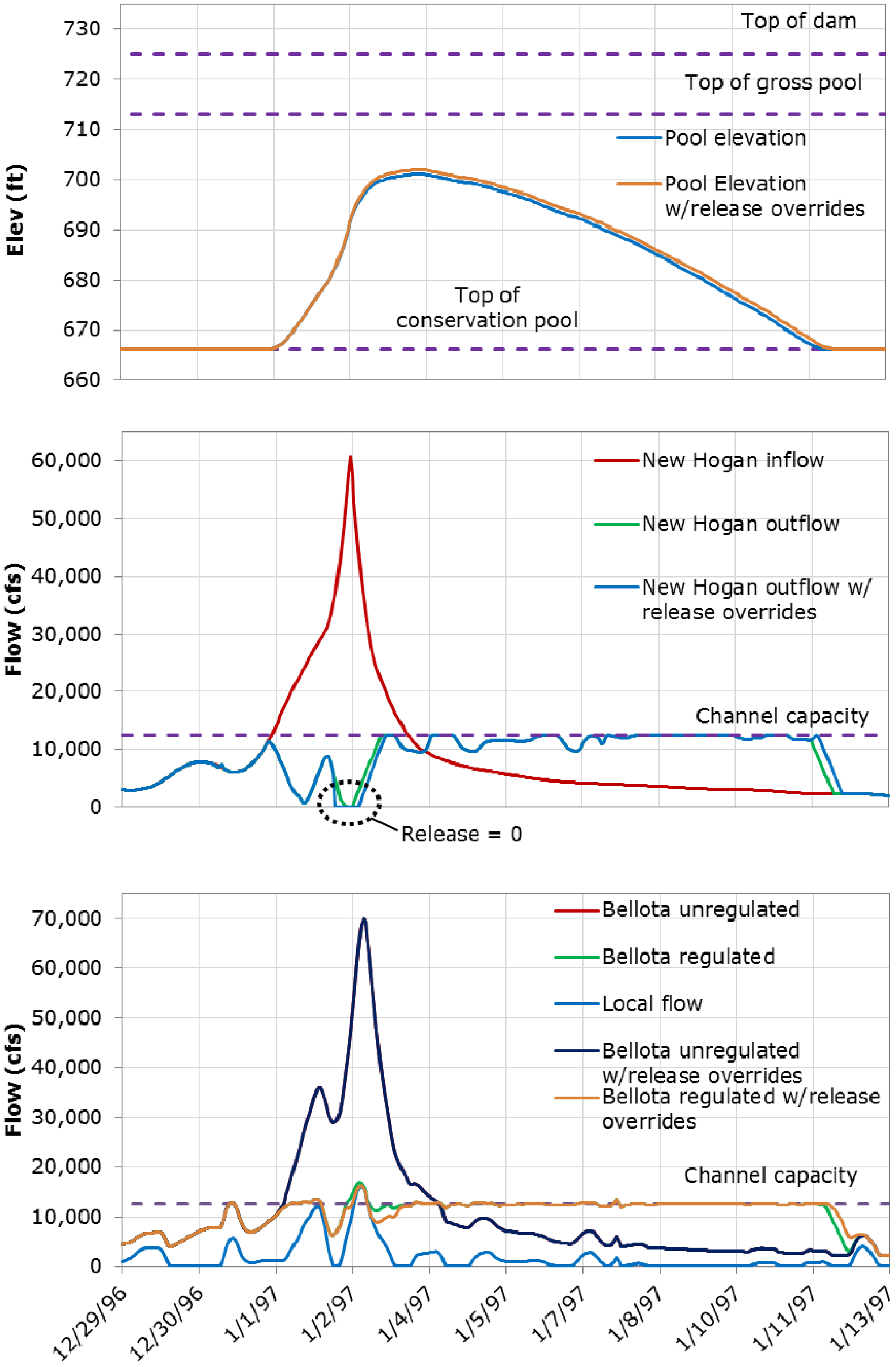


Figure 37. 1997 event scaled to the p=0.005 3-day flow using the Bellota frequency curve; reservoir releases set to 0 cfs during the peak; channel capacity at Bellota is exceeded because of local flows

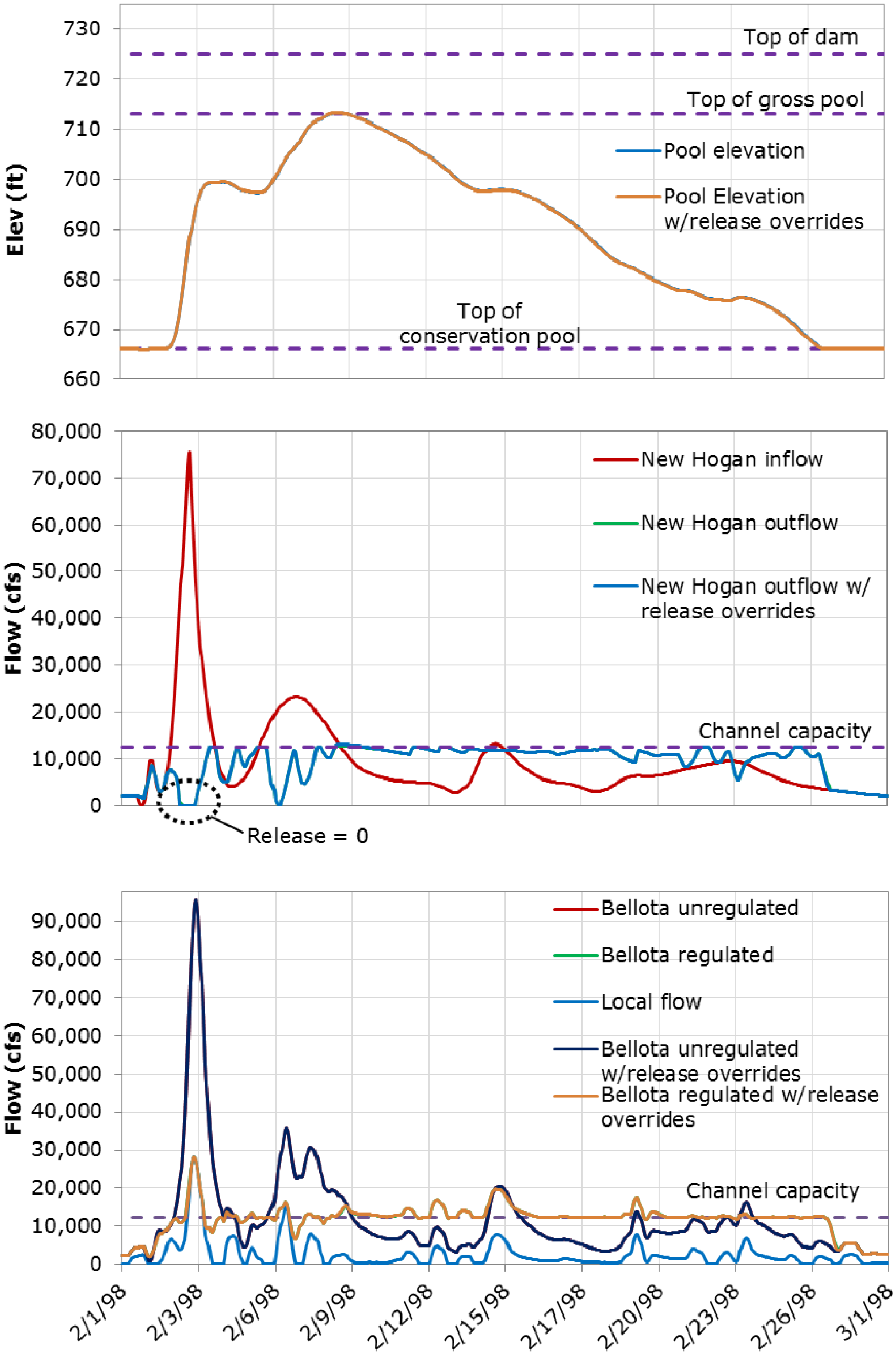


Figure 38. 1998 event scaled to the p=0.005 3-day flow using the Bellota frequency curve; reservoir releases set to 0 cfs during the peak; channel capacity at Bellota is exceeded because of local flows

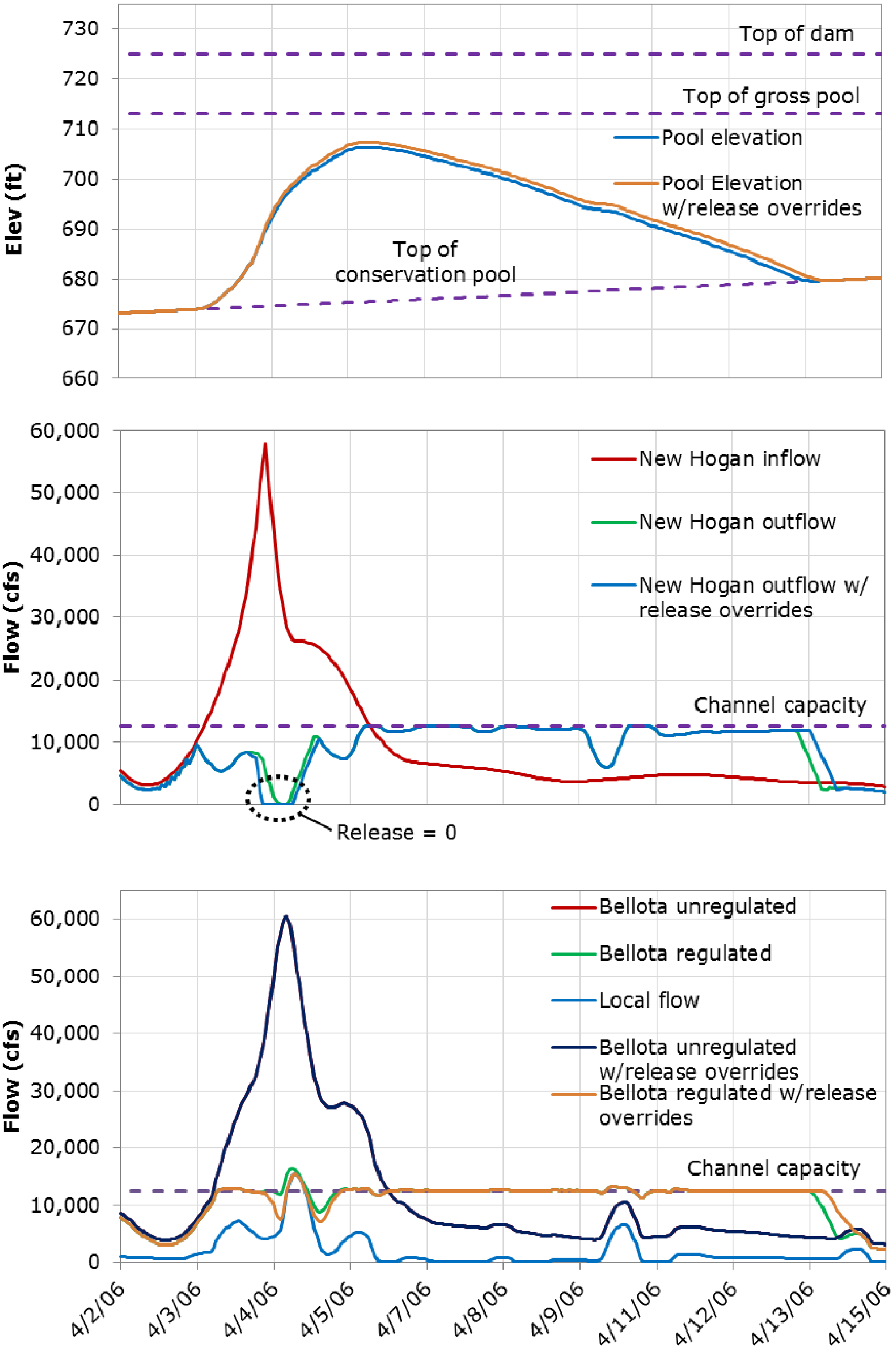


Figure 39. 2006 event scaled to the p=0.005 3-day flow using the Bellota frequency curve; reservoir releases set to 0 cfs during the peak; channel capacity at Bellota is exceeded because of local flows

Attachment G. List of files on CD delivered to the Corps

Table 23 describes the analysis files on the CD delivered to the Corps.

Table 23. Description of files on CD delivered to the Corps

ID (1)	File (2)	Description (3)
1	LSJQMethod2.7zip	HEC-ResSim model and simulations of scaled events
2	LSJQ_Re-opAnalysis.7zip	HEC-ResSim simulations of re-operation analysis
3	NewHogan_re-operation_plan_rev.pdf	Analysis plan
4	Plots.zip	New Hogan and Bellota reservoir routings
5	Scalings.xlsx	Scale factors for all simulations
6	CalaverasRiverLocalFlowFreq.zip	Limited-use local flow-frequency curve input files and program executable

Appendix 2

Lower San Joaquin River Feasibility Study Littlejohn Creek above Farmington, Ca. Hydrologic Analysis

US Army Corps of Engineers.

Sacramento District

23 June 2014

Contents

List of Tables

List of Figures

Attach 1: **Littlejohn Creek frequency analysis and hydrographs, June 23, 2011**

Background

The Corps of Engineers, Sacramento District, Hydrology Section (SPK) tasked David Ford Consulting Engineers, Inc (DFC) with the derivation of unregulated and regulated flow-frequency curves at Littlejohn Creek at Farmington Dam and Littlejohn Creek at Farmington (main control point for Farmington Dam). Their report is titled: "Lower San Joaquin River feasibility study: Littlejohn Creek frequency analysis and hydrographs" dated June 23, 2011. After DFC performed their analysis, revisions were made by SPK in February of 2012. These include 1) a newer version of HEC-ResSim was utilized for flood routing since the version DFC utilized had difficulty maintaining the objective flow releases downstream – mainly due to local flow fluctuations 2) SPK reduced to four rather than nineteen the number of pattern floods used for scaling and routing through Res-Sim. As floods equal to or exceeding the 1% ACE event are the primary focus of alternatives in this study, SPK used only patterns that were representative of rare floods. The parts of the DFC analysis that remain valid and are incorporated into SPK's adopted hydrology are 1) unregulated frequency curve analyses including derivation of local flows during historic events 2) analysis of the critical duration and 3) the peak to volume characteristic curves. The parts of the DFC report that are superseded include 1) adopted unregulated to regulated peak flow transform and final regulated peak flow frequency curves at each index point. The DFC Report is attached to this Appendix and superseded sections have red watermarks labeled as such. The SPK report describes the final adopted hydrology for the feasibility study.

The lower watershed downstream of the Farmington gage was analyzed by Petersen Brustad, Inc (PBI) using a rainfall runoff model. See Appendix 3 for details on that analysis. The various frequency hydrographs developed at the Farmington gage by SPK (as described in this chapter) became boundary condition input to the HMS model of the French Camp Slough produced by PBI. One of the major purposes of the HMS model was to produce concurrent local flow hydrographs for areas downstream of Farmington, during a specific ACE flood event occurring at the Farmington gage.

It should be noted that an unregulated flow frequency curve at Farmington was the foundation for derivation of a regulated flow frequency curve at the Farmington gage control point. As such, the adopted regulated quantile flows are based on many different storm centerings that the gage has encountered during its long period of record.

The study area for the upper Littlejohn Creek watershed is shown in figure 1 below.

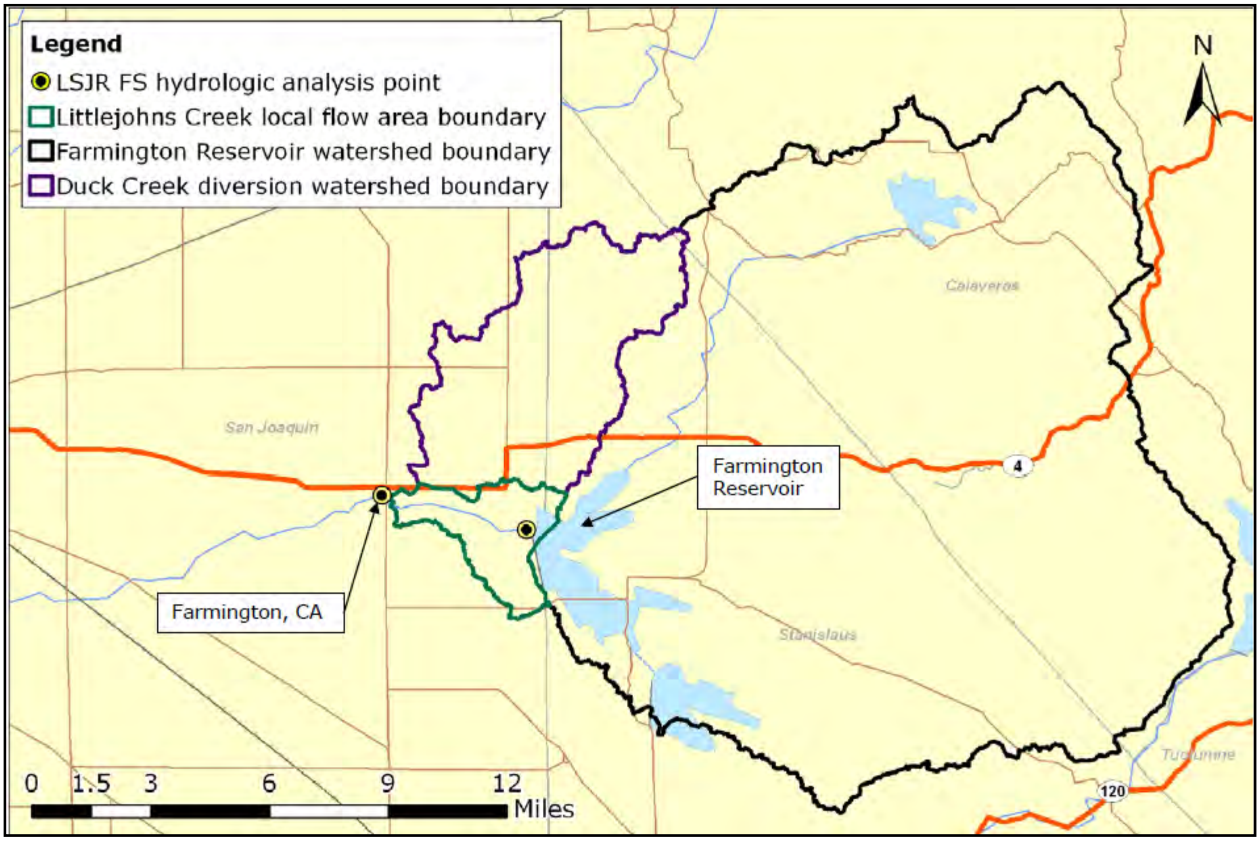


Figure 1. Upper Littlejohn Creek Study area

2.0 Watershed description

The watershed that is the subject of this report—Littlejohn Creek basin—is part of the lower San Joaquin River basin. It is located in Calaveras, San Joaquin, and Stanislaus counties. Located on Littlejohn Creek approximately 20 miles upstream of Stockton, CA, is Farmington Reservoir, a “dry dam” whose primary purpose is flood control.

The principal feature of the watershed is Farmington Reservoir, which drains approximately 212 mi^2. The watershed above the reservoir is wing-shaped and extends 20 miles upstream into the foothills of the western Sierra Nevada. Elevations range from approximately 2,600 ft to approximately 115 ft at the dam.

In addition to runoff from the foothills, Farmington Reservoir receives flows from a diversion on the Stanislaus River at Goodwin Dam, the Stockton East Tunnel, and the Farmington-Stockton East Canal. These flows occur primarily during the summer months and not during the flood season, typically defined as October 1 to May 1 of each water year.

Downstream of Farmington Dam, approximately 3.5 miles, is the Duck Creek Diversion, which diverts flow into Littlejohn Creek from Duck Creek above the town of Farmington. The watershed above the diversion structure on Duck Creek is approximately 28 mi^2. The channel capacity of Duck Creek below the diversion structure is 700 cfs, and the diversion structure itself has a peak capacity of 500 cfs. In addition, the confluence of Littlejohn Creek and Rock Creek is approximately 2 miles downstream of Farmington Dam.

From the town of Farmington, Littlejohn Creek continues west, splitting into the North Fork Littlejohn Creek and South Fork Littlejohn Creek. Flow finally joins French Camp Slough before continuing on to the San Joaquin River. The confluence of Littlejohn Creek and French Camp Slough is located approximately 25 miles downstream of Farmington Dam.

Farmington Reservoir operates to maintain peak flows below the downstream channel capacity of 2,000 cfs near the town of Farmington, including anticipated coincident flows from the Duck Creek Diversion (USACE 2004).

3.0 Procedure for Analysis

The following steps were used to derive hydrographs for Littlejohn Creek at Farmington.

- Develop unregulated flow time series including Farmington Dam inflow and local flow (between dam and the Farmington gage). This analysis was performed by DFC
- Develop 1-, 3-, 7-, 15-, and 30-day unregulated volume-frequency curves at Farmington Dam and Littlejohn Creek at Farmington following the procedures in *Guidelines for determining flood flow frequency, Bulletin 17B* (IACWD 1982), EM 1110-2-1415 (USACE 1993) and using recent USGS regional skew analysis.
- If hourly unregulated flow is not available, convert daily unregulated hydrographs to hourly hydrographs using algorithm which preserves daily volume.
- Input historic and scaled unregulated hourly hydrographs into HEC-ResSim (both reservoir inflow and local flow) to create regulated hourly hydrographs at Farmington gage.
- Perform critical duration analysis at Farmington control point gage to determine volume duration that will be used in unregulated to regulated transform.
- Fit at Farmington gage location, flow transforms to the event maxima datasets identified from the unregulated flow and corresponding simulated regulated time series.
- Developed a regulated flow-frequency curve and associated volumes by applying the flow transforms.
- Developed "expected" outflow hydrographs for Littlejohn Creek at Farmington for 8 flood frequencies: p=0.5, p=0.2, p=0.10, p=0.05, p=0.02, p=0.01, p=0.005 and p=0.002. (Here the term expected hydrograph refers to a hydrograph that has a peak corresponding to the regulated flow frequency curve and associated volumes matching those from the family of characteristic curves corresponding to the given regulated peak flow.)

Figure 2 below illustrates the overall process.

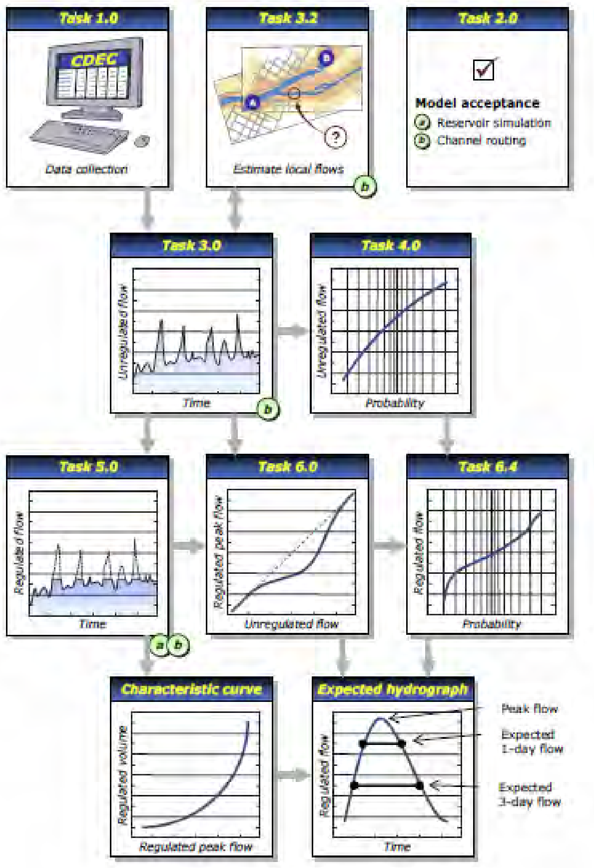


Figure 2: Process Flowchart

4.0 Unregulated flow time series development

SPK's Hydrology Section constructed unregulated flow time series at Farmington Dam (for the Central Valley Hydrology Study) while DFC produced an unregulated times series at Littlejohn Creek at Farmington. DFC used the unregulated times series data provided by SPK for Farmington Dam to construct the downstream control point time series. DFC fitted unregulated volume-frequency curves for both of these locations. Thus, for unregulated conditions, the reservoir inflows were needed. For development of the unregulated flow time series downstream of the reservoir, a routing model was required to simulate the translation, attenuation, and combination of the unregulated flow hydrographs through the system. These flow hydrographs included the upstream boundary conditions (derived reservoir inflows) and intermediate area boundary conditions (estimated local flows). The routing yielded unregulated flow time series that served as the basis of: (1) the unregulated frequency analysis and (2) the unregulated-regulated flow transform. For this analysis, we developed an unregulated flow time series on the Littlejohn Creek by: a) calculating daily unregulated reservoir inflow time series b) developing local flow time series for the area between dam and the reservoir's control point at Farmington d) completing the unregulated flow time series at the Farmington analysis point.

Obtain daily reservoir inflow. The Corps developed the daily unregulated reservoir inflow time series for Farmington Reservoir using the continuity equation, in which, for a given time step, the average inflow equals the outflow plus the change in reservoir storage. For the calculation of these inflows, the source of the observed reservoir outflows and observed changes in storage was the Corps's database. By convention in the Central Valley, these calculations were completed on a 1-day time step, thus midnight to midnight values were used. This is consistent with the work completed for the *Sacramento and San Joaquin river basins comprehensive study* (Comp Study) completed in 2002 (USACE 2002).

Estimate local flow. For Littlejohn Creek, local flows needed to be estimated for the area between Farmington Reservoir and Farmington, CA, shown in **Error! Reference source not found.**1. The estimation approaches we used were:

- Option 1. Direct calculation of local flow using known releases from Farmington Reservoir, known diversions from Duck Creek, and the observed flows at Farmington, CA, routing hourly flows as necessary. In the case of missing streamgage data, local flows values were interpolated as needed.
- Option 2. Estimation of local flows as:

  $$Q_{Local} = 0.04\left(Q_{FRM}\right) \qquad (1)$$

  where Q_{Local} is the local flow estimate for a given time, and Q_{FRM} is the unregulated inflow to Farmington Reservoir. The Corps estimates local flows for the purpose of real-time reservoir operations using this option and this is the option used to estimate local flows in the Comp Study (USACE 2002).

In **Error! Reference source not found.**1 we summarize the selected approaches for local flow estimation on Littlejohn Creek by water year. This flow represents the total local flow contribution at Farmington, CA. Details on the development of the local flow time series on Littlejohn Creek in Attachment 1 to this appendix.

Table 1. Selected local flow estimation approaches for the area on the Littlejohn Creek between Farmington Reservoir and Farmington, Ca

Time period (water year) (1)	**Time step (2)**	**Selected approach[1] (3)**
1951-1968	Daily	Option 1: directly calculate local flow.
1969-1970	Daily	Option 2: 0.04 times reservoir inflow.
1971-1972	Daily	Option 1: directly calculate local flow.
1973	Daily	Option 2: 0.04 times reservoir inflow.
1974-1996	Daily	Option 1: directly calculate local flow.
1996-2008	Hourly	Option 1: directly calculate local flow.

1. The approach listed is the predominant method for estimating local flows over the time period given. See Attachment 1 for further detail.

Complete unregulated flow time series

For the unregulated frequency analysis, DFC used the daily unregulated reservoir inflow time series provided by SPK directly as the unregulated time series corresponding to Farmington Reservoir. For the reservoir's operation point on Littlejohn Creek at Farmington, CA, DFC combined the daily unregulated inflow time series with the estimated local flows by adding the 2 time series together. No routing of the unregulated reservoir inflows was performed because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the reservoir and the operation point is approximately 2 hours, which is less than the 1-day time step of the inflows. In addition, there is little attenuation of flood peaks in this reach because of its length and channel geometry. DFC confirmed this by comparing observed releases from Farmington Reservoir, observed diversions from Duck Creek, and observed flows on Littlejohn Creek at Farmington, CA. The unregulated flow time series at Farmington, CA, does not include diversions from Duck Creek.

5.0 Unregulated frequency analysis

Accepted procedures to develop unregulated flow-frequency curves are specified in *Bulletin 17B* (IACWD 1982). The current standard-of-practice is to fit a Pearson III (LPIII) distribution to the logarithmic transforms of annual maximum series identified from streamgage data. Additional guidance for fitting frequency curves to volumes for a given duration is provided by EM 1110-2-1415 (USACE 1993). For this analysis, DFC used the unregulated inflows to Farmington Dam to develop such an annual maximum series. However, because DFC only had records of regulated flows on Littlejohn Creek at Farmington, DFC could not fit a frequency curve directly using this method. Thus, DFC used the synthesized unregulated flow time series at this location and fitted a volume-frequency curve to that series. For this analysis DFC developed unregulated frequency curves that generally follow procedures specified in *Bulletin 17B* (IACWD 1982) with modification from the EMA procedure. This new procedure is being evaluated by the Bulletin 17C Committee for possible adoption for new federal guidelines for flow frequency. HQ USACE has given districts permission to use EMA. The EMA procedure includes different procedures for handling historic floods and a new outlier detection test called Multiple Grubbs-Beck. In some cases, the Multiple Grubbs-Beck test can result in a larger number of low outliers being censored than the Grubbs-Beck test used in Bulletin 17B.

For each analysis location, DFC:

- Identified the annual maximum series.
- (Task 4.1) Calculated regional skew values for each duration of interest using relationships developed by the USGS.
- (Task 4.2) Fitted LPIII distributions to the annual maximum series using the expected moment algorithm (EMA) enabled flow-frequency software PeakfqSA, version 0.937. This was developed by Tim Cohn of the USGS and is based on the USGS's flow-frequency software PeakFQ (Cohn 2007).

- Reviewed and adopted the curves, checking them for consistency and comparing them to previously accepted values.

Identify annual maximum series

DFC identified the annual maximum series by extracting, from the unregulated flow time series, the volumes associated with the 1-, 3-, 7-, 15-, and 30-day durations. This information is detailed in attachment 1 (see pages 21 and 61). Note DFC developed a peak unregulated flow-frequency curve for Farmington Dam for completeness; however this is not required for this analysis. In addition, DFC did not develop a peak flow-frequency curve for Littlejohn Creek at Farmington because the temporal resolution of the unregulated flow time series, 1 hour to as long as 1 day, is not an appropriate representation of instantaneous unregulated peak flow values.

Calculate regional skew values

For this analysis, DFC calculated regional skew values for the peak flows and 1-, 3-, 7-, 15-, and 30-day volumes using the relationships developed by the USGS (USGS 2010). In these relationships, the regional skew value is a function of the average basin elevation. The values calculated for each analysis location and duration of interest are shown in attachment 1.

Fit frequency curves

To fit frequency curves to the annual maximum series DFC used: (1) the statistics of the logarithmic transforms of unregulated flow time series (mean, standard deviation, and skew), and (2) the regional skew values for the peak flow, and 1-, 3-, 7-, 15-, and 30-day calculated using relationships developed by the USGS (2010). The “at station” statistics were calculated using the EMA option in PeakfqSA. The weighted skew is automatically calculated by the PeakfqSA software used here.

Review and adopt curves

After fitting, DFC reviewed the frequency curves for consistency and appropriateness. Specifically, DFC:

- Compared the curve of a given duration to the curves associated with the other durations at the same analysis location.
- Compared the curves at a given location to the curves at the other analysis location to check for consistency. Figure 13 shows a cfs per mi^2 plot used by DFC to check for consistency. The plot shows results from EMA prior to adjustments and smoothing.
- Compared the curves to those published in the Comp Study. DFC found the frequency curves on Littlejohn Creek were consistent between durations at each location. The curves do not “cross,” and flow quantiles for a given duration at the downstream location are greater than those of the upstream location, as would be expected.

As a comparison, DFC considered the volume-frequency curves developed for Farmington Reservoir in the Comp Study (USACE 2002). The annual maximum series in the Comp Study ended in 1998.

They found that compared to the flow quantiles in the Comprehensive Study, the quantiles of the curves fitted here are: (1) smaller for the 1 day duration, and (2) larger for durations equal 3-days or greater. (Here the only exception is the 3-day p=0.5 quantile which was found to be approximately 9% less than that of the Comp Study. However, they found that the 1-day and 3-day flow quantiles for p=0.01 and p=0.005 annual exceedence probabilities were consistent with those from nearby watersheds on a flow-per-square mile basis. In this analysis, the peak flow-frequency quantiles varied by as much as 9%, as compared to those in the Comp Study, because of (1) the additional 6 events include, 1999 through 2004, and (2) the use of EMA in fitting the curve.

DFC adopted the unregulated frequency curves for the 2 analysis locations, Farmington Reservoir and Farmington, CA, shown in figures 3 and 4. These are the curves that use manually specified low outlier thresholds. The detailed parameters used to fit these curves are included in Attachment 1. The final parameters and statistics used to fit LPIII distributions to develop the unregulated frequency curves at Farmington Reservoir and Littlejohn Creek at Farmington are shown in Table 12 and 3 below. Quantiles values are shown in Figures 14 and 15.

In some cases, the use of a regional skew can result in analytical curves that do not fit the observed data as well as curves that only use a station skew. This is especially true for the unregulated frequency curve shown in Figure 4 (Littlejohn Creek at Farmington, Ca). As can be seen in Table 3, the regional skew is significantly less negative than the station skew for the entire family of curves, which results in the analytical curves rising above (overshooting) the observed data on the upper end. SPK feels the curves at this location are probably conservative in nature and should be modified if an alternative proceeds to PED on Littlejohn Creek. As of the writing of this appendix, no alternatives were economically viable on Littlejohn Creek due to floodplain damages not being high enough to justify the cost of a project. If this issue was corrected, the resulting hydrology would produce smaller floodplains and less damages; therefore, the current hydrology does not adversely impact the feasibility study. In the near future, the Ca DWR Central Valley Hydrology Study will modify the hydrology on Littlejohn Creek because of the unregulated frequency curve having a poor fit and modified hydrology will be available on the website link: < cvhydrology.org >.

Table 1. Unregulated frequency curves parameters and statistics: Farmington Reservoir

Statistic (1)	**Peak flows (2)**	**1-day volumes (3)**	**3-day volumes (4)**	**7-day volumes (5)**	**15-day volumes (6)**	**30-day volumes (7)**
Station mean[1]	3.810	3.301	3.114	2.948	2.733	2.540
Station standard deviation[1]	0.449	0.668	0.661	0.601	0.612	0.615
Station skew[1]	-0.978	-1.410	-1.410	-1.410	-1.410	-1.410
Station skew associated MSE[2]	0.370	0.276	0.275	0.274	0.274	0.273
Regional skew[3]	-0.608	-0.734	-0.690	-0.587	-0.644	-0.632
Regional skew associated AVP[4]	0.140	0.049	0.058	0.049	0.052	0.062
Adopted mean[5]	3.811	3.321	3.135	2.970	2.754	2.561
Standard deviation[5]	0.445	0.610	0.601	0.538	0.553	0.556
Adopted standard deviation	0.445	0.507	0.531	0.538	0.553	0.556
Weighted skew[5,6]	-0.692	-0.858	-0.812	-0.675	-0.733	-0.721
Number of systematic events	34	58	58	58	58	58
Number of high outliers	0	0	0	0	0	0
Number of EMA iterations	2	1	1	1	1	1
Specified low outlier threshold (cfs)	—	282	201	178	105	71
Number of low outliers	0	8	8	8	8	8
Number of zero events	0	0	0	0	0	0
Number of missing events	19	0	0	0	0	0
Number of EMA censored observations	1	8	8	8	8	8
Corresponding censored events[7]	1). 1977	1.) 1977 2.) 1976 3.) 1990 4.) 1989 5.) 1988 6.) 1961 7.) 2003 8.) 1994	1.) 1977 2.) 1976 3.) 1990 4.) 1988 5.) 1989 6.) 1961 7.) 1994 8.) 2003	1.) 1977 2.) 1976 3.) 1989 4.) 1988 5.) 1990 6.) 1961 7.) 1994 8.) 2003	1.) 1977 2.) 1976 3.) 1988 4.) 1989 5.) 1990 6.) 1961 7.) 1994 8.) 2003	1.) 1977 2.) 1976 3.) 1988 4.) 1989 5.) 1990 6.) 1961 7.) 1994 8.) 2003
Record length	53	58	58	58	58	58

Notes:

1. Statistic calculated using the series of logarithmic transforms and EMA without regional skew; rounded to nearest thousandth.
2. Mean square error; rounded to nearest thousandth.
3. Regional skew values calculated using relationships developed by the USGS; rounded to nearest thousandth.
4. Average variance of prediction, analogous to MSE; rounded to nearest thousandth.

5. Statistic calculated using the series of logarithmic transforms and EMA with regional skew; rounded to nearest thousandth.
6. Skew value calculated by weighting the station and regional skew values inversely proportional to their associated errors: (MSE and AVP) and EMA; rounded to nearest thousandth.
7. Events are listed by water year in order of increasing flow or volume.

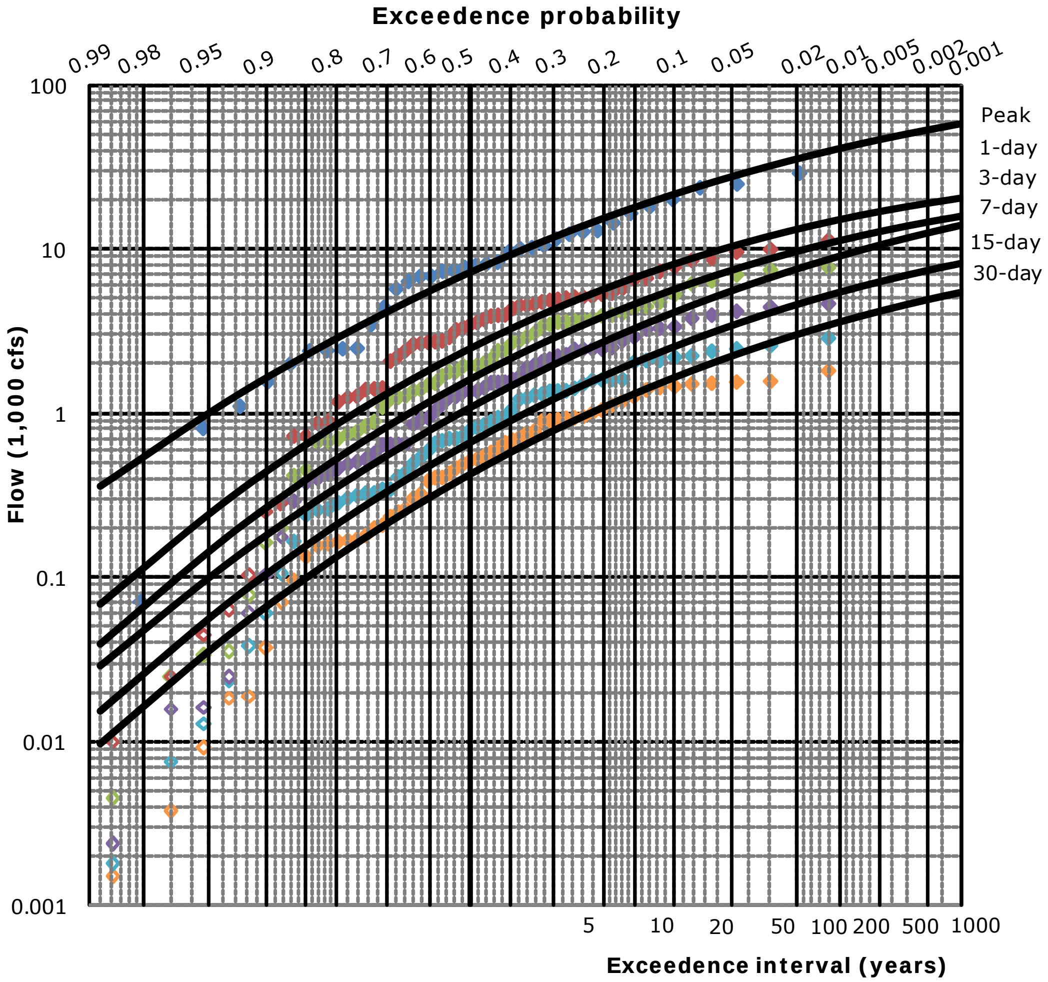
Table 3. Unregulated frequency curves parameters and statistics: Littlejohn Creek at Farmington, CA

Statistic (1)	**1-day volumes (2)**	**3-day volumes (3)**	**7-day volumes (4)**	**15-day volumes (5)**	**30-day volumes (6)**
Station mean[1]	3.339	3.169	2.992	2.797	2.628
Station standard deviation[1]	0.621	0.593	0.579	0.573	0.539
Station skew[1]	-1.410	-1.410	-1.410	-1.410	-1.268
Station skew associated MSE[2]	0.278	0.276	0.276	0.276	0.251
Regional skew[3]	-0.734	-0.690	-0.587	-0.644	-0.632
Regional skew associated AVP[4]	0.049	0.058	0.049	0.052	0.062
Adopted mean[5]	3.356	3.186	3.011	2.815	2.639
Standard deviation[5]	0.573	0.545	0.525	0.523	0.507
Adopted standard deviation	0.573	0.545	0.525	0.523	0.556
Weighted skew[5,6]	-0.849	-0.786	-0.670	-0.722	-0.695
Number of systematic events	58	58	58	58	58
Number of high outliers	0	0	0	0	0
Number of EMA iterations	1	1	1	1	1
Specified low outlier threshold (cfs)	307	254	178	117	82
Number of low outliers	7	7	7	7	7
Number of zero events	0	0	0	0	0
Number of missing events	0	0	0	0	0
Number of EMA censored observations	7	7	7	7	6
Corresponding censored events[7]	1.) 1976 2.) 1977 3.) 1961 4.) 1989 5.) 1990 6.) 1988 7.) 2003	1.) 1976 2.) 1977 3.) 1961 4.) 1989 5.) 1990 6.) 1988 7.) 2003	1.) 1976 2.) 1977 3.) 1961 4.) 1989 5.) 1990 6.) 1988 7.) 2003	1.) 1976 2.) 1961 3.) 1977 4.) 1990 5.) 1989 6.) 1988 7.) 2003	1.) 1961 2.) 1989 3.) 1990 4.) 1977 5.) 1989 6.) 2003
Record length	58	58	58	58	58

Notes:

1. Statistic calculated using the series of logarithmic transforms and EMA without regional skew; rounded to nearest thousandth.
2. Mean square error; rounded to nearest thousandth.

3. Regional skew values calculated using relationships developed by the USGS; rounded to nearest thousandth.
4. Average variance of prediction, analogous to MSE; rounded to nearest thousandth.
5. Statistic calculated using the series of logarithmic transforms and EMA with regional skew; rounded to nearest thousandth.
6. Skew value calculated by weighting the station and regional skew values inversely proportional to their associated errors: (MSE and AVP) and EMA; rounded to nearest thousandth.



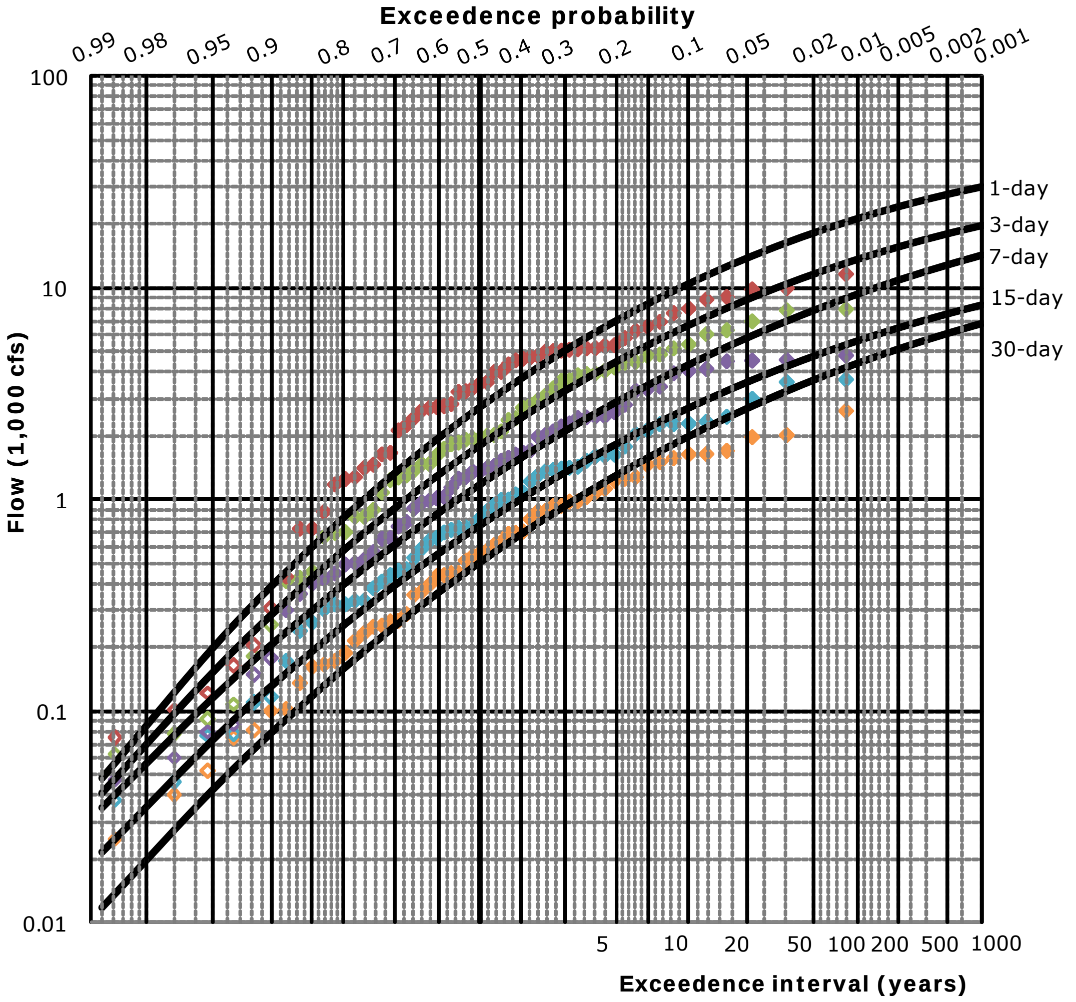
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.811	0.445	-0.692
1-day	3.321	0.507	-0.858
3-day	3.135	0.531	-0.812
7-day	2.970	0.538	-0.675
15-day	2.754	0.553	-0.733
30-day	2.561	0.556	-0.721

Notes:

- Median plotting positions.
- Drainage area: 212 sq. miles.
- Period of systematic record: 1951-2008.
  (Peak flow data intermittent 1952-2004).
- Record lengths
  Peak flows: 53 years.
  Volumes: 58 years.
- Regional skew values developed by USGS.
- Low outliers for volumes: 8 smallest events.
- Hollow points are censored events.

Figure 3: Littlejohn Creek at Farmington Dam Unregulated Flow Frequency Curves



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.356	0.573	-0.849
3-day	3.186	0.545	-0.786
7-day	3.011	0.525	-0.670
15-day	2.815	0.523	-0.722
30-day	2.639	0.556	-0.695

Notes:

- Median plotting positions.
- Drainage area: 219 sq. miles.
- Period of systematic record: 1951-2008.
- Record length: 58 years.
- Regional skew values developed by USGS.
- Low outliers for 1-, 3, 7, and 15-day volumes: 7 smallest events.
- Low outliers for 30-day volumes: 6 smallest events.
- Hollow points are censored events.

Figure 4: Littlejohn Creek at Farmington, Ca Unregulated Flow Frequency Curves

Smooth unregulated flow time series. The daily unregulated flow time series are appropriate for frequency analysis. However daily upstream and intermediate boundary conditions do not have the temporal resolution required by the CVHS procedures for assessing the effects of regulation, particularly releases as indicated on the emergency spillway release diagram (ESRD). Therefore, the daily reservoir inflows and daily estimated local flows were "smoothed" to hourly time series for input into HEC-ResSim by SPK staff. This smoothing was completed using a mass balance algorithm that interpolates the shape of the hydrograph and estimates peak hourly flows while maintaining daily volumes consistent with the original time series.

6.0 Regulated flow time series development

As mentioned before, SPK developed the adopted regulated times series for this study. To develop regulated flow-frequency curves, the unregulated volume duration- frequency curves are transformed through the unregulated- regulated flow transform. The unregulated-regulated flow transform captures the system's response to large, varied events, and is created using the unregulated and regulated flow time series data.

SPK simulated the 1956, 1958, 1986, and 1998 events with HEC-ResSim version 3.1.8 RC4. This version corrects defects in the downstream rule logic. The choice of events was made predominately by choosing the highest floods of record. The 2006 event (and all other smaller events) did not scale high enough to aid in definition of the 0.002 AEP flow transform and was removed from the analysis. The transform was extended to the 0.002 AEP event by linear extrapolation. The largest floods for Littlejohn Creek at Farmington, Ca is shown in Table 4 below in terms of the unregulated 1-day and 7-day maximum annual flows. As indicated below, 10-days was determined to be the critical duration for the control point below Farmington Dam. To create transforms, one must first perform a critical duration analysis.

Determine critical duration. DFC performed a critical duration analysis at two locations. Details on this analysis can be viewed in Attachment 1 (see page 76). In their analysis DFC identified the duration of the unregulated annual maximum series that consistently estimates the largest flow for each probability. In selecting the critical duration, they considered both the "goodness of fit" of each transform and which duration estimates the greater peak regulated flows. From their analysis, they determined that the critical duration at Farmington Dam and Littlejohn Creek at Farmington, Ca to be 10 days. Thus, the appropriate unregulated-regulated flow transforms used in this analysis were associated with these durations.

Water year	**1-day unregulated flow (cfs)**	**Water year**	**7-day unregulated flow (cfs)**
1998	11,270	**1998**	4,630
2006	9,910	**1986**	4,420
1986	9,560	**1965**	4,160
1965	8,760	**1958**	3,950
1956	8,500	**1956**	3,770
1958	7,270	**2006**	3,350

Table 4: Largest floods at Littlejohn Creek at Farmington

Reservoir Regulation Simulation Criteria

SPK's Hydrology Section performed the final reservoir simulations in HEC-ResSim (version 3.1.8 RC4). Only four pattern floods were used to develop the transforms in this analysis as opposed to the DFC analysis which used many additional patterns. As rare floods are of primary interest in this study, SPK determined that only the rarest flood patterns should be used for reservoir routing as they are the most representative of these types of events including the local flow runoff characteristics.

The HEC-ResSim model was developed as part of the Central Valley Hydrology Study. An Agency Technical Review (ATR) was performed by a retired annuitant working at HEC (Dan Barcellos). The model was setup to follow the rules in the latest approved Water Control Diagram.

Starting storage assumption: Starting storage is assumed to be bottom of flood control as defined in the Water Control Diagram. For each event modeled, 45 days of scaled historic inflow (including pre- and post-waves around the main flood wave) were ran for each simulation. One consistent ratio was applied to all ordinates of the historically based 45 day inflow hydrograph pattern. The purpose of the longer simulation was to partially compensate for the starting storage assumption, i.e. measure the impact of multiple waves of inflow to the dam over time upon its operation. Figure 5 shows the Farmington Dam storage at the beginning of the 1997 flood event.

Adjustments for common floods: For the more common events, the antecedent storage condition might have the reservoir below bottom of flood control. In other words, there is water supply space available to absorb the inflow volume during an event. Another factor is that reservoir mangers have a history of making releases at less than objective flow rates if forecasts indicate the event will be small. To compensate for these realities, SPK's Hydrology Section produced a graphical peak flow frequency curve at the Farmington, Ca gage for the period after the dam was built. The gage record for this period includes both reservoir outflow and local flow. For probabilities of 0.5 to 0.02 ACE, the adopted regulated n-year hydrographs were adjusted to match the graphical peak curve based on historic data. Adjusting the hydrograph to match historic data for common events compensates for our starting storage assumptions, and for the decisions water managers make during these types of events.

Seasonal floods: The scaled events keep their historic time stamp in the dssfile when input into HEC-ResSim. The 1958 flood occurred in early April (maximum 1-day flow occurred April 3rd). The ResSim model has a smaller amount of flood space at this time of year due to the seasonality of the rule curve in the Water Control Diagram. As such, it turned out the 1958 flood pattern was the most difficult for the ResSim model to control for the 0.01 ACE and more rare floods. The probability assigned to the scaled 1958 floods came from the 10-day rainflood frequency curve which includes December through March flood events. This is a conservative way of estimating the probability of a specific flood occurring in spring. The true probability of such a flood occurring in April is best evaluated by performing a seasonal flow frequency analysis, which undoubtedly would assign it a more rare frequency than our current method. In hindsight, if SPK conducted this study a second time, it should take this into consideration. Since the median transform was used to define the adopted regulated frequency curve for the 0.01 ACE frequency and more rare events, the current use of the 1958 flood pattern did not adversely impact the outcome of the analysis. This is because the 1958 transform fell on the high side of the four transforms for these frequency events.

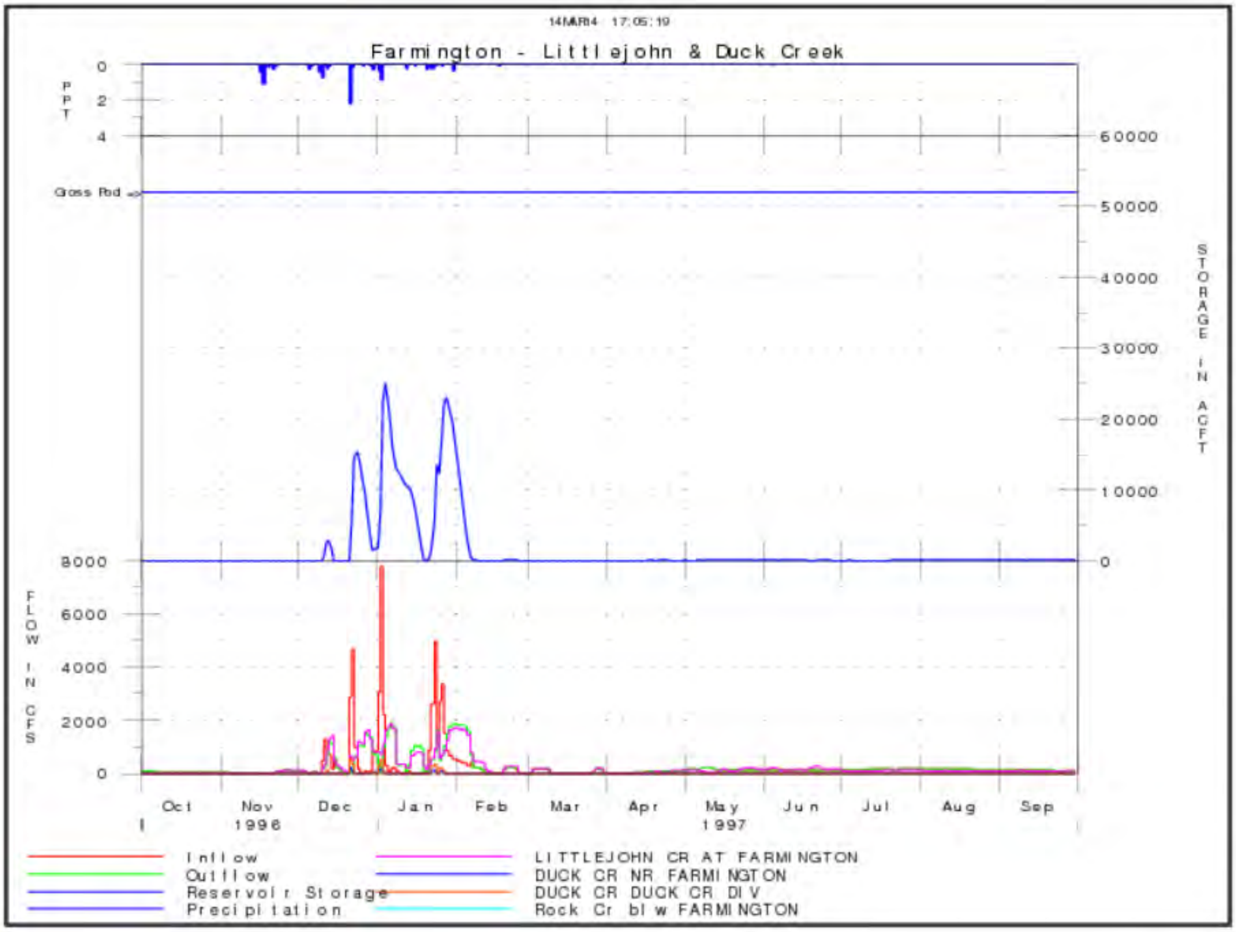


Figure 6: Storage at Farmington Dam at start of 1997 flood event

Selection of Pattern Floods Used in ResSim Routings. The main focus of this feasibility study is to provide urban areas like Stockton flood protection from rare floods. Many tributaries studied in this feasibility study currently have levees that were originally designed to provide protection from the 0.01 ACE event. The sponsors have a keen interest to achieve protection from the 0.005 ACE event. As such, SPK chose to pick some of the rarest historic events as a template for modeling alternatives in this watershed. The rarer flood patterns should also provide a better estimate of the local flow runoff that the reservoir will have to deal with when a really rare event occurs. Within the 58 years of recorded flow, the highest four ranking floods (ranked largest to smallest using the 1-day and 7-day unregulated volumes) are shown in Table 4 above. The flood patterns used for the reservoir routings were the 1956, 1958, 1986, and 1998 events.

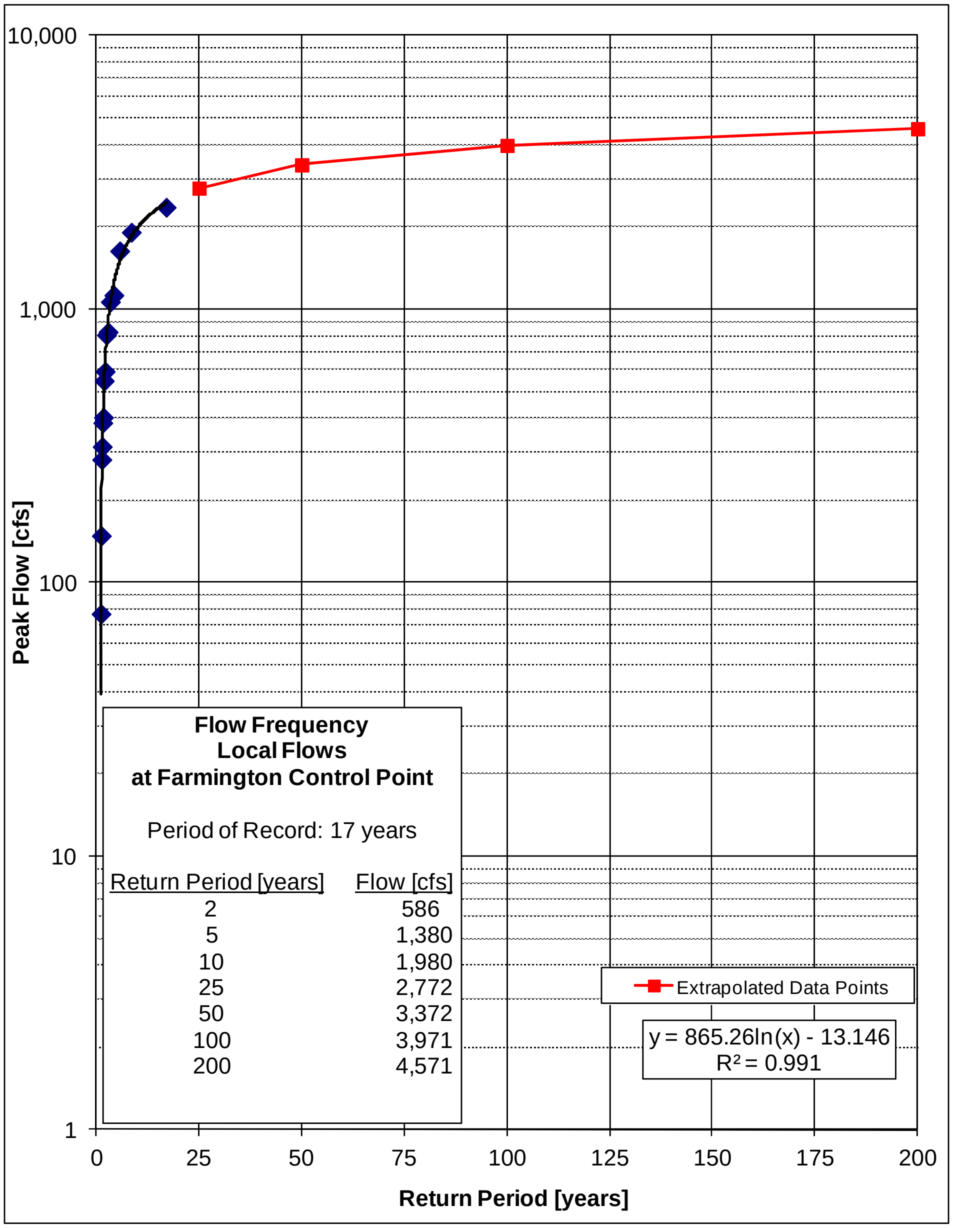
In summary, since rare floods like the 0.005 ACE event are important for the evaluation of alternatives in this feasibility study, the rarest events were selected as pattern floods to scale and route through HEC-ResSim. The local flow that occurred during these large events is considered the best representation of what might happen in a flood of this magnitude.

Validating the Transform: USACE guidance indicates that a local flow frequency curve should be developed to determine the lower boundary of a regulated frequency curve developed from an unregulated to regulated transform based on reservoir routings. Theoretically, the transform can exceed the local flow frequency curve but should not fall below it. This is due to the fact that the local flow cannot be controlled and therefore will always impact an analysis point. Local flow does not include reservoir releases.

Since 58 years of recorded regulated flows (includes both local flow and reservoir releases) are available at Littlejohn Creek at Farmington, a graphical frequency curve based on plotting positions was used to determine the 0.50 through 0.02 ACE frequencies for this location.

Estimation of local flow is more important for rare floods like the 0.01 and 0.005 ACE events for which there is significant uncertainty and for which an unregulated to regulated transform must be created. For this effort, PBI developed a calibrated HEC-HMS rainfall runoff model. The model was calibrated for the area between Farmington Dam and the Littlejohn Creek at Farmington, Ca stream gage. After calibrating the model, PBI input various design storms into the model to estimate the local flow peak instantaneous values for various frequencies. Results of the analysis are shown in Figure 6 below.

The unregulated to regulated transform for Littlejohn Creek at Farmington, Ca determined a peak regulated flow of 9900 cfs and 12,900 cfs respectively, for the 0.01 and 0.005 ACE events. This is well above the local flow frequency curve produced by PBI which helps validate the transform per USACE guidance.



Return Period [years]	Flow [cfs]
2	586
5	1,380
10	1,980
25	2,772
50	3,372
100	3,971
200	4,571

Figure 6. Local flow frequency curve at Farmington, Ca by PBI

Note: Local flow does not include reservoir releases.

7.0 Create Littlejohn Creek at Farmington, Ca Hydrographs For Specific Frequencies

The following steps were performed to extract an outflow hydrograph for each “n-year” event corresponding to the regulated flow-frequency curve for Littlejohn Creek at Farmington, Ca.

1. Simulate the 1956, 1958, 1986, and 1998 events with HEC-ResSim version 3.1.8 RC4. This version corrects issues in the downstream rule logic of the version used by DFC. Perform

simulations to develop regulated flow time series for scale factors from 1.0 to 3.0 of reservoir inflow and local flow, which are input to the simulation model. The four events were chosen out of a list of the highest floods of record.

2. Extract the 1-day unregulated flow volume and regulated peak flow at Farmington, Ca from the DSS files output from simulations in step 1. The 1-day unregulated flow volume was identified as the "critical duration" by DFC in Attachment 1 (see page 76) for the .01 to 0.005 ACE events. So, the independent variable (x-axis) of the flow-flow transform is the 1-day unregulated flow, with the peak regulated flow being the dependent (y-axis) value. Then use a spreadsheet to input the 1-day unregulated flow and peak regulated flow data pairs to compute the transform for each pattern. SPK's Hydrology Section decided to adopt the median transform to develop a regulated peak flow frequency curve. To compute the median curve, an average regulated peak flow value (y-axis) is computed for each x value from the two innermost transforms (note: we developed four transforms). Figure 7 displays the four individual event based transforms plus the average and median transforms for the Farmington gage location. Table 5 displays individual values from the average and median transforms. The median transform was adopted for the study.

3. The regulated hydrographs for the 0.5 to 0.02 ACE flows at Littlejohn Creek at Farmington, Ca were *revised to fit observed conditions at the Farmington gage* via a family of graphical curves using 58 years of historic data. It is noted that using this approach may limit the ability of the District to evaluate alternatives involving reservoir reoperation or reconfiguration. This is because it is not possible to generate equivalent graphical frequency curves for with-project conditions. Currently, reservoir reoperation is not one of the alternatives being moved forward in the analysis. The methodology described above uses the HEC-ResSim program, with unimpaired inflow data input to the reservoir and local flow areas, with operational rules documented in the Water Control Manuals. This provides a consistent reservoir operation that follows the Congressionally authorized plan of operation. In actual operations as shown by the historically observed flows, the reservoir was operated differently. That is, for smaller, frequent events, the reservoir was not drawn down as quickly as the water control plan suggests, but holds runoff in storage longer while making smaller, lower, releases. Figure 8 shows the actual operation for the January 1997 flood, while Figure 9 shows the hypothetical operations (note: the inflow hydrograph for the hypothetical simulation is derived from daily inflow values smoothed into hourly values using an algorithm which preserves the historic daily volume). Besides modifying the peak of the hydrograph for these frequency events, the volume was also modified to match a graphical frequency analysis of historically observed flows. The runoff volume was found by computing the 1, 3, 7, and 15-day flow volumes from historic daily regulated flow time series at Farmington, and then extracting annual maximums and computing the plotting positions of the resulting annual maximums, then interpolating the 0.5 to 0.02 ACE flow magnitudes. The derived values are shown in Table 6 below.

   a. For the target frequency, select a 1997 pattern hydrograph with the scale factor that provides the proper unregulated volume based on critical duration (10-day for Farmington, Ca control point) unregulated frequency curve.
   b. Based on the scale factor chosen in (a) above, obtain the corresponding Res-Sim output hydrograph at Farmington, Ca.

   c. For the target frequency, find the appropriate peak flow and volumes based on Table 6.
   d. Input the regulated hydrographs found in step b and the peak and volumes found in step c into HyBART in order to balance/adjust the hydrograph.

4. For the 0.01 to 0.002 ACE events, regulated peak flows were derived by the unregulated to regulated transform method show in Figure 7. The procedure to derive final regulated hydrographs is described below.

   a. For the target frequency, select a 1997 pattern hydrograph with the scale factor that provides the proper unregulated volume based on critical duration (10-day for Farmington, Ca control point) unregulated frequency curve.
   b. Based on the scale factor chosen in (a) above, obtain the corresponding Res-Sim output hydrograph at Farmington, Ca.
   c. For the target frequency, find the appropriate peak flow (from the transform in Figure 7) and the concurrent volumes based on the DFC peak to volume regression analyses. DFC analyzed regulated peak flow to volume relationships from a regression analysis using multiple pattern events. The analysis was based on routing scaled historic flood patterns through Res-Sim and analyzing the resulting regulated flow hydrographs to obtain matching peak and volume data pairs. The data pairs were then used in a regression analyses, with peak being the known value x and volume being the prediction value y. Relationships were derived by DFC for regulated peak to regulated 1-, 3-, 7-, 15-, and 30-day volumes. The DFC analysis can be viewed in attachment 1 (see page 84).
   d. Input the regulated hydrographs found in step b and the peak and volumes found in step c into HyBART in order to balance/adjust the hydrograph.
   e. Create plot similar to the one shown in Figure 10 based on all hydrographs produced in HyBART including the 0.5 to 0.02 ACE events. Ensure consistency between all frequencies so that the lines do not cross each other. The final adopted peak and volumes are plotted in Figure 10. Note: The 0.5 to 0.02 frequency hydrographs remain consistent with the family of graphical curves based on 58 years of data while the 0.01 through 0.005 ACE event hydrographs generally follow the DFC peak to volume relationships.

In summary, Table 9 displays the final adopted regulated peak and volumes for each frequency event. Table 9 values were input to the program HyBART, a hydrograph balancing routine, along with pattern hydrographs from Res-Sim simulations of the 1997 flood. Simulated patterns were used rather than the actual observed pattern as the simulated and observed patterns are significantly different. The program HyBART creates balanced hydrographs that match the regulated peak flows and volumes in table 9 and follow the pattern of the 1997 flood event. HyBART creates a balanced hydrograph using all input peak flows and volumes. The Res-Sim model output hydrograph most closely associated with a specific frequency (based on critical duration) was selected as the input hydrograph for HyBART to achieve the same frequency balanced hydrograph. The 1997 flood event pattern hydrographs for scale factors of the observed flood of from 1.0 to 2.6 are shown in figure 11.

> The resulting regulated flow hydrographs for the 0.5 annual chance exceedance probability (ACE) to 0.002 ACE events are consolidated in the spreadsheet: MSB-RegFlowFreq-1997SimPattern-Hydrographs.xlsx. A plot of the balanced regulated flows is shown below in figure 12. The hydrographs in figure 12 were eventually provided to PBI to route through the HEC-HMS model to compute additional hydrographs for index points downstream of Farmington. The HMS model used a 1997 pattern storm to compute concurrent local runoff from sub-basins located downstream of the Farmington.

The 1997 event was chosen as the one event for producing specific frequency floods for the following reasons: a) It was a recent event in which hourly **hyetograph patterns** were available b) The various frequency hydrographs produced in this analysis became input to the HMS model produced by PBI, wherein the rainfall runoff model produced concurrent runoff for areas downstream of the Farmington gage. c) In order to synchronize the two efforts, the same flood event (1997 flood) needed to be modeled in order for the timing of the total watershed runoff to be consistent with a real event.

8.0 Risk Analysis

USACE policy is to use risk analysis as part of its planning and design processes. SPK's Hydrology Section is assigned the task of providing hydrologic risk parameters for use in the Flood Damage Analysis (FDA) program. One of the most important of these is the assignment of a period of record for study index points. This section provides some guiding thoughts on that parameter. For the analysis, the assigned period of record for ability of nature, a human operator would be reticent to assume that rule is foolproof. As such a human operator would probably release less than the reservoir model, which would have the impact of filling up the reservoir storage faster. Under these circumstances, the reservoir would provide a lower level of protection from extremely rare floods since the downstream channel is being used less efficiently.

Another factor in this discussion is the method in which both reservoir inflow and local flow are scaled by the same factor for routing through the HEC-ResSim model. From experience with the Central Valley Hydrology Study, SPK has learned that scaling reservoir inflow and local flow by the same factor can sometimes result in a conservative estimate of local flow. The standard deviation and skew of reservoir inflow frequency curve and the local flow frequency curve are normally quite different. Typically, the local flow frequency curve flattens out at the upper end faster than the reservoir inflow frequency curve. This is because the upper watershed's runoff is driven by higher rainfall in the mountains due to orographics which can results in a higher standard deviation (slope of the curve). Scaling the local flow hydrograph and the reservoir inflow hydrograph by the same factor can result in local flow becoming increasingly rare in relation the reservoir inflow frequency. For example, scaling a specific flood by a factor of 1.8 (that originally had 0.04 reservoir inflow frequency and 0.10 local flow frequency) might result in a reservoir inflow and coincident local flow that are both equivalent to a 0.01 ACE event. This changes the dynamics of rare floods as opposed to what really happens in nature, and is probably not typical. SPK feels this method can result in conservative estimates of local flow runoff.

The two issues above may have a cancelling effect, one being less conservative and one being too conservative. Further sensitivity analyses or refinement of the hydrology could be done in PED phase to assess the above concerns. For the feasibility study, it is currently recommended that the period of record assigned to the Mormon Slough at Bellota gage in the FDA program be 50 years (as opposed to the unregulated frequency curve period of record of 104 years at this location).

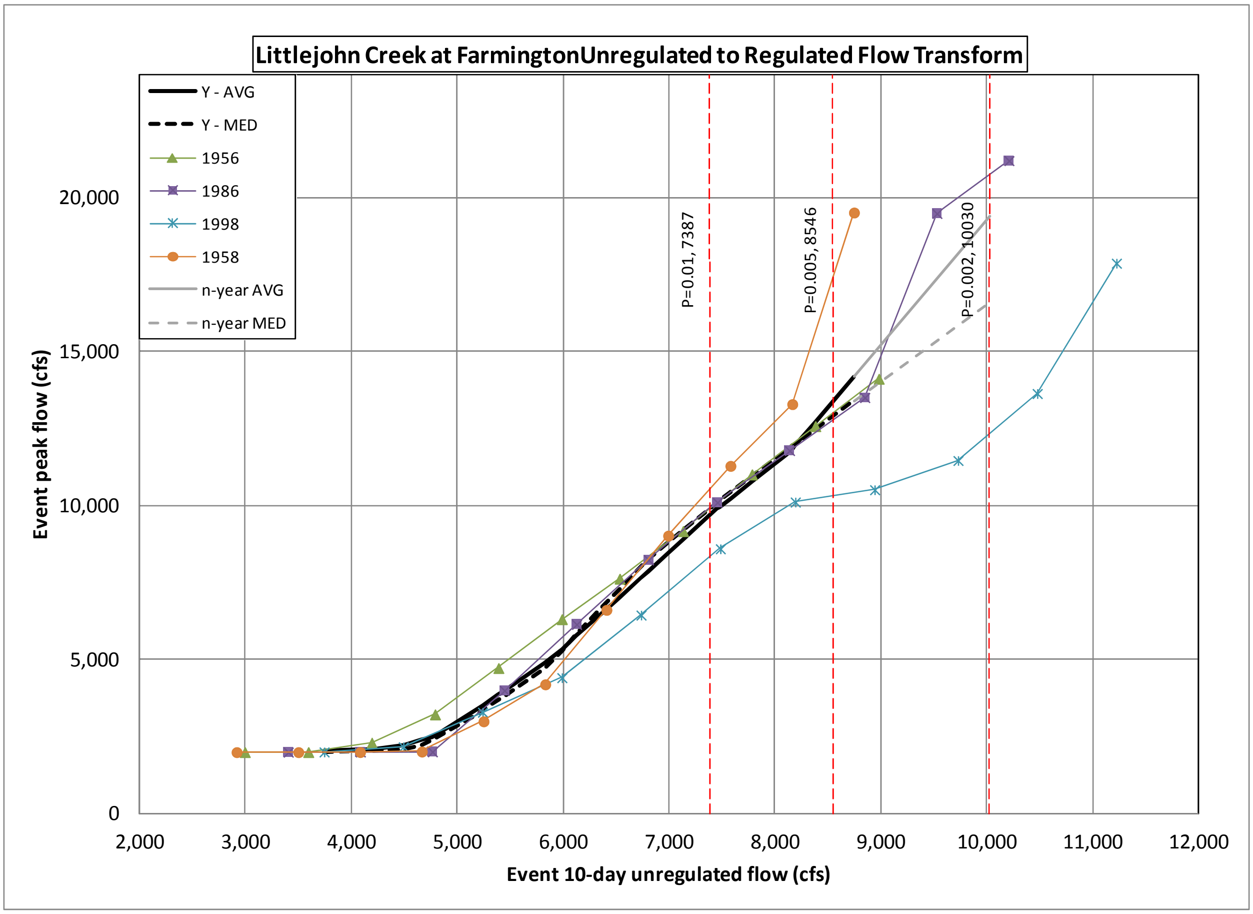


Figure 7. Unregulated 10-Day Flow to Regulated Peak Flow Transform at Farmington, Ca

N-probability Events			
AEP	**Unregulated cfs**	**AVG transform**	**MEDIAN transform**
0.01	7,387	9,672	9,905
0.005	8,546	13,482	12,894
0.002	10,030	19,960	16,598

Table 5: 10-day Unregulated Flow and Regulated Peak Flow Comparison at Farmington, Ca

Note: The median transform for the 0.01 – 0.002 AEP events was chosen for use as it appears to represent a better fit to the data. This table has been truncated as the values from table 6 shown below will be used for the 0.5 to 0.02 AEP events.

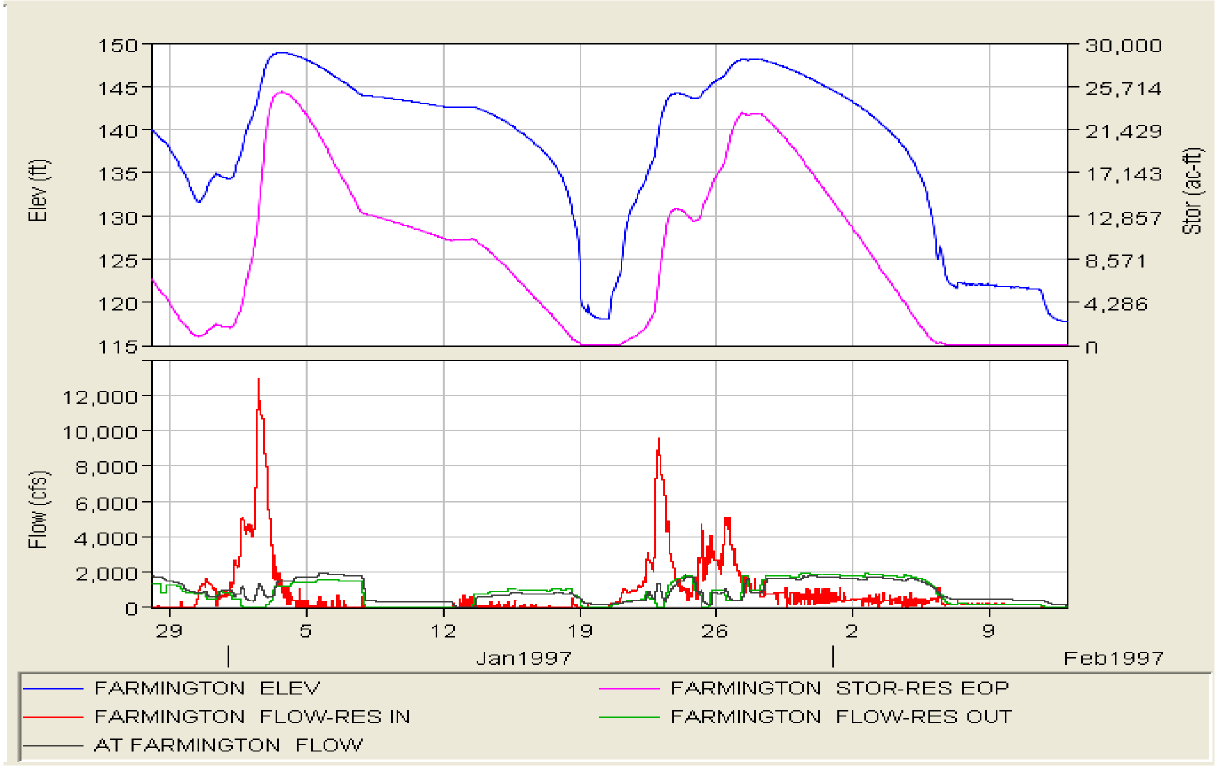


Figure 8. Actual operation of Farmington dam during the 1997 flood event.

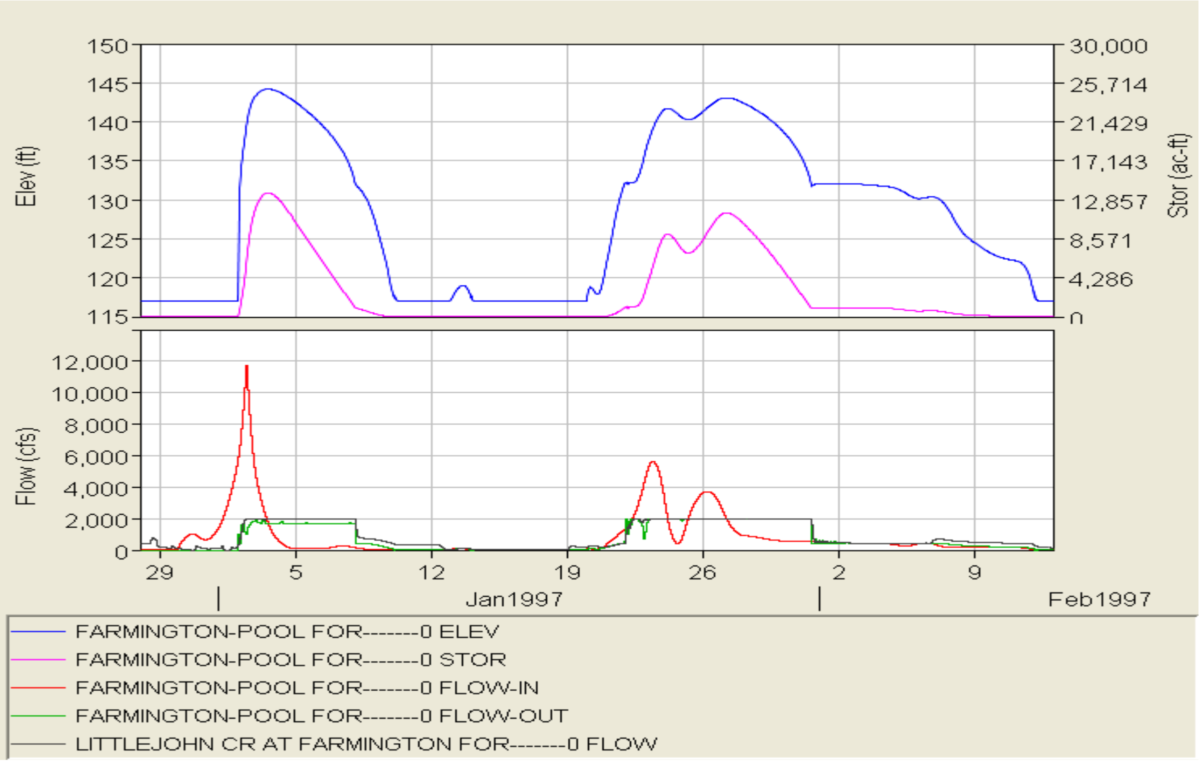


Figure 9. Simulated operation of Farmington dam for the 1997 flood event.

Note: The inflow for the simulated operation is different than the inflow shown in Figure 8 because the reservoir inflow for Figure 9 was produced by an algorithm that smooths daily flows into hourly flows while preserving the historic daily volume.

Farmington 1Day Annual Maximums

		Peak	1-day	3-day	7-day	15-day	
No.	Prob	Y-Axis	Y-Axis	Y-Axis	Y-Axis	Y-Axis	1/Prob
1	0.98438	43	37	25	17	10	1.016
2	0.96875	71	62	34	25	25	1.032
3	0.95313	86	75	63	48	43	1.049
4	0.93750	145	126	93	77	69	1.067
5	0.92188	189	156	109	83	74	1.085
6	0.90625	236	164	146	122	102	1.103
7	0.89063	239	205	183	142	106	1.123
8	0.87500	346	237	216	145	115	1.143
9	0.85938	357	301	245	187	118	1.164
10	0.84375	420	321	249	194	128	1.185
11	0.82813	479	365	353	257	141	1.208
12	0.81250	536	416	404	309	171	1.231
13	0.79688	555	466	418	327	177	1.255
14	0.78125	557	523	460	337	186	1.280
15	0.76563	602	537	484	345	222	1.306
16	0.75000	739	573	525	353	240	1.333
17	0.73438	795	642	604	355	247	1.362
18	0.71875	811	691	627	372	249	1.391
19	0.70313	958	715	644	461	296	1.422
20	0.68750	968	758	676	469	329	1.455
21	0.67188	974	786	682	500	330	1.488
22	0.65625	978	841	685	501	356	1.524
23	0.64063	1,043	850	709	503	360	1.561
24	0.62500	1,060	921	755	514	384	1.600
25	0.60938	1,103	929	764	591	390	1.641
26	0.59375	1,179	959	834	595	395	1.684
27	0.57813	1,192	1,025	870	602	405	1.730
28	0.56250	1,216	1,036	873	667	421	1.778
29	0.54688	1,341	1,057	912	695	472	1.829
30	0.53125	1,346	1,166	988	696	485	1.882
31	0.51563	1,388	1,170	1,007	744	544	1.939
32	0.50000	1,400	1,206	1,041	797	550	2.000
33	0.48438	1,417	1,243	1,150	817	564	2.065
34	0.46875	1,430	1,365	1,151	871	574	2.133
35	0.45313	1,560	1,390	1,209	875	595	2.207
36	0.43750	1,599	1,401	1,215	881	616	2.286
37	0.42188	1,612	1,520	1,308	925	650	2.370
38	0.40625	1,635	1,529	1,399	1,024	737	2.462

Table 6. Peak Flow and 1-, 3-, 7-, and 15-day Flow Volumes with plotting positions for Littlejohn Creek at Farmington, CA.

Farmington 1Day Annual Maximums, continued							
		Peak	1-day	3-day	7-day	15-day	
No.	Prob	Y-Axis	Y-Axis	Y-Axis	Y-Axis	Y-Axis	1/Prob
39	0.39063	1,823	1,530	1,437	1,126	811	2.560
40	0.37500	1,841	1,600	1,442	1,190	861	2.667
41	0.35938	1,865	1,621	1,446	1,216	891	2.783
42	0.34375	1,921	1,670	1,512	1,328	895	2.909
43	0.32813	2,027	1,762	1,633	1,347	1,013	3.048
44	0.31250	2,048	1,763	1,657	1,362	1,019	3.200
45	0.29688	2,102	1,780	1,673	1,376	1,049	3.368
46	0.28125	2,117	1,840	1,697	1,386	1,056	3.556
47	0.26563	2,128	1,850	1,699	1,498	1,069	3.765
48	0.25000	2,128	1,850	1,733	1,545	1,078	4.000
49	0.23438	2,132	1,853	1,788	1,579	1,089	4.267
50	0.21875	2,149	1,867	1,788	1,592	1,104	4.571
51	0.20313	2,163	1,868	1,793	1,607	1,122	4.923
52	0.18750	2,197	1,880	1,809	1,645	1,205	5.333
53	0.17188	2,216	1,901	1,830	1,661	1,220	5.818
54	0.15625	2,311	1,910	1,833	1,669	1,231	6.400
55	0.14063	2,312	1,993	1,833	1,700	1,232	7.111
56	0.12500	2,328	2,009	1,871	1,709	1,250	8.000
57	0.10938	2,359	2,010	1,897	1,737	1,324	9.143
58	0.09375	2,374	2,023	1,938	1,770	1,497	10.667
59	0.07813	2,383	2,050	1,981	1,776	1,549	12.800
60	0.06250	2,388	2,064	1,989	1,798	1,614	16.000
61	0.04688	2,821	2,452	2,011	1,826	1,677	21.333
62	0.03125	3,336	2,900	2,373	1,940	1,883	32.000
63	0.01563	3,958	3,440	2,723	2,225	1,959	64.000
Interpolated Values							
Event#	Prob	Peak	1-day	3-day	7-day	15-day	1/Prob
32	0.5	1,400	1,206	1,041	797	550	2
50-51	0.2	2170	1870	1796	1614	1138	5
57-58	0.1	2368	2018	1921	1756	1426	10
61-62	0.04	2615	2089	2002	1839	1736	25
62-63	0.02	3744	3486	2070	1900	1843	50
64	0.01	9900	8600	7400	5400	3800	100
65	0.005	12900	12000	10000	7400	4400	200
66	0.002	16600	15200	12000	8600	5200	500
Values in yellow are from Tansform Curve & Table							

Table 6 (continued). Peak Flow and 1-, 3-, 7-, and 15-day flow volumes with plotting positions for Littlejohn Creek at Farmington, CA.

Regulated Peak Flow values and associated volumes:
Littlejohn Creek at Farmington

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration)			
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)
0.5	1,400	1,206	1,041	797	550
0.2	2,170	1,870	1,796	1,614	1,138
0.1	2,368	2,018	1,921	1,756	1,426
0.04	2,615	2,089	2,002	1,839	1,736
0.02	3,744	3,486	2,070	1,900	1,843
0.01	9,900	8,600	7,400	5,400	3,800
0.005	12,900	12,000	10,000	7,400	4,400
0.002	16,600	15,200	12,000	8,600	5,200

1) Revised to reflect graphical fit of observed data from Oct1949 to Dec2011 for the 0.5 to the 0.02 AEP. The 0.01 to 0.002 AEP events are from the revised flow transform and regulated flow-freq curve. The volumes were computed from the regulated peak to volume transforms in the Ford report.

Table 7. Regulated Peak Flows and Associated Volumes for Littlejohn Creek at Farmington.

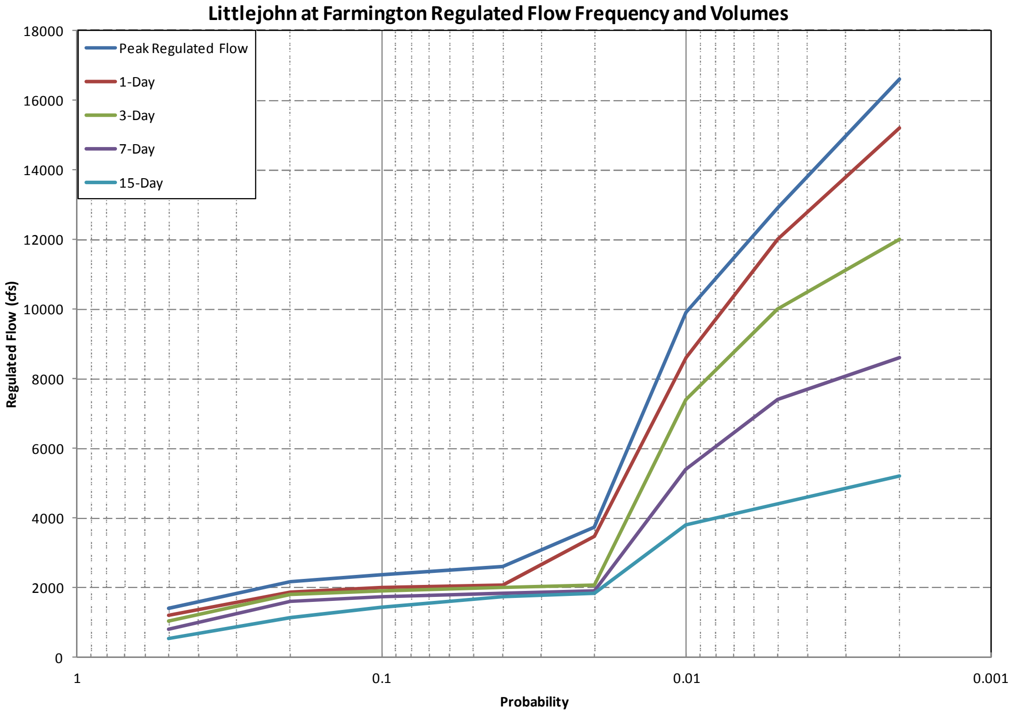


Figure 10. Regulated Peak Flow and Associated Volumes at Littlejohn Creek at Farmington.

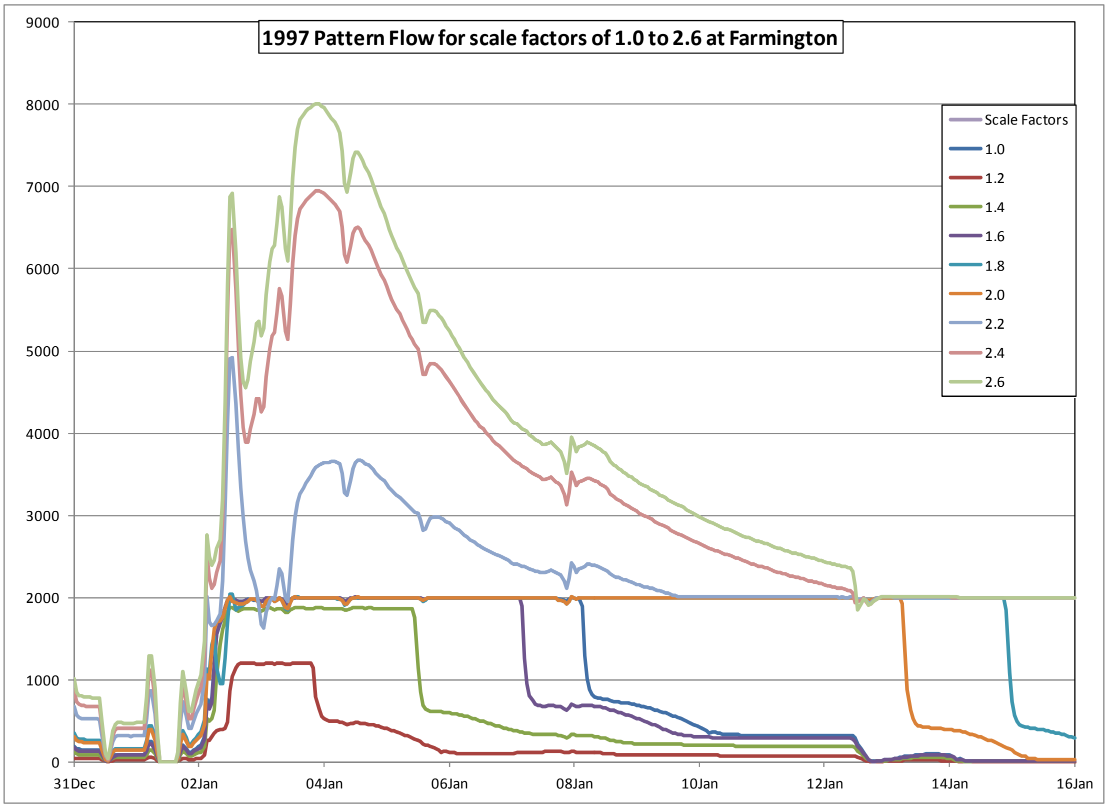


Figure 11. 1997 Pattern Flows for scale factors from 1.0 to 2.6 at Farmington.

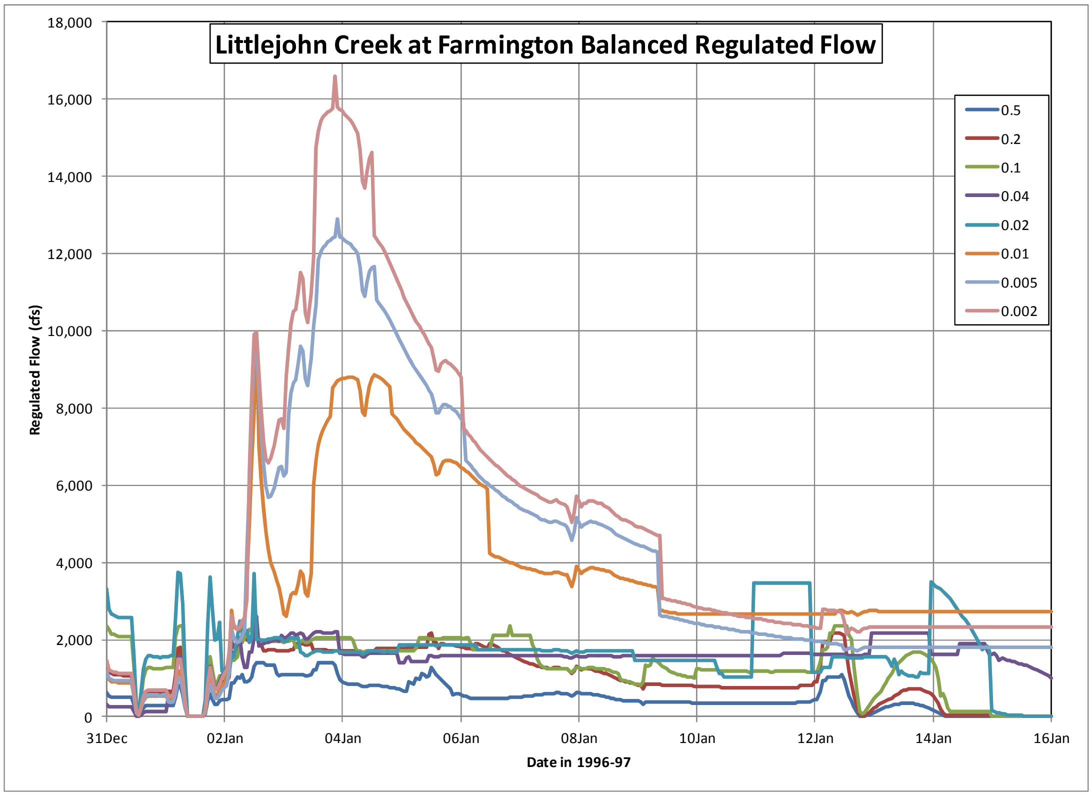
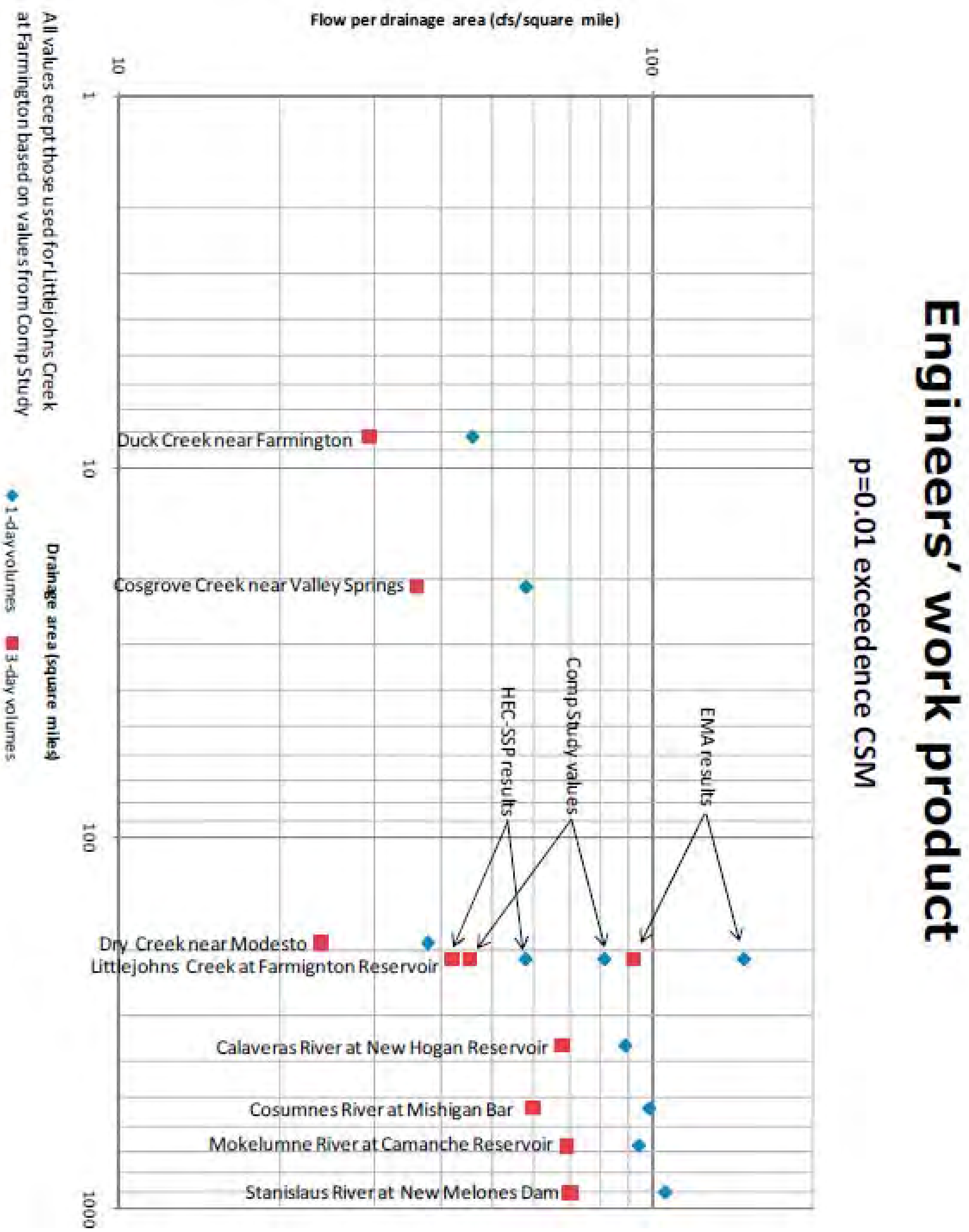


Figure 12. Littlejohn Creek at Farmington Regulated Flow Hydrographs, 31Dec96 to 16Jan97.



Engineers' work product

p=0.01 exceedence CSM

All values ecept those used for Littlejohns Creek at Farmington based on values from Comp Study

Figure 13: 0.01 ACE CSM Plot for Unregulated Frequency Curves
Note: Values shown are for original statistics prior to adjustments

ACE	Peak	1-day	3-day	7-day	15-day	30-day
50	10082	5625	3772	2372	1574	1090
20	22801	12962	8594	5395	3553	2393
10	32641	18584	12337	7801	5116	3402
4	45622	25878	17283	11068	7230	4747
2	55262	31192	20957	13566	8838	5761
1	64645	36272	24533	16059	10438	6763
0.5	73706	41087	27986	18527	12017	7745
0.2	85113	47014	32330	21723	14056	9005

Figure 14: Unregulated Frequency Curve Quantiles for Littlejohn Creek at Farmington Dam.

ACE	Peak	1-day	3-day	7-day	15-day	30-=day
50.00	10,082	5,625	3,772	2,372	1,574	1,090
20.00	22,801	12,962	8,594	5,395	3,553	2,393
10.00	32,641	18,584	12,337	7,801	5,116	3,402
4.00	45,622	25,878	17,283	11,068	7,230	4,747
2.00	55,262	31,192	20,957	13,566	8,838	5,761
1.00	64,645	36,272	24,533	16,059	10,438	6,763
0.50	73,706	41,087	27,986	18,527	12,017	7,745
0.20	85,113	47,014	32,330	21,723	14,056	9,005

Figure 15: Unregulated Frequency Curve Quantiles for Littlejohn Creek at Farmington, CA

Appendix 2- Attachment 1

Lower San Joaquin Feasibility Study Littlejohn Creek frequency analysis and hydrographs

Lower San Joaquin River feasibility study: Littlejohn Creek frequency analysis and hydrographs

June 23, 2011

US Army Corps of Engineers Sacramento District
W91238-09-D-0004, TO 0004

David Ford Consulting Engineers, Inc.
2015 J Street, Suite 200
Sacramento, CA 95811
Ph. 916.447.8779
Fx. 916.447.8780

Engineer's certification

I, Michael Konieczki, hereby certify on 6/23/2011 that I am a professional engineer licensed in the state of California and that the accompanying report was prepared by me or under my supervision.

MICHAEL V. KONIECZKI
No. 74357
Exp. 6/30/2013
CIVIL

Contents

Tables

Figures

Executive summary

Situation

In the lower San Joaquin River feasibility study (LSJR FS) the Sacramento District of the US Army Corps of Engineers (Corps) and the San Joaquin Area Flood Control Agency (SJAFCA) are studying alternative flood risk reduction measures that will provide protection against a flood with a probability of exceedence in any given year equal 0.005 (i.e., a "200-year flood").

The LSJR FS includes hydrologic analyses of the study region. This same region is also being studied in conjunction with a separate project to map the floodplains adjacent to the federal-state levee system in the Central Valley. Because the products of the various hydrologic analyses being conducted in the lower San Joaquin River basin will be used for several purposes by multiple agencies and stakeholders, the firms and agencies involved are using consistent analytical procedures and methods where possible. These procedures are specified in the *Sacramento and San Joaquin river basins: Procedures for hydrologic analysis* (hereinafter, *Procedures document*) and the *Central Valley hydrology study (CVHS): Technical procedures document* (hereinafter, *Technical procedures document*). Attachment 1 provides a table that explains how the procedures detailed in the present document align with the procedural steps detailed in the *Procedures document* and the *Technical procedures document*.

In this report we detail our hydrologic analyses at 2 sites on Littlejohn Creek: (1) Farmington Reservoir, and (2) Farmington Reservoir's operation point at Farmington, CA. These sites are shown in Figure 1.

Tasks

Our tasks were to: (1) develop a regulated flow-frequency curve and associated volumes at each location, and (2) derive an "expected" outflow hydrograph at Farmington Reservoir.

Actions

To complete the tasks above, we:

- Developed unregulated volume-frequency curves at Farmington Reservoir and Farmington, CA, following the procedures in *Guidelines for determining flood flow frequency, Bulletin 17B* (IACWD 1982) and EM 1110-2-1415 (USACE 1993) and using a regional skew provided by the Corps.
- Simulated reservoir releases and routed historical and scaled floods, including local flows, on Littlejohn Creek using an HEC-ResSim model provided by the Corps.
- Fitted, at each location, flow transforms to the event maxima dataset identified from the unregulated flow and simulated release time series.
- Developed, at each location, a regulated flow-frequency curve and associated volumes by applying the flow transforms.
- Developed "expected" outflow hydrographs for Farmington Reservoir for 8 flood frequencies: p=0.5, p=0.2, p=0.10, p=0.05, p=0.02, p=0.01,

p=0.005 and p=0.002. (Here the term expected hydrograph refers to a Farmington Reservoir outflow hydrograph with a peak flow that matches the regulated flow-frequency curve and with associated volumes matching those from the family of characteristic curves corresponding to the given regulated peak flow.)

Results

The results of our analysis include:

- Unregulated volume-frequency curves for Farmington Reservoir (as shown in Figure 2).
- Unregulated volume-frequency curves for Littlejohn Creek at Farmington, CA (as shown in Figure 3).
- Unregulated-regulated flow transform for Farmington Reservoir (as shown in Figure 4).
- Regulated flow-frequency curve and associated volumes for Farmington Reservoir (as shown in Table 1 and in Table 2).
- Unregulated-regulated flow transform for Littlejohn Creek at Farmington, CA (as shown in Figure 5).
- Regulated flow-frequency curve and associated volumes for Littlejohn Creek at Farmington, CA (as shown in Table 3 and in Table 4).
- Expected hydrograph properties for Farmington Reservoir. (Note: these are the same values shown in Table 1).

In addition, these intermediate values and information are included with the original report on DVD:

- HEC-DSS time series of the floods-of-records.
- HEC-DSS time series of the scaled historical floods.
- HEC-DSS time series of developed local flows below Farmington Reservoir (detailed in Attachment 2).
- The tabulated event maxima datasets for the 2 analysis sites.
- Simulated reservoir releases and routed flows from the HEC-ResSim reservoir simulation model.
- Tabulated unregulated-regulated flow transforms for the 2 analysis sites.
- Tabulated families of regulated characteristic curves for the 2 analysis sites.

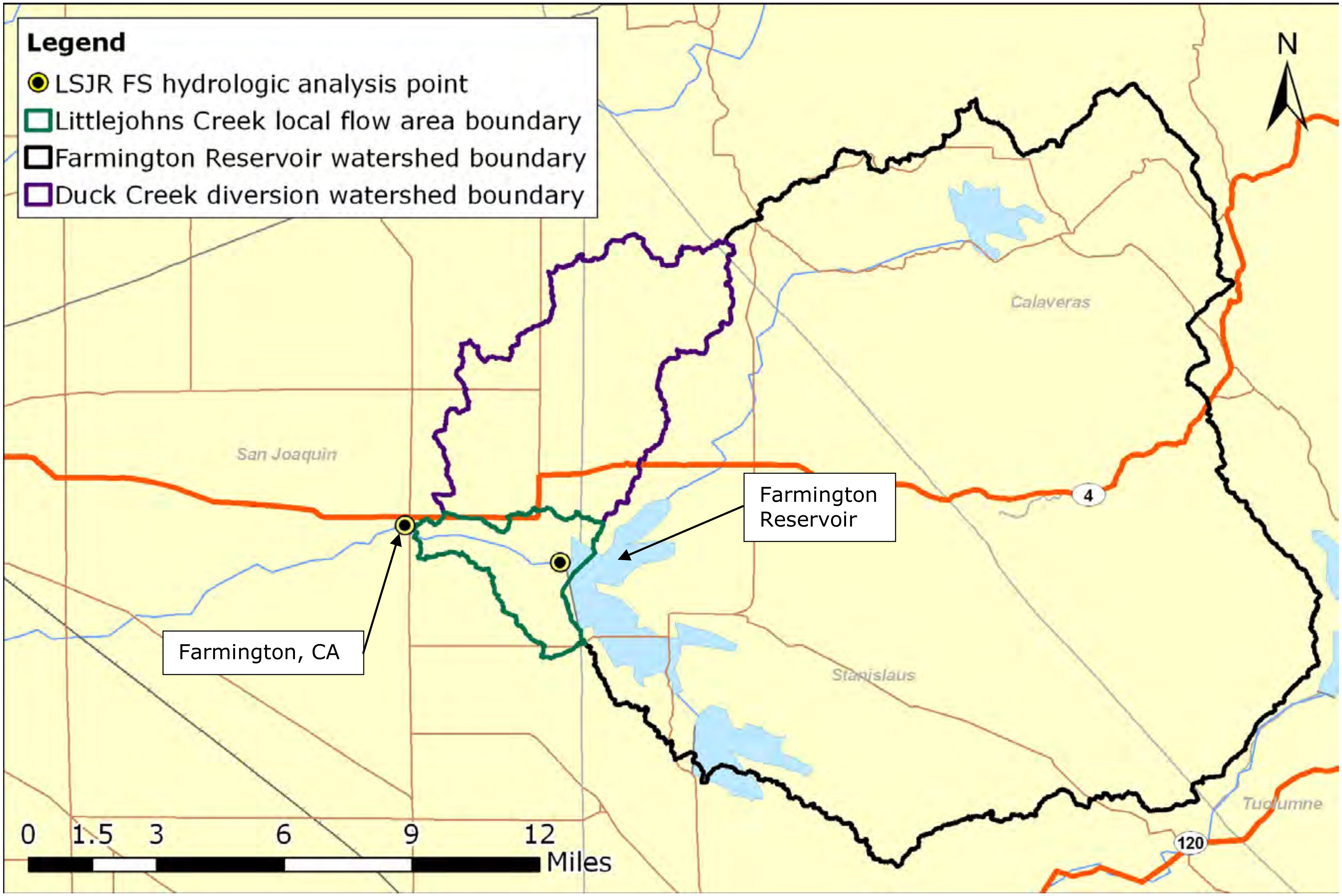
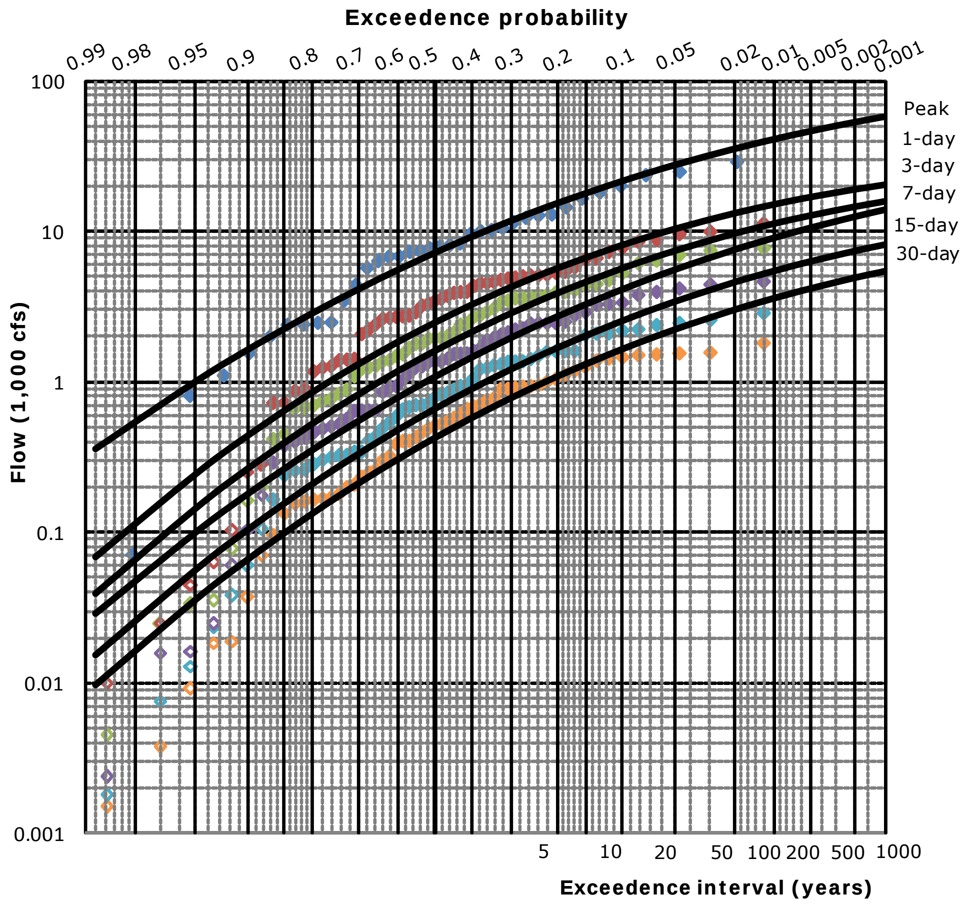


Figure 1. Littlejohn Creek study area



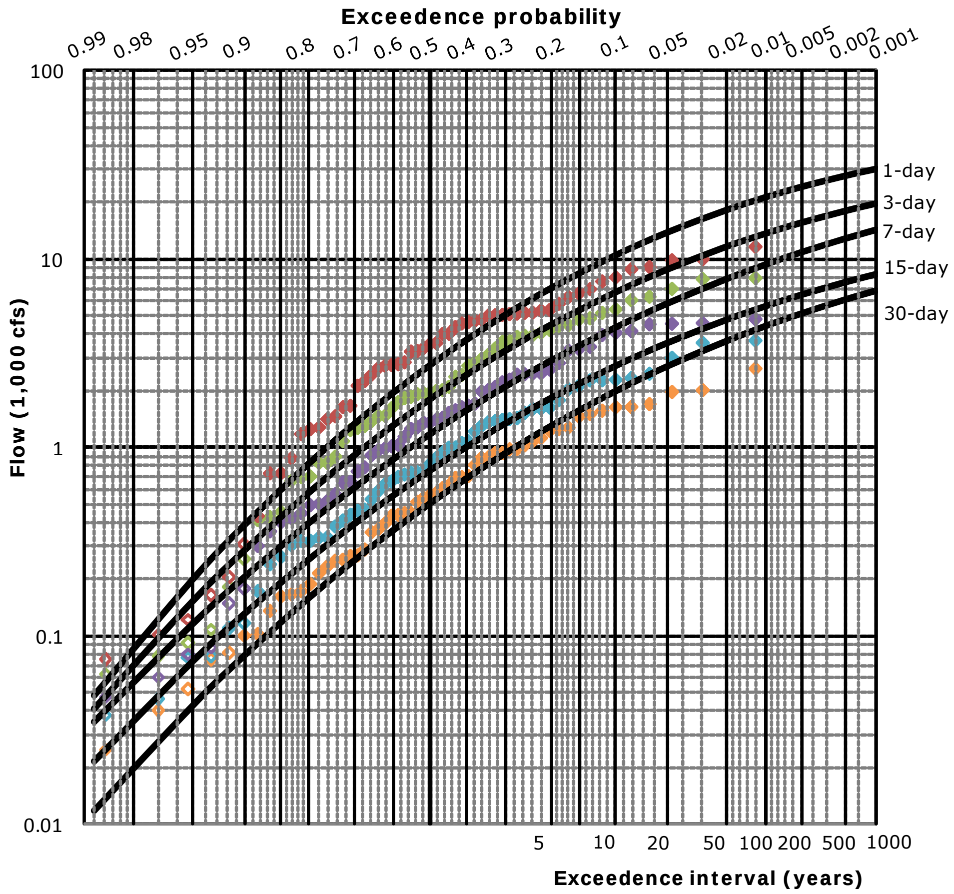
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.811	0.445	-0.692
1-day	3.321	0.507	-0.858
3-day	3.135	0.531	-0.812
7-day	2.970	0.538	-0.675
15-day	2.754	0.553	-0.733
30-day	2.561	0.556	-0.721

Notes:

- Median plotting positions.
- Drainage area: 212 sq. miles.
- Period of systematic record: 1951-2008.
  (Peak flow data intermittent 1952-2004).
- Record lengths
  Peak flows: 53 years.
  Volumes: 58 years.
- Regional skew values developed by USGS.
- Low outliers for volumes: 8 smallest events.
- Hollow points are censored events.

Figure 2. Unregulated frequency curves: Farmington Reservoir



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.356	0.573	-0.849
3-day	3.186	0.545	-0.786
7-day	3.011	0.525	-0.670
15-day	2.815	0.523	-0.722
30-day	2.639	0.556	-0.695

Notes:

- Median plotting positions.
- Drainage area: 219 sq. miles.
- Period of systematic record: 1951-2008.
- Record length: 58 years.
- Regional skew values developed by USGS.
- Low outliers for 1-, 3, 7, and 15-day volumes: 7 smallest events.
- Low outliers for 30-day volumes: 6 smallest events.
- Hollow points are censored events.

Figure 3. Unregulated frequency curves: Littlejohn Creek at Farmington, CA

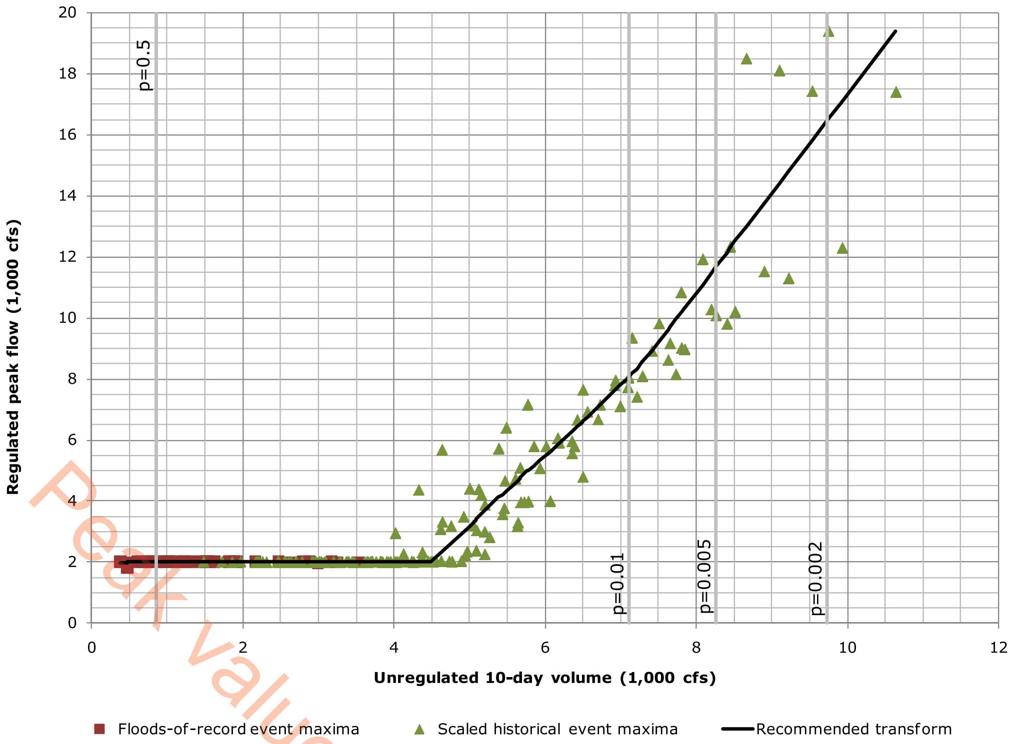


Figure 4. Unregulated-regulated flow transform: Farmington Reservoir

Table 1. Regulated peak flow-frequency quantiles: Farmington Reservoir

Annual exceedence probability (1)	**1/annual exceedence probability (2)**	**Regulated peak flow (cfs) (3)**
0.500	2	2,000
0.200	5	2,000
0.100	10	2,000
0.050	20	2,000
0.020	50	5,360
0.010	100	8,077
0.005	200	11,671
0.002	500	16,444

Table 2. Regulated peak flow values and associated volumes: Farmington Reservoir

Annual exceedence probability of regulated peak flow (1)	**Regulated peak flow (cfs) (2)**	**Associated volumes[1] (as average flow for given duration)**				
		1-day (cfs) (3)	**3-day (cfs) (4)**	**7-day (cfs) (5)**	**15-day (cfs) (6)**	**30-day (cfs) (7)**
0.500	2,000	2,000	1,994	1,987	1,910	1,491
0.200	2,000	2,000	1,994	1,987	1,910	1,491
0.100	2,000	2,000	1,994	1,987	1,910	1,491
0.050	2,000	2,000	1,994	1,987	1,910	1,491
0.020	5,360	5,213	4,601	3,469	2,776	2,458
0.010	8,077	7,833	6,783	4,996	3,614	3,052
0.005	11,671	11,307	9,746	7,397	4,662	3,536
0.002	16,444	15,928	13,704	10,695	5,043	3,563

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at Farmington Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

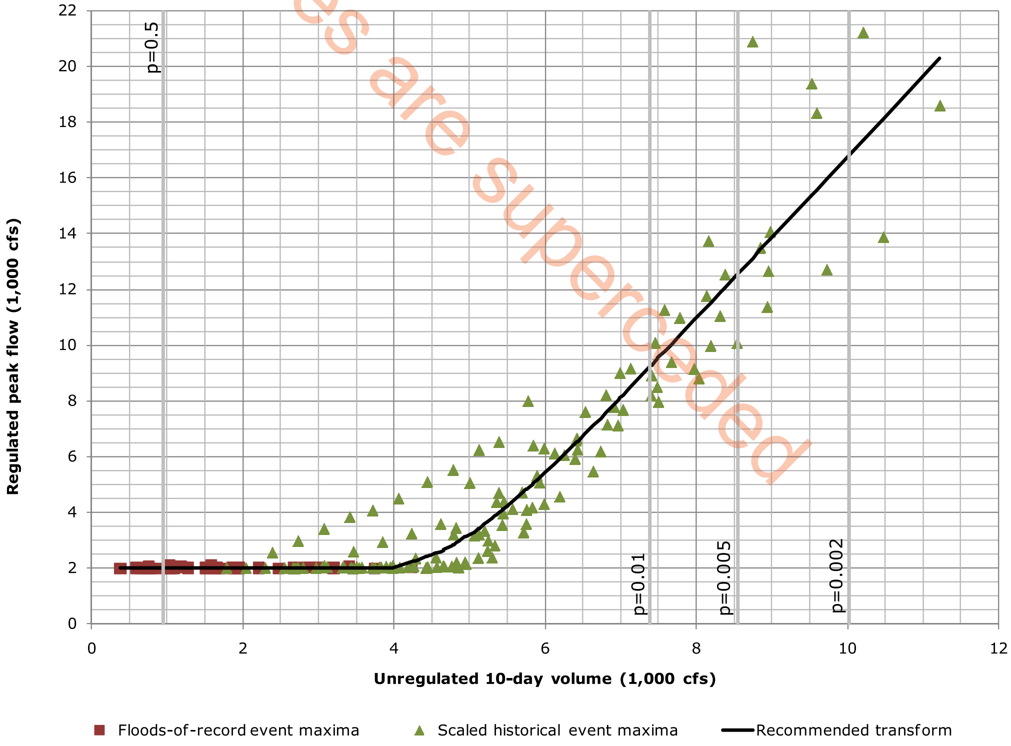


Figure 5. Unregulated-regulated flow transform: Littlejohn Creek at Farmington, CA

Table 3. Regulated peak flow-frequency quantiles: Littlejohn Creek at Farmington, CA

Annual exceedence probability (1)	1/annual exceedence probability (2)	Regulated peak flow (cfs) (3)
0.500	2	2,000
0.200	5	2,000
0.100	10	2,000
0.050	20	2,633
0.020	50	5,964
0.010	100	9,231
0.005	200	12,548
0.002	500	16,839

Table 4. Regulated peak flow values and associated volumes: Littlejohn Creek at Farmington, CA

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration)				
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)	30-day (cfs) (7)
0.500	2,000	2,000	1,967	1,960	1,296	827
0.200	2,000	2,000	1,967	1,960	1,296	827
0.100	2,000	2,000	1,967	1,960	1,296	827
0.050	2,633	2,073	2,073	2,073	2,016	1,869
0.020	5,964	5,622	4,978	3,742	2,923	2,616
0.010	9,231	8,741	7,430	5,576	3,943	3,211
0.005	12,548	11,773	9,833	7,268	4,649	3,613
0.002	16,839	15,385	12,070	8,790	5,291	3,781

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at Farmington Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Watershed description

The watershed that is the subject of this report—Littlejohn Creek basin—is part of the lower San Joaquin River basin. It is located in Calaveras, San Joaquin, and Stanislaus counties. Located on Littlejohn Creek approximately 20 miles upstream of Stockton, CA, is Farmington Reservoir, a "dry dam" whose primary purpose is flood control.

The principal feature of the watershed, shown in Figure 6, is Farmington Reservoir, which drains approximately 212 mi^2. The watershed above the reservoir is wing-shaped and extends 20 miles upstream into the foothills of the western Sierra Nevada. Elevations range from approximately 2,600 ft to approximately 115 ft at the dam.

In addition to runoff from the foothills, Farmington Reservoir receives flows from a diversion on the Stanislaus River at Goodwin Dam, the Stockton East Tunnel, and the Farmington-Stockton East Canal. These flows occur primarily during the summer months and not during the flood season, typically defined as October 1 to May 1 of each water year.

Downstream of Farmington Dam, approximately 3.5 miles, is the Duck Creek Diversion, which diverts flow into Littlejohn Creek from Duck Creek above the town of Farmington. The watershed above the diversion structure on Duck Creek is approximately 28 mi^2. The channel capacity of Duck Creek below the diversion structure is 700 cfs, and the diversion structure itself has a peak capacity of 500 cfs. In addition, the confluence of Littlejohn Creek and Rock Creek is approximately 2 miles downstream of Farmington Dam.

From the town of Farmington, Littlejohn Creek continues west, splitting into the North Fork Littlejohn Creek and South Fork Littlejohn Creek. Flow finally joins French Camp Slough before continuing on to the San Joaquin River. The confluence of Littlejohn Creek and French Camp Slough is located approximately 25 miles downstream of Farmington Dam.

Farmington Reservoir operates to maintain peak flows below the downstream channel capacity of 2,000 cfs near the town of Farmington, including anticipated coincident flows from the Duck Creek Diversion (USACE 2004).

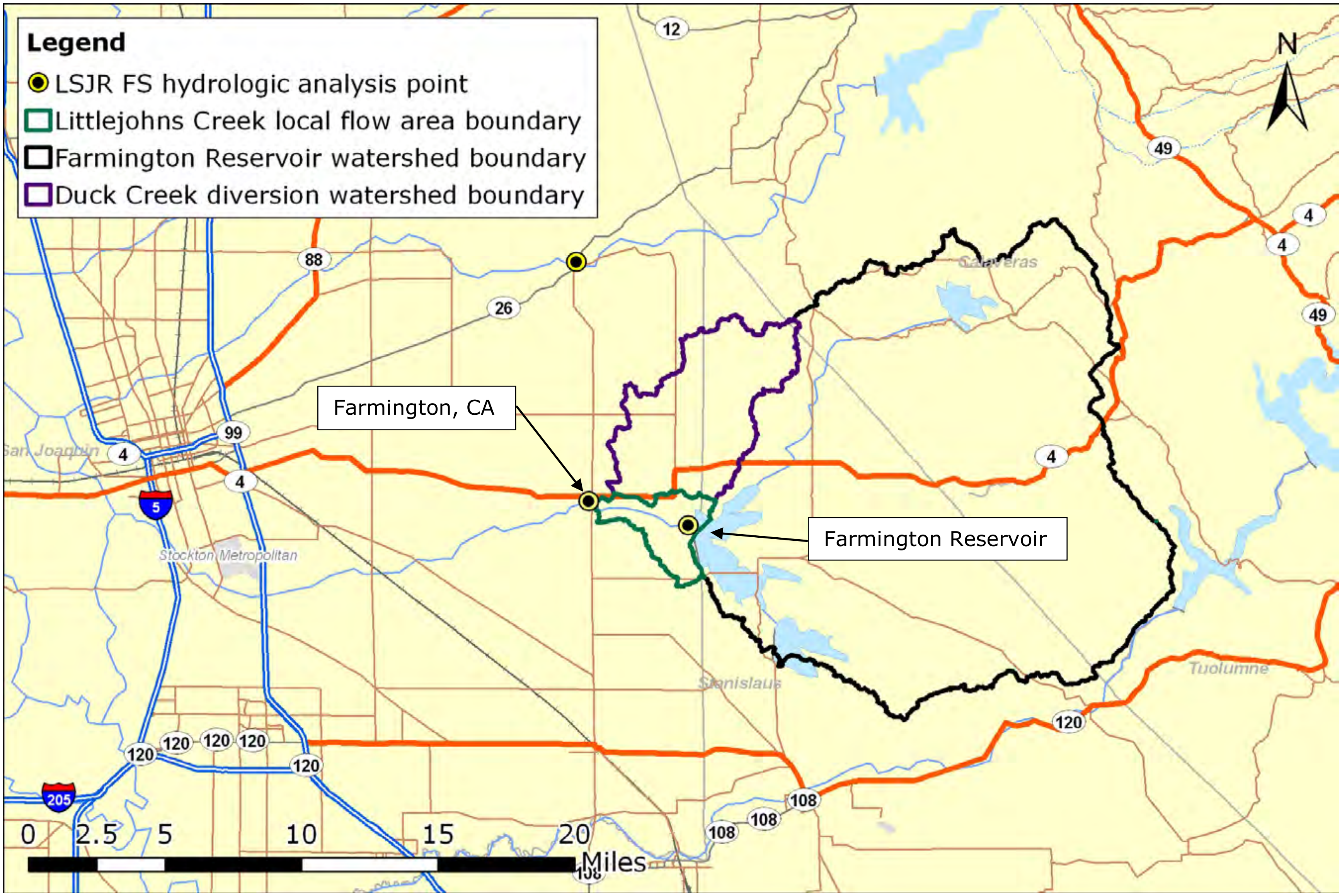


Figure 6. Lower San Joaquin River feasibility study area: Littlejohn Creek

Analysis procedure

Overview of CVHS procedure

The primary tasks for the CVHS are described in the *Procedures document*. More detail for these tasks is provided in the *Technical procedures document*. As a review of those tasks and to provide context for the procedures used in this analysis, here we summarize the procedure steps and categorize them into 2 groups. They are:

- Group 1. Unregulated frequency analysis at selected points. This comprises *Procedures document* Task 1, Task 2 (reservoir simulation models), Task 3, and Task 4. (References throughout this report to numbered tasks use numbers from the *Procedures document*.)
- Group 2. Assement of the effects of the regulation (flood control) system to convert the unregulated frequency curves to regulated flow-frequency curves at the same selected points. This comprises *Procedures document* Task 2 (channel routing models), Task 5, Task 6, and Task 7.

Group 1 focuses on completing a frequency analysis to characterize the annual exceedence probability of a given flow (unregulated). Thus, all statements of probability originate here.

Group 2 reflects the impact of regulation in the system. This second group accounts for various historical storm distributions and reservoir operations, with an emphasis on large events.

Application to the lower San Joaquin River feasibility study

In Figure 7, we illustrate the general work flow of the analysis procedure as applied to the LSJR FS. In this document we note before each analysis step the corresponding CVHS procedures task applicable, if any.

For unregulated frequency analysis for the 2 sites on Littlejohn Creek, Farmington Reservoir and Farmington, CA, we:

- (Task 1) Obtained reservoir inflow and streamgage data for use in developing the unregulated flow time series from the Corps.
- (Task 2) Obtained accepted reservoir simulation and channel routing models from the Corps.
- (Task 3) Developed unregulated flow time series at each location corresponding to a period-of-record of floods. This step includes the development of local flows for the ungaged area between New Hogan Dam and Farmington, CA.
- (Task 4) Computed and adopted unregulated 1-, 3-, 7-, 15-, and 30-day volume-frequency curves at each location. Note: we developed peak unregulated flow-frequency curves for Farmington Reservoir for completeness; they are not required for this analysis.

For regulated system analysis for the 2 sites on Littlejohn Creek we:

- (Task 5) Developed regulated flow time series at each location by simulating and routing reservoir releases. Here, historical and scaled historical events were used in development of the time series.

- (Task 6) Fitted flow transforms. First, the unregulated and corresponding regulated event maxima datasets were identified (these are data points to which the transforms were fitted). Then, the critical duration of each analysis location was determined using these series. The flow transforms were then developed by fitting curves to the event maxima datasets. Note here, the term flow transforms refers to: (1) the unregulated-regulated flow transform, and (2) the family of regulated characteristic curves.
- (Task 6.4) Applied flow transforms to develop a regulated peak flow-frequency curve and associate volumes for the 1-, 3-, 7-, 15-, and 30-day durations at each location.

For development of the expected hydrograph properties for Farmington Reservoir outflows we identified the peak regulated flows and associated regulated volume-duration characteristics for 8 exceedence probabilities: p=0.5, p=0.2, p=0.1, p=0.05, p=0.02, p=0.01, p=0.005, and p=0.002.

Attachment 1 provides a table explaining how the procedures detailed here align with the procedural steps detailed in the *Procedures document* and the *Technical procedures document*.

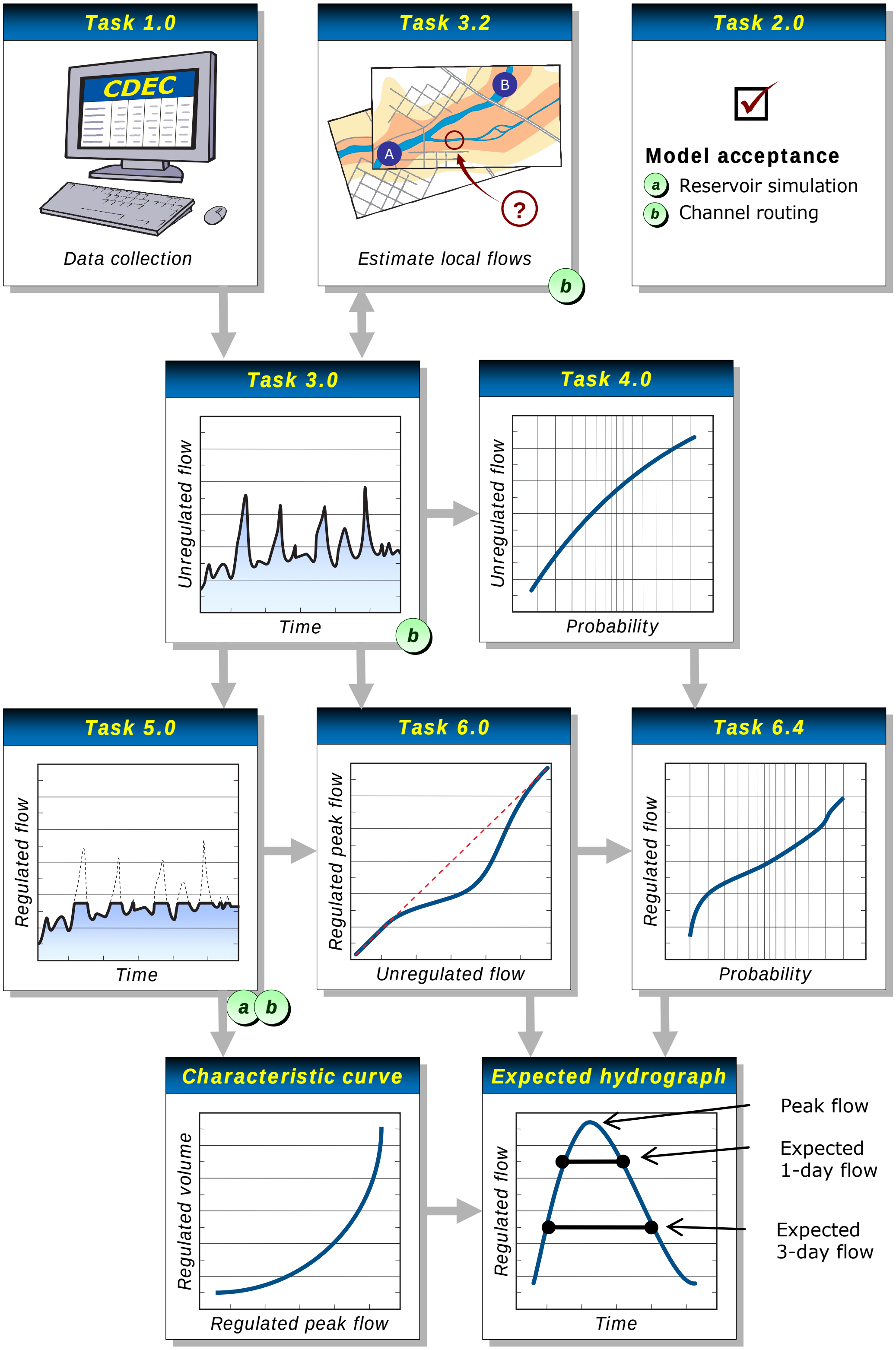


Figure 7. LSJR analysis procedure workflow

Unregulated flow time series development

We constructed unregulated flow time series at each analysis location in the study area and fitted unregulated volume-frequency curves to these series using procedures that are consistent with Corps guidance.

The locations most upstream at which we developed unregulated flow time series were the project reservoirs. Thus, for unregulated conditions, the reservoir inflows were needed.

For development of the unregulated flow time series downstream of the reservoir, a routing model was required to simulate the translation, attenuation, and combination of the unregulated flow hydrographs through the system. These flow hydrographs included the upstream boundary conditions (derived reservoir inflows) and intermediate area boundary conditions (estimated local flows). The routing yielded unregulated flow time series that served as the basis of: (1) the unregulated frequency analysis and (2) the unregulated-regulated flow transform.

For this analysis, we developed an unregulated flow time series for the 2 analysis locations on Littlejohn Creek by:

- (Task 1) Obtaining daily unregulated reservoir inflow time series developed by the Corps.
- (Task 3.2) Developing local flow time series for the area between Farmington Reservoir and the reservoir's control point at Farmington, CA (shown in Figure 8).
- (Task 3.3) Completing the unregulated flow time series at each analysis point.

Obtain daily reservoir inflow

We obtained the daily unregulated reservoir inflows from the Corps. The Corps developed the daily unregulated reservoir inflow time series for Farmington Reservoir using the continuity equation, in which, for a given time step, the average inflow equals the outflow plus the change in reservoir storage. For the calculation of these inflows, the source of the observed reservoir outflows and observed changes in storage was the Corps's database. By convention in the Central Valley, these calculations were completed on a 1-day time step, thus midnight to midnight values were used. This is consistent with the work completed for the *Sacramento and San Joaquin river basins comprehensive study* (Comp Study) completed in 2002 (USACE 2002).

Estimate local flow

For Littlejohn Creek, local flows needed to be estimated for the area between Farmington Reservoir and Farmington, CA, shown in Figure 8. The estimation approaches we used were:

- Option 1. Direct calculation of local flow using known releases from Farmington Reservoir, known diversions from Duck Creek, and the observed flows at Farmington, CA, routing hourly flows as necessary. In the case of missing streamgage data, local flows values were interpolated as needed.

- Option 2. Estimation of local flows as:

$$Q_{Local} = 0.04\left(Q_{FRM}\right) \tag{1}$$

where Q_{Local} is the local flow estimate for a given time, and Q_{FRM} is the unregulated inflow to Farmington Reservoir. The Corps estimates local flows for the purpose of real-time reservoir operations using this option (John High, personal communication, 11/9/2009) and this is the option used to estimate local flows in the Comp Study (USACE 2002).

In Table 5 we summarize the selected approaches for local flow estimation on Littlejohn Creek by water year. This flow represents the total local flow contribution at Farmington, CA. We detail the development of the local flow time series on Littlejohn Creek in Attachment 2.

Table 5. Selected local flow estimation approaches for the area on Littlejohn Creek between Farmington Reservoir and Farmington, CA

Time period (water year) (1)	Time step (2)	Selected approach[1] (3)
1951-1968	Daily	Option 1: directly calculate local flow.
1969-1970	Daily	Option 2: 0.04 times reservoir inflow.
1971-1972	Daily	Option 1: directly calculate local flow.
1973	Daily	Option 2: 0.04 times reservoir inflow.
1974-1996	Daily	Option 1: directly calculate local flow.
1996-2008	Hourly	Option 1: directly calculate local flow.

1. The approach listed is the predominant method for estimating local flows over the time period given. See Attachment 2 for further detail.

Complete unregulated flow time series

For the unregulated frequency analysis, we used the daily unregulated reservoir inflow time series provided by the Corps directly as the unregulated time series corresponding to Farmington Reservoir. For the reservoir's operation point on Littlejohn Creek at Farmington, CA, we combined the daily unregulated inflow time series with the estimated local flows by adding the 2 time series together. We did not route the unregulated reservoir inflows because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the reservoir and the operation point is approximately 2 hours, which is less than the 1-day time step of the inflows. In addition, there is little attenuation of flood peaks in this reach because of its length and channel geometry. We confirmed this by comparing observed releases from Farmington Reservoir, observed diversions from Duck Creek, and observed flows on Littlejohn Creek at Farmington, CA. The unregulated flow time series at Farmington, CA, does not include diversions from Duck Creek.

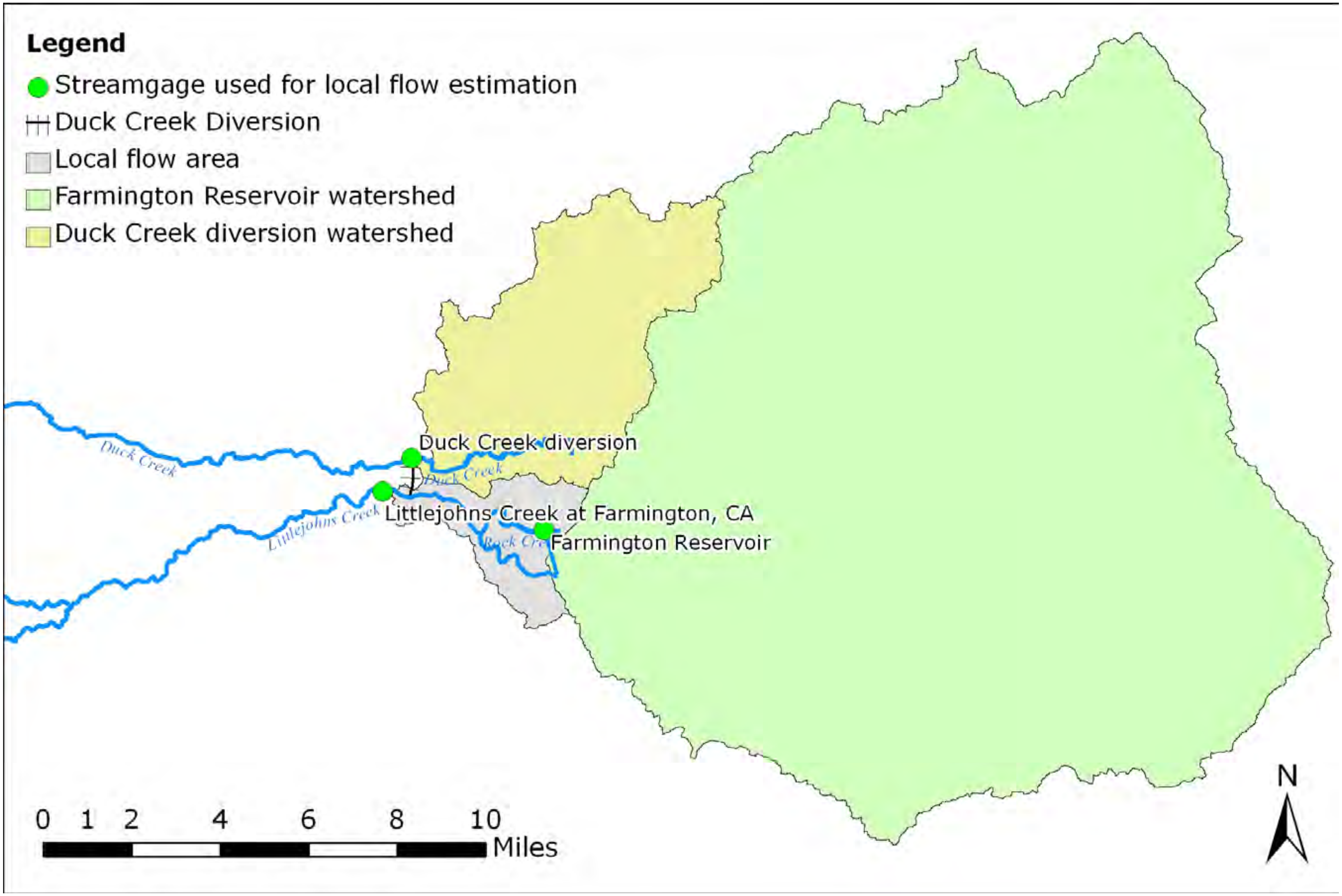


Figure 8. Littlejohn Creek local flow area between Farmington Reservoir and Farmington, CA, and study streamgages

Unregulated frequency analysis

Commonly accepted procedures to develop unregulated flow-frequency curves are specified in *Bulletin 17B* (IACWD 1982). The current standard-of-practice is to fit a Pearson III (LPIII) distribution to the logarithmic transforms of annual maximum series identified from streamgage data. Additional guidance for fitting frequency curves to volumes for a given duration is provided by EM 1110-2-1415 (USACE 1993).

For this analysis, the unregulated inflows to Farmington Reservoir can be used to develop such an annual maximum series. However, because we only had records of regulated flows on Littlejohn Creek at Farmington, CA, we could not fit a frequency curve directly using this method. Thus, we used the synthesized unregulated flow time series at this location and fitted a volume-frequency curve to that series using procedures that are consistent with Corps guidance.

For this analysis we developed unregulated frequency curves following the procedures specified in *Bulletin 17B* (IACWD 1982), EM 1110-2-1415 (USACE 1993), and the current standards of practice. For each analysis location, we:

- Identified the annual maximum series.
- (Task 4.1) Calculated regional skew values for each duration of interest using relationships developed by the USGS.
- (Task 4.2) Fitted LPIII distributions to the annual maximum series following *Bulletin 17B* procedures and Corps guidance using the expected moment algorithm (EMA) enabled flow-frequency software PeakfqSA, version 0.937. This was developed by Tim Cohn of the USGS and is based on the USGS's flow-frequency software PeakFQ (Cohn 2007).
- Reviewed and adopted the curves, checking them for consistency and comparing them to previously accepted values.

Identify annual maximum series

We identified the annual maximum series by extracting, from the unregulated flow time series, the volumes associated with the 1-, 3-, 7-, 15-, and 30-day durations. This information is detailed in Attachment 3.

We developed a peak unregulated flow-frequency curve for Farmington Reservoir for completeness; however this is not required for this analysis. The peak annual maximum series was provided by the Corps and is included in Attachment 3. In addition, we did not develop a peak flow-frequency curve for Littlejohn Creek at Farmington, CA, because the temporal resolution of the unregulated flow time series, 1 hour to as long as 1 day, is not an appropriate representation of instantaneous unregulated peak flow values.

Calculate regional skew values

For this analysis, we calculated regional skew values for the peak flows and 1-, 3-, 7-, 15-, and 30-day volumes using the relationships developed by the USGS (USGS 2010). In these relationships, the regional skew value is a function of the average basin elevation. The values calculated for each analysis location and duration of interest are shown in Attachment 4.

Fit frequency curves

To fit frequency curves to the annual maximum series we used: (1) the statistics of the logarithmic transforms of unregulated flow time series (mean, standard deviation, and skew), and (2) the regional skew values for the peak flow, and 1-, 3-, 7-, 15-, and 30-day calculated using relationships developed by the USGS (2010). The "at station" statistics were calculated using the EMA option in PeakfqSA.

As a first step, the curves were fitted using a straightforward *Bulletin 17B* procedure in which all data points were included in the analysis and low outliers were identified by the *Bulletin 17B* outlier test (implemented automatically by the program). The station statistics were then appropriately adjusted. This includes weighting the station skew and regional skew values by the inverse of their associated errors. This weighting procedure is included in *Bulletin 17B*, and the weighted skew is automatically calculated by PeakfqSA.

We found that this initial fitting of the frequency curves: (1) was sensitive to low flow values, and (2) the 1-day and 3-day flow quantiles for p= 0.01 and p=0.005 annual exceedence probabilities were uncharacteristically large on a flow-per-square mile basis.

We then refitted the frequency curves setting the low outlier thresholds for each duration. Specifically, we set these thresholds consistent with those used in the Comp Study. In addition, we adjusted the standard deviations, following guidance in EM 1110-2-1415 (USACE 1993), for consistency. This fitting is detailed Attachment 4.

Review and adopt curves

After fitting, we reviewed the frequency curves for consistency and appropriateness. Specifically, we:

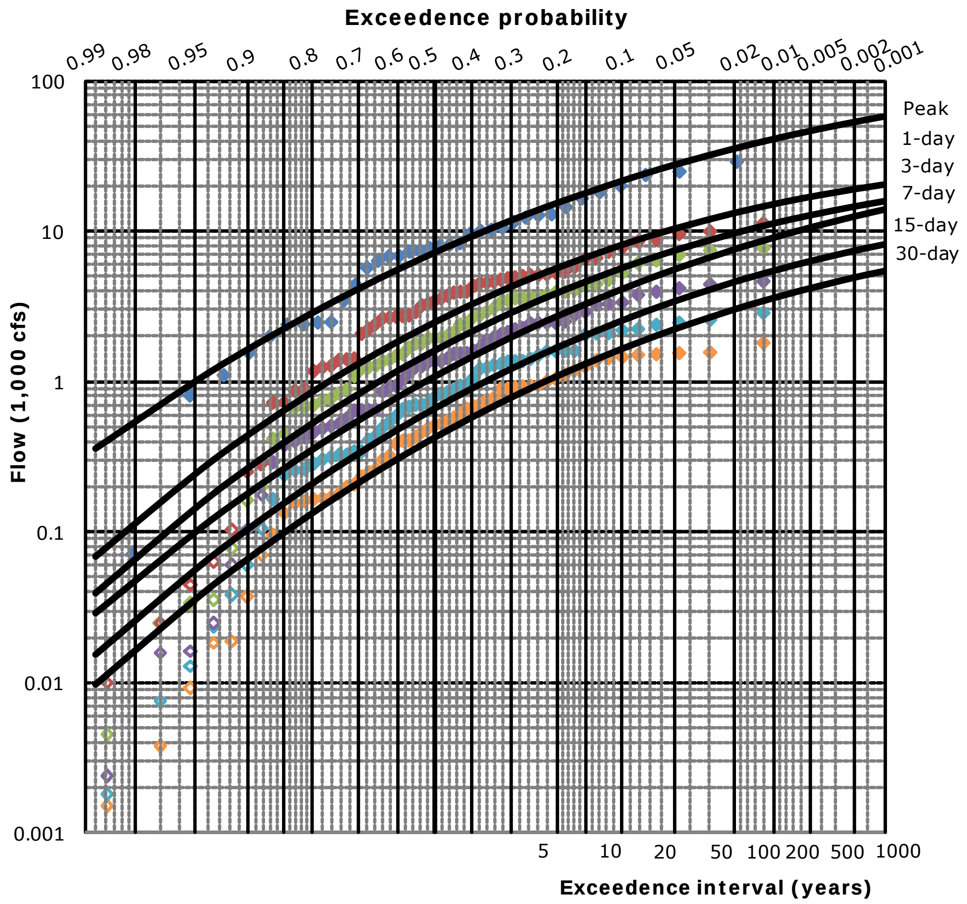
- Compared the curve of a given duration to the curves associated with the other durations at the same analysis location.
- Compared the curves at a given location to the curves at the other analysis location to check for consistency.
- Compared the curves to those published in the Comp Study.

We found the frequency curves on Littlejohn Creek were consistent between durations at each location for the frequencies of interest. The curves do not "cross," and flow quantiles for a given duration at the downstream location are greater than those of the upstream location, as would be expected.

As a comparison, we considered the volume-frequency curves developed for Farmington Reservoir in the Comp Study (USACE 2002). The annual maximum series in the Comp Study ended in 1998.

We also found that compared to the flow quantiles in the Comp Study the quantiles of the curves fitted here are: (1) smaller for the 1 day duration, and (2) larger for durations equal 3-days or greater. (Here the only exception is the 3-day p=0.5 quantile which we found to be approximately 9% less than that of the Comp Study.) However, we found that the 1-day and 3-day flow quantiles for p=0.01 and p=0.005 annual exceedence probabilities were consistent with those from nearby watersheds on a flow-per-square mile basis. In this analysis, the peak flow-frequency quantiles varied by as much as 9%, as compared to those in the Comp Study, because of (1) the additional 6 events include, 1999 through 2004, and (2) the use of EMA in fitting the curve.

We adopted the unregulated frequency curves for the 2 analysis locations, Farmington Reservoir and Farmington, CA, shown in Figure 9 and Figure 10. These are the curves that use manually specified low outlier thresholds. The detailed parameters used to fit these curves are included in Attachment 4.



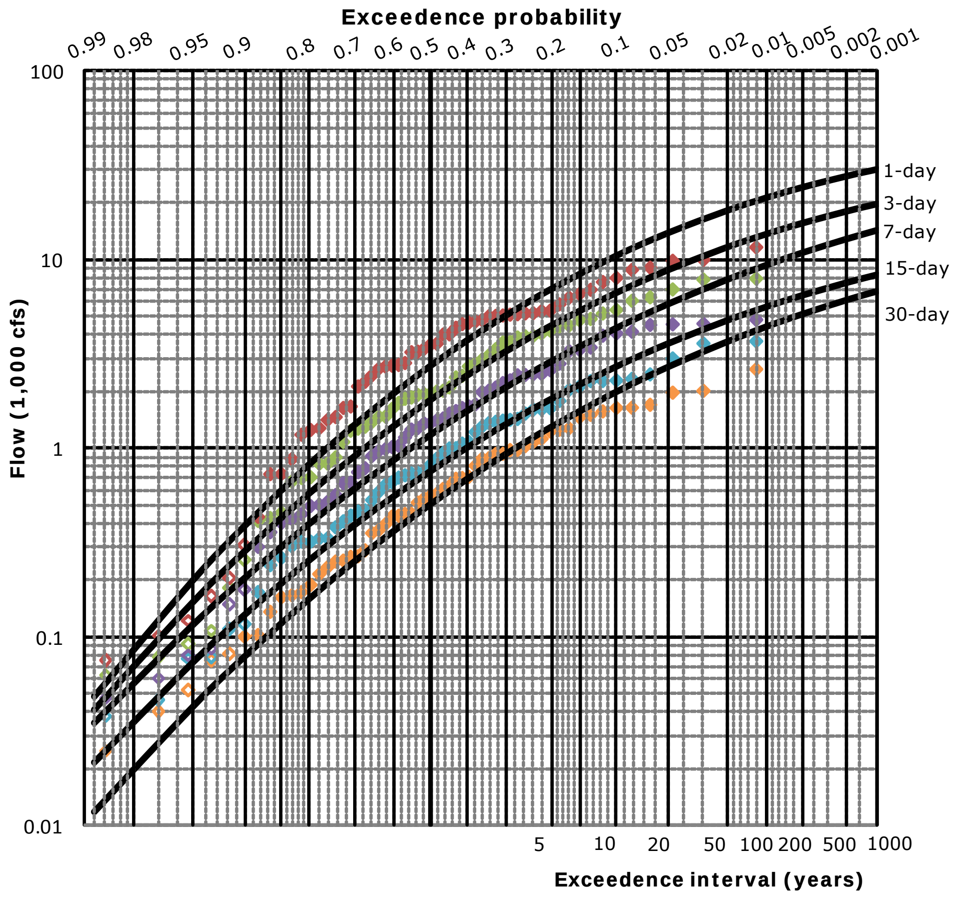
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.811	0.445	-0.692
1-day	3.321	0.507	-0.858
3-day	3.135	0.531	-0.812
7-day	2.970	0.538	-0.675
15-day	2.754	0.553	-0.733
30-day	2.561	0.556	-0.721

Notes:

- Median plotting positions.
- Drainage area: 212 sq. miles.
- Period of systematic record: 1951-2008.
  (Peak flow data intermittent 1952-2004).
- Record lengths
  Peak flows: 53 years.
  Volumes: 58 years.
- Regional skew values developed by USGS.
- Low outliers for volumes: 8 smallest events.
- Hollow points are censored events.

Figure 9. Unregulated frequency curves: Farmington Reservoir



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.356	0.573	-0.849
3-day	3.186	0.545	-0.786
7-day	3.011	0.525	-0.670
15-day	2.815	0.523	-0.722
30-day	2.639	0.556	-0.695

Notes:

- Median plotting positions.
- Drainage area: 219 sq. miles.
- Period of systematic record: 1951-2008.
- Record length: 58 years.
- Regional skew values developed by USGS.
- Low outliers for 1-, 3, 7, and 15-day volumes: 7 smallest events.
- Low outliers for 30-day volumes: 6 smallest events.
- Hollow points are censored events.

Figure 10. Unregulated frequency curves: Littlejohn Creek at Farmington, CA

Regulated flow time series development

To develop regulated flow-frequency curves, the unregulated volume-duration-frequency curves are transformed through the unregulated-regulated flow transform. The unregulated-regulated flow transform captures the system's response to large, varied events, and is created using the unregulated and regulated flow time series. To develop the regulated flow time series we took selected historical events from the unregulated flow time series and simulated those in the regulated system. In addition, scaled historical events were used to represent events larger than those seen in the historical record for definition of the flow transforms. We then compiled the maximum unregulated and regulated flows for various durations to develop the event maxima datasets.

For this analysis we developed the regulated flow time series at each analysis location by:

- Smoothing the unregulated flow time series, using those series as boundary conditions to the reservoir simulation model.
- Identifying floods-of-record (discrete events) required to develop the flow transforms.
- Scaling historical events to represent events larger than those in the historical record.
- (Task 5.1 and Task 5.2) Simulating and routing reservoir releases of historical and scaled events.

Smooth unregulated flow time series

The daily unregulated flow time series are appropriate for frequency analysis. However daily upstream and intermediate boundary conditions do not have the temporal resolution required by the CVHS procedures for assessing the effects of regulation, particularly releases as indicated on the emergency spillway release diagram (ESRD). Therefore, the daily reservoir inflows and daily estimated local flows were "smoothed" to hourly time series. This smoothing was completed using a mass balance algorithm that interpolates the shape of the hydrograph and estimates peak hourly flows while maintaining daily volumes consistent with the original time series. These smoothed times series were provided by the Sacramento District Hydrology Section for use in this analysis.

Identify floods-of-record

Events rarer than p=0.5 annual exceedence event are needed to define the flow transforms. To develop the flow transforms we used both historical events and scaled historical events. The 40 historical events used were those with 1-day volumes greater than 2,000 cfs (a threshold slightly lower than volume corresponding to the p=0.5 exceedence event.)

To select the subset of events used for scaling, we identified: (1) the 14 large flood events for the San Joaquin River basin (listed in the Comp Study historical storm matrices), and (2) the 5 largest events for Littlejohn Creek watershed (of which only the 2006 event was not included in the Comp Study matrices). We list these events in Table 6. In Table 6, column 1 lists the water year of the event, column 2 and column 3 list the associated start and end dates, column 4 lists the 1-day volume, and column 5 indicates the selection basis. On Littlejohn Creek, 4 of the 5 largest inflow events are included in the Comp Study historical storm matrix. We identified these dates by visual inspection of unregulated inflow time series provided by the Corps. The time windows defined by these dates was used for extraction of the event maxima (unregulated and regulated) for development of the flow transforms.

The Comp Study lists both a January and February event for the 1969 water year in the San Joaquin River basin. However, a large February inflow event is not present in the Farmington Reservoir unregulated inflow time series. Therefore, for this analysis we treat the 1969 flood as a single event.

Table 6. Littlejohn Creek floods-of-record scaled to develop flow transforms

Water year[1] (1)	**Start date (2)**	**End date (3)**	**1-day max volume (cfs) (4)**	**Selection basis (5)**
1998	1/26/1998	2/28/1998	11,270	Comp Study storm matrix event
2006	3/26/2006	4/30/2006	9,912	Largest inflow event
1986	1/26/1986	2/28/1986	9,555	Comp Study storm matrix event
1965	12/20/1964	1/20/1965	8,760	Comp Study storm matrix event
1956	12/20/1955	2/5/1956	8,497	Comp Study storm matrix event
1997	12/28/1996	2/12/1997	7,777	Comp Study storm matrix event
1958	3/12/1958	4/12/1958	7,272	Comp Study storm matrix event
1983	11/20/1982	3/31/1983	6,620	Comp Study storm matrix event
1982	12/27/1981	4/20/1982	6,522	Comp Study storm matrix event
1951	11/17/1950	12/31/1950	5,284	Comp Study storm matrix event
1980	1/10/1980	3/10/1980	4,921	Comp Study storm matrix event
1995	1/1/1995	3/31/1995	4,854	Comp Study storm matrix event
1967	1/20/1967	4/30/1967	4,324	Comp Study storm matrix event
1969[2]	1/10/1969	3/10/1969	3,707	Comp Study storm matrix event
1978	1/4/1978	3/20/1978	3,447	Comp Study storm matrix event

1. Events are in order of increasing 1-day flow volume
2. For the purposes of this analysis we treat the 1969 flood as 1 single event.

Scale historical floods

In addition to the 40 historical floods-of-record, events larger than these recorded were required to develop the flow transforms throughout the full range of interest. To obtain those, we scaled the time series for the subset of historical event s listed in Table 6 uniformly by factors at 0.2 intervals from 1.2 through 3.0 for use in simulating reservoir releases. This yielded a total of 10 scaled time series for each event. Both the unregulated reservoir inflow and estimated local flow time series were scaled uniformly to maintain the coincidence and timing of the system.

Scaled historical events were used only for the development of the flow transforms. The events were not used for fitting the unregulated flow frequency curves. This use of scaled historical events is consistent with the guidance in EM 1110-2-1415.

Simulate and route historical and scaled floods

We simulated reservoir operation and routed flows for both the historical floods-of-record and scaled historical events using the computer program HEC-ResSim, version 3.1 Beta III, developed by the USACE Hydrologic Engineering Center (HEC). Given a reservoir network, operating rules and constraints, and a set of inflows and downstream local flows, HEC-ResSim routes the flows through the system and simulates releases for the reservoirs. These releases are based on the rules and constraints defined in the water control manual.

An HEC-ResSim reservoir network includes representation of the physical properties of the reservoirs and links from reservoirs to downstream points of interest. Hydrologic routing model parameters are required to represent the movement of the flood wave between nodes in the network. Required physical properties include elevation-volume relationships, elevation-maximum outflow relationships, and physical limitations of the reservoir outlets.

The operating rules defined for a reservoir for HEC-ResSim include release functions based on reservoir pool elevation, reservoir inflow, and downstream flow constraints. Rate of change constraints are also included in the operation rule sets. For Littlejohn Creek, Farmington Reservoir operates to meet downstream flow constraints at Farmington, CA, which is just below the inflow from the Duck Creek diversion, approximately 3.5 miles downstream of the reservoir.

Simulate reservoir operation

For this analysis, we used the representation of the Littlejohn Creek system in HEC-ResSim developed by the Corps; that will be used for the CVHS. This includes a representation of the network and the reservoir operation rules. The HEC-ResSim schematic of the Littlejohn Creek system is shown in Figure 11. The major features of the network shown in Figure 11 are: Farmington Reservoir, the diversion from Duck Creek, and the reservoir control point at Farmington, CA.

For reference, Farmington Reservoir is operated to maintain flows in Littlejohn Creek at Farmington, CA, below 2,000 cfs. The complete set of operating rules is defined in the Farmington Reservoir water control manual (USACE 2004).

With this model, we simulated the 15 historical floods-of-record and associated scaled events for a total of 165 simulations. Consistent with the standard-of-practice for such analysis, for the reservoir routings, we used only the dedicated flood control storage space for the attenuation of the reservoir inflows. Thus, at the start of the simulation, the reservoir water surface elevation equals the elevation of the bottom of the flood control pool. The simulation time step for this analysis is 1 hour.

After completing the reservoir simulations, we reviewed the results from the HEC-ResSim computer program. We found that the simulated releases were consistent with our knowledge of the system operation and water control manual.

Route reservoir releases

We used Muskingum routing to route flows on Littlejohn Creek. A detailed channel model of Littlejohn Creek does not currently exist. Although the *Procedures document* calls for the hydraulic routing of reservoir releases, we found that Littlejohn Creek can be adequately simulated with hydrologic routing because: (1) the analysis locations on Littlejohn Creek are not affected by backwater and therefore do not require evaluation of stages to develop regulated flow-frequency curves, and (2) the reservoir release hydrographs do not rise quickly. The results from the reservoir simulation and routing are provided on a DVD with the original report.

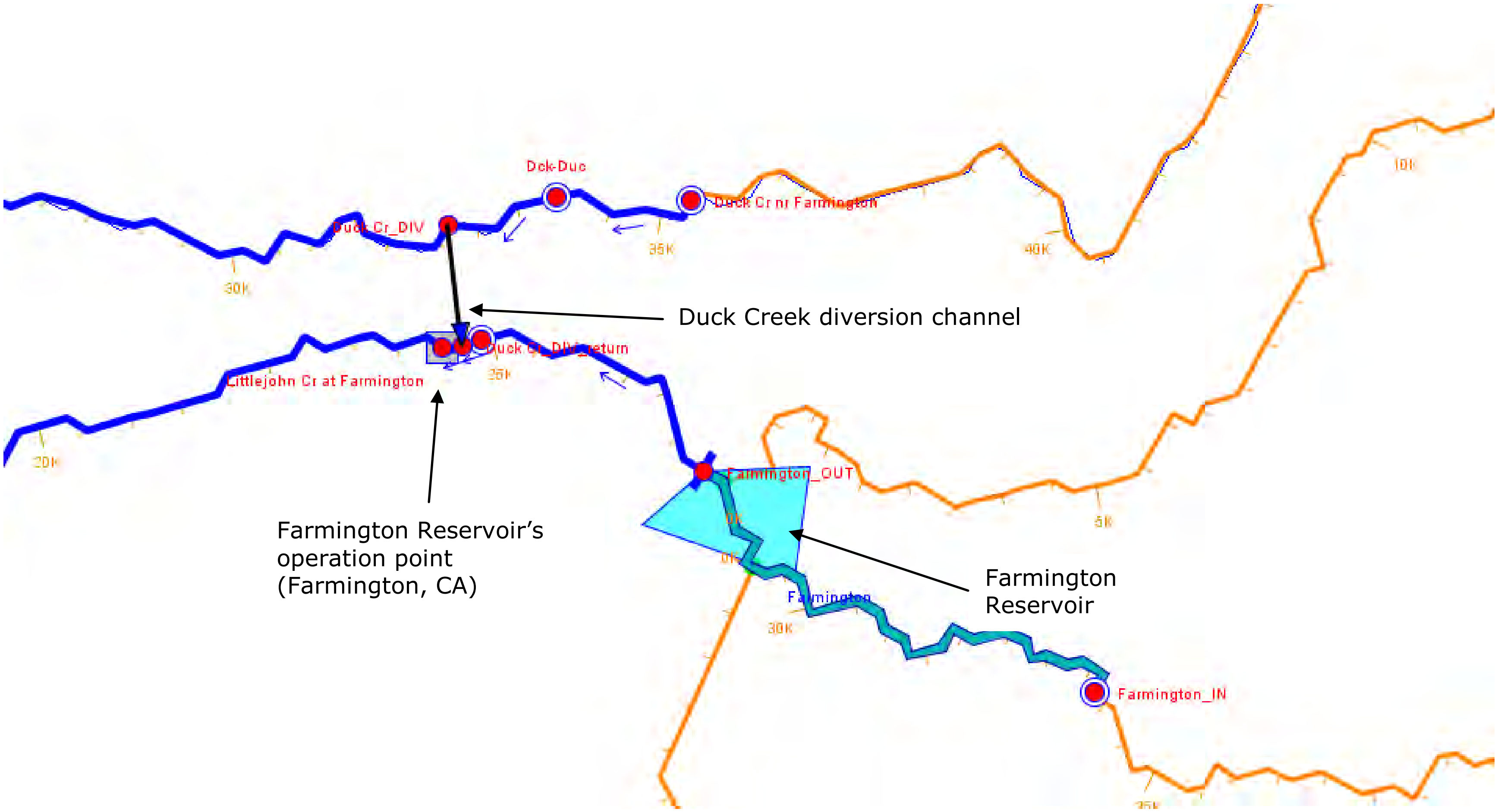


Figure 11. Screenshot of HEC-ResSim system schematic: Littlejohn Creek system

Flow transform fitting and application

Once the regulated flow time series were developed, the next step was to pair, by event, the unregulated and regulated flow time series. Using these pairings, the event properties, such as the volumes for given durations, and in the case of the regulated time series, peak flows, were identified. The result of this pairing and identification was the event maxima dataset. Specifically, the event maxima dataset consists of unregulated and regulated flows of various durations for a given historical or scaled historical event.

Once the event maxima datasets were compiled, a transform curve was fitted to develop the unregulated-regulated flow transforms. This curve translated the unregulated flow of a given quantile to the corresponding regulated flow for that same quantile. This process is illustrated in Figure 12.

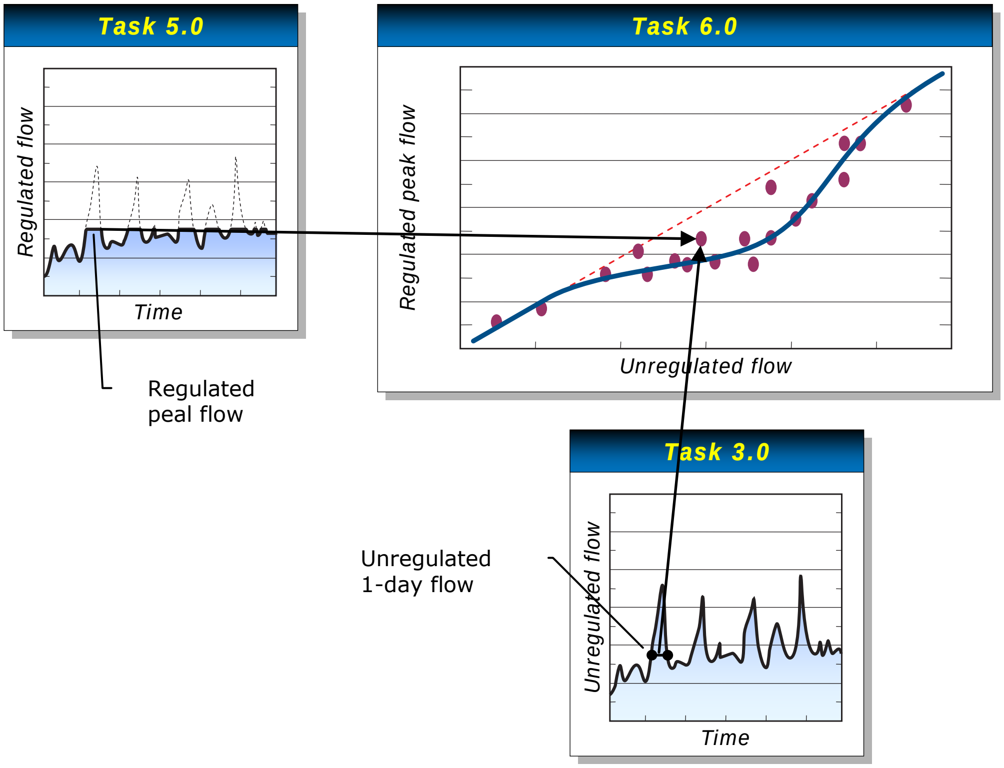


Figure 12. Flow transform development process

For the unregulated-regulated flow transform, the regulated flow value used was the peak flow. The unregulated flow value was the unregulated flow corresponding to the critical duration for that analysis location. The critical duration was found through an analysis of unregulated and regulated flows for historical and scaled historical events.

Additional transform curves were fitted to develop the family of characteristic curves. These curves identified the associated regulated volume duration characteristics of a given peak regulated flow.

For this analysis, we developed the flow transforms by:

- (Task 6.1) Identifying unregulated and regulated event maxima for the floods-of-record.
- (Task 6.2) Fitting the unregulated-regulated flow transform for each duration of interest.
- Determining the critical duration to identify the appropriate unregulated-regulated transform to use at each analysis location.
- Fitting the family of characteristic curves.
- Reviewing and accepting the flow transforms.

We then applied the flow transforms to the unregulated frequency curves to develop the regulated flow-frequency curves (Task 6.4).

Identify event maxima datasets

We identified the event maxima datasets using inspection and HEC-DSS utilities. For each analysis location, we:

- Identified the properties of the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations for unregulated flows associated with the floods-of-record. The durations we used are consistent with those specified in the *Technical procedures document* for analyzing critical duration.
- Identified the peak regulated flows from the regulated flow time series of the historical floods-of-record and scaled historical events. Note that here, peak regulated flow corresponds to the maximum hourly value regulated flow time series, and not a true instantaneous peak.
- Identified the properties of the 1-, 3-, 7-, 15-, and 30-day durations for regulated flows associated with the historical floods-of-record and scaled historical events. We did not include all the durations used in the critical duration analysis consistent with those specified in the *Technical procedures document* and the current standard-of-practice for flow-frequency analysis.

The event maxima datasets are tabulated in an MS Excel file on a DVD provided with the original report. The tabulated information lists each historical and scaled historical event used in this analysis and the associated volumes for the (1) unregulated flow volumes corresponding to the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations, and (2) regulated flow volumes corresponding to the peak, 1-, 3-, 7-, 15-, and 30-day durations.

Note that the unregulated event maxima do not include diversions from Duck Creek, while the regulated event maxima include diversions from Duck Creek.

Fit unregulated-regulated flow transforms

We developed the unregulated-regulated flow transforms for the 2 analysis locations by fitting transform curves through the pairs of event unregulated volumes and regulated peak flows. The unregulated volumes used were the average flows associated with the durations previously noted. We fitted these curves to the data pairs of historical and scaled events using the robust locally weighted scatterplot smoothing (LOWESS) regression technique. (The LOWESS procedure is detailed in the *Technical procedure document*.)

Here, we fitted these transforms for the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations. The event maxima datasets include both historical and scaled events to define the extreme end of the flow transform curves. Fitting of the transforms are detailed in Attachment 5.

The CVHS analysis procedure requires 1 single unregulated-regulated transform for statements of probability. To identify which duration is most appropriate, the critical duration for the given analysis location must be determined as described in the next subsection.

Determine critical duration

We determined critical duration at each analysis location by: (1) applying the unregulated-regulated flow transforms to the unregulated flow–frequency curves to develop hypothetical regulated flow-frequency curves, and (2) identifying the duration of the unregulated annual maximum series that consistently estimates the largest flow for each probability. In selecting the critical duration, we considered both the "goodness of fit" of each transform and which duration estimates the greater peak regulated flows. This procedure is described in more detail in Attachment 5.

From this analysis we determined that the critical duration at Farmington Reservoir and at Farmington, CA, is 10 days. Thus, the appropriate unregulated-regulated flow transforms used in this analysis were associated with this duration. The critical duration associated with the downstream operation point is shorter than that of the reservoir because of the effects of local flow.

After determining the critical duration associated with each analysis location, we reviewed and adjusted the unregulated-regulated flow transforms initially fitted with the LOWESS procedure as detailed in Attachment 5. We then adopted the flow transforms for Farmington Reservoir and Farmington, CA, shown in Figure 13 and Figure 15. In Figure 13 and Figure 15, some scaled historical event maxima for more common events have regulated peaks exceeding the channel capacity (2,000 cfs) because of large local flows and diversions from Duck Creek.

Fit family of regulated characteristic curves

We developed the families of regulated characteristic curves for Farmington Reservoir and at Farmington, CA, by fitting most likely curves through the pairs of event regulated volumes as average flows and regulated peak flows, similar to the procedure we used to fit the unregulated-regulated transforms. The data pairs (from the event maxima datasets) we used include both historical and scaled events to define the extreme ends of the flow transform curve.

The family of regulated characteristic curves for Farmington Reservoir and Farmington, CA, are shown in Figure 14 and Figure 16, and are detailed in Attachment 6. These curves associate regulated peak flows to regulated characteristic volumes. We fitted characteristic curves for the 1-, 3-, 7-, 15-, and 30-day durations. We compare these families of curves in Figure 17.

On Littlejohn Creek, the typical duration of releases from Farmington Reservoir for events in the given range of interest is less than 15 days.

Therefore we include the 15-day and 30-day characteristic curves here for completeness, and in keeping with the CVHS procedures.

Review and adopt flow transforms

After fitting the flow transforms and characteristic curves, we reviewed the resulting functions for consistency. Specifically, we compared each transform to (1) the transforms associated with different durations at the same analysis location, and (2) the transforms at the other analysis location. We found:

- The unregulated-regulated flow transforms were consistent between analysis location, i.e., the regulated peak flow for a given quantile at the downstream location was greater than that of the upstream location.
- At both analysis locations, the families of regulated characteristic curves were consistent between durations, i.e., they do not cross. This is expected.
- The fit of the curves at Farmington, CA, was sensitive to large diversions from Duck Creek such as those in the 1995 event and its corresponding scaled events. For scaled versions of this event, the diverted exceeded channel capacity before the Farmington Reservoir flood control pool was filled.

Based on this review, we adopted these flow transforms for the 2 analysis locations.

Apply flow transforms

We developed a regulated peak flow-frequency curve and the associated regulated 1-, 3-, 7-, 15-, and 30-day volumes at Farmington Reservoir and at Farmington, CA, by combining the appropriate information from the unregulated frequency curves, the flow transforms, and the families of regulated characteristic curves. The regulated flow-frequency curves for Farmington Reservoir and Farmington, CA, are shown in Table 7 and Table 9 and their associated volumes are tabulated in Table 8 and Table 10.

To apply the flow transforms and develop regulated flow-frequency curve associated volumes at each analysis location we:

- Identified the unregulated flow quantiles associated with the critical duration that correspond to the probabilities of interest.
- Identified the regulated peak flows that correspond to the flow quantiles identified in the previous step using the flow transform.
- Identified the regulated flow characteristics that correspond to the regulated peaks identified in the previous step using the family of regulated characteristic curves.

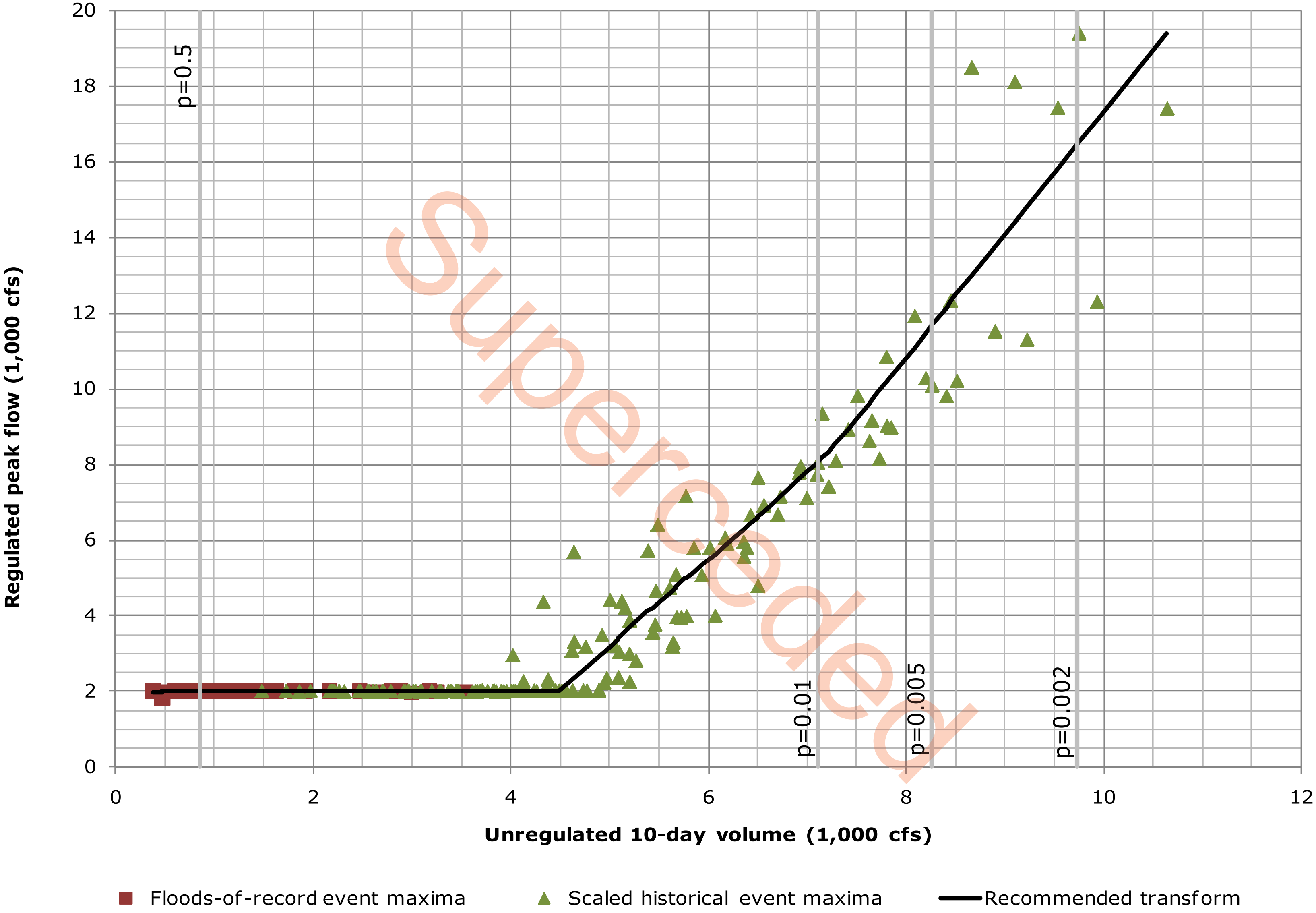


Figure 13. Unregulated-regulated flow transform: Farmington Reservoir

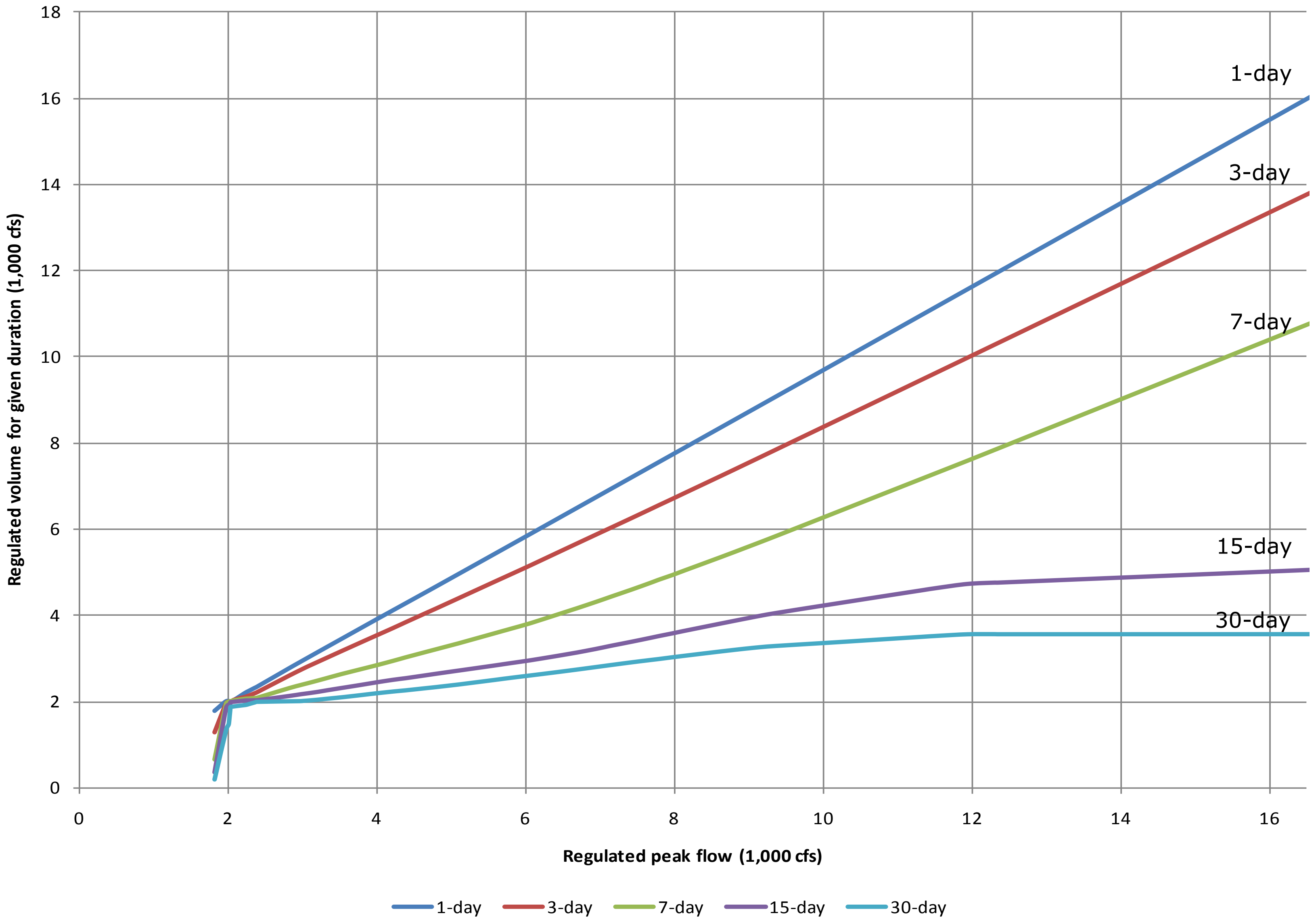


Figure 14. Family of regulated characteristic curves: Farmington Reservoir

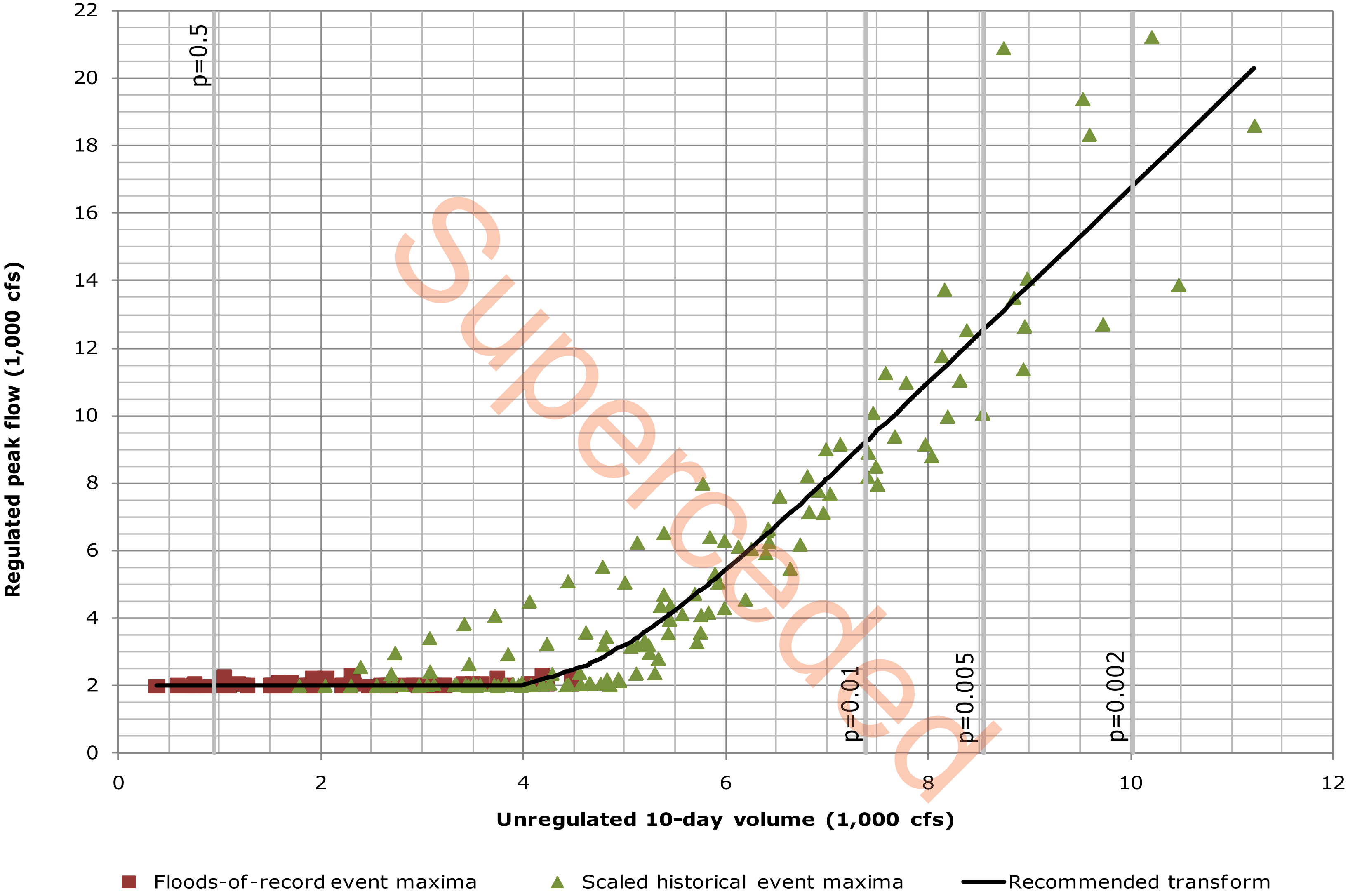


Figure 15. Unregulated-regulated flow transform: Littlejohn Creek at Farmington, CA

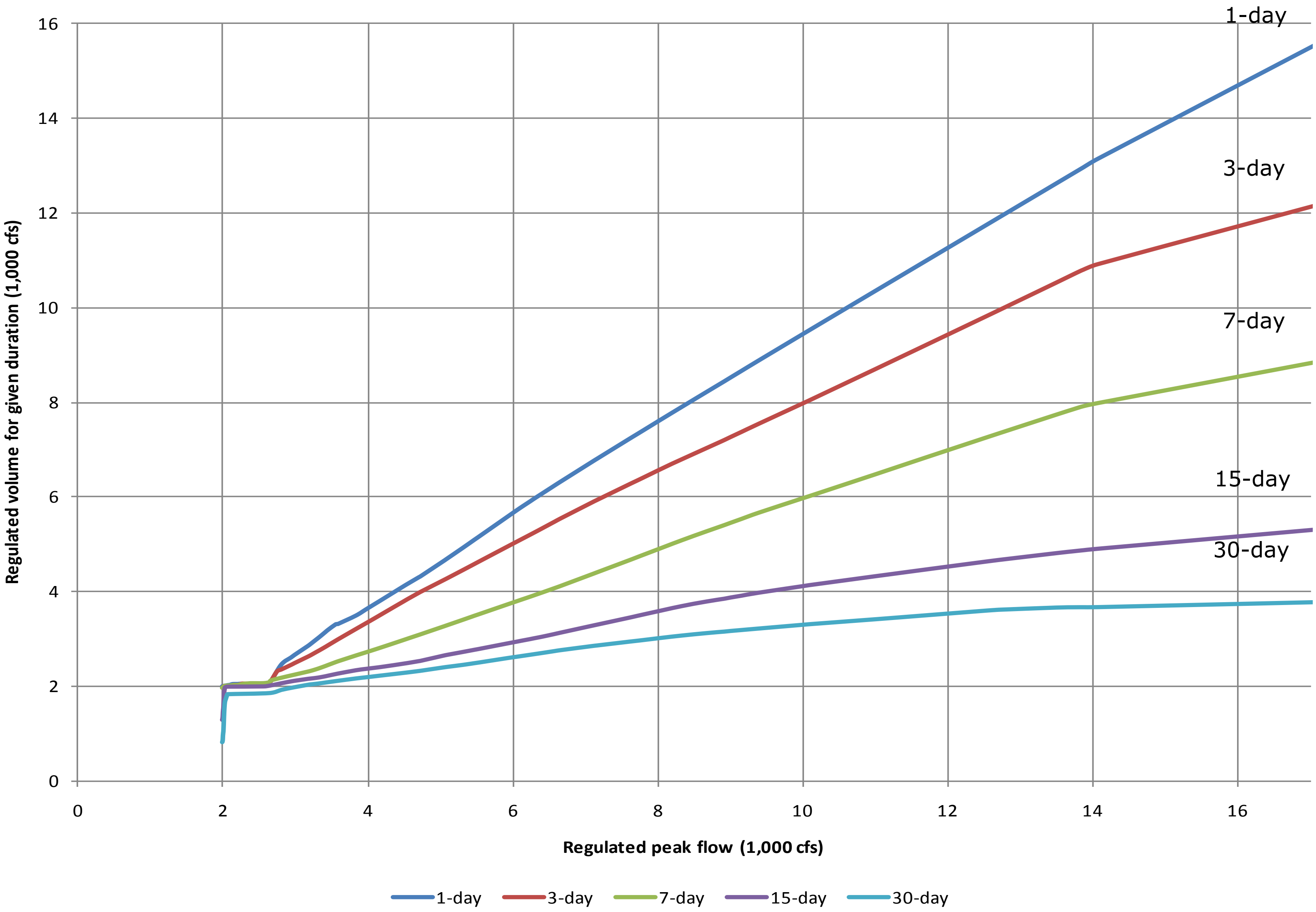

1-day
3-day
7-day
15-day
30-day
16
14
12
10
8
6
4
2
0
0
2
4
6
8
10
12
14
16
Regulated volume for given duration (1,000 cfs)
Regulated peak flow (1,000 cfs)
1-day
3-day
7-day
15-day
30-day

Figure 16. Family of regulated characteristic curves: Littlejohn Creek at Farmington, CA

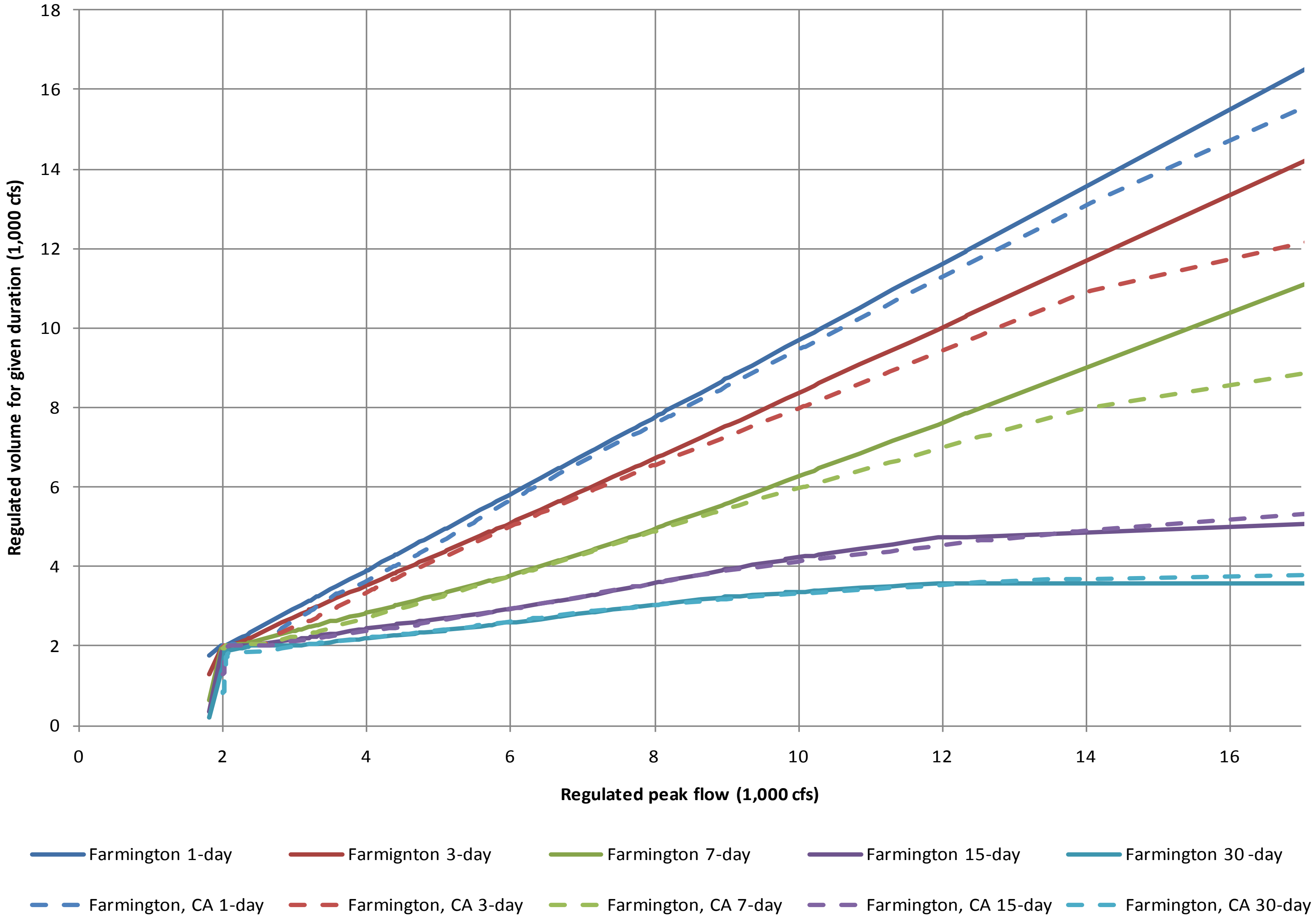


Figure 17. Comparison of the families of characteristic curves for Farmington Reservoir and Farmington, CA

Table 7. Regulated peak flow-frequency quantiles: Farmington Reservoir

Annual exceedence probability (1)	1/annual exceedence probability (2)	Regulated peak flow (cfs) (3)
0.500	2	2,000
0.200	5	2,000
0.100	10	2,000
0.050	20	2,000
0.020	50	5,360
0.010	100	8,077
0.005	200	11,671
0.002	500	16,444

Table 8. Regulated peak flow values and associated volumes: Farmington Reservoir

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration)				
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)	30-day (cfs) (7)
0.500	2,000	2,000	1,994	1,987	1,910	1,491
0.200	2,000	2,000	1,994	1,987	1,910	1,491
0.100	2,000	2,000	1,994	1,987	1,910	1,491
0.050	2,000	2,000	1,994	1,987	1,910	1,491
0.020	5,360	5,213	4,601	3,469	2,776	2,458
0.010	8,077	7,833	6,783	4,996	3,614	3,052
0.005	11,671	11,307	9,746	7,397	4,662	3,536
0.002	16,444	15,928	13,704	10,695	5,043	3,563

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at Farmington Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Table 9. Regulated peak flow-frequency quantiles: Littlejohn Creek at Farmington, CA

Annual exceedence probability (1)	1/annual exceedence probability (2)	Regulated peak flow (cfs) (3)
0.500	2	2,000
0.200	5	2,000
0.100	10	2,000
0.050	20	2,633
0.020	50	5,964
0.010	100	9,231
0.005	200	12,548
0.002	500	16,839

Table 10. Regulated peak flow values and associated volumes: Littlejohn Creek at Farmington, CA

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration)				
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)	30-day (cfs) (7)
0.500	2,000	2,000	1,967	1,960	1,296	827
0.200	2,000	2,000	1,967	1,960	1,296	827
0.100	2,000	2,000	1,967	1,960	1,296	827
0.050	2,633	2,073	2,073	2,073	2,016	1,869
0.020	5,964	5,622	4,978	3,742	2,923	2,616
0.010	9,231	8,741	7,430	5,576	3,943	3,211
0.005	12,548	11,773	9,833	7,268	4,649	3,613
0.002	16,839	15,385	12,070	8,790	5,291	3,781

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at Farmington Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Expected hydrograph properties

The expected (design) hydrograph for a given exceedence probability is a Farmington Reservoir outflow hydrograph with a peak flow that matched the regulated flow-frequency curve (as shown in Table 7) and with associated volumes matching those from the family of characteristic curves corresponding to the given regulated peak flow (as shown in Table 8). The properties of the expected hydrographs for the p=0.5, p=0.2, p=0.1, p=0.05, p=0.02, p=0.01, p=0.005, and the p=0.002 exceedence probabilities are shown in Table 11.

An expected hydrograph can be formed by applying these properties to a specific hydrograph shape. As part of future work, we will identify specific historical event patterns to which the expected hydrograph properties can be applied. For this identification, we will follow the example event selection procedure provided in the *CVHS Product uses* document (USACE 2009c).

Options for expected hydrograph development and application using study products were submitted by Ford Engineers to the Corps on June 23, 2010. From that memorandum, the Corps selection Option 1: Selected event-based reservoir release hydrographs.

Table 11. Expected hydrograph properties: Farmington Reservoir outflow

Annual exceedence probability of regulated peak flow (1)	**1/annual exceedence probability of peak flow (2)**	**Regulated peak flow (cfs) (3)**	**Associated volumes[1] (as average flow for given duration)**				
			1-day (cfs) (4)	**3-day (cfs) (5)**	**7-day (cfs) (6)**	**15-day (cfs) (7)**	**30-day (cfs) (8)**
0.500	2	2,000	2,000	1,994	1,987	1,910	1,491
0.200	5	2,000	2,000	1,994	1,987	1,910	1,491
0.100	10	2,000	2,000	1,994	1,987	1,910	1,491
0.050	20	2,000	2,000	1,994	1,987	1,910	1,491
0.020	50	5,360	5,213	4,601	3,469	2,776	2,458
0.010	100	8,077	7,833	6,783	4,996	3,614	3,052
0.005	200	11,671	11,307	9,746	7,397	4,662	3,536
0.002	500	16,444	15,928	13,704	10,695	5,043	3,563

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at Farmington Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Results

The results of this frequency analysis include:

- Unregulated frequency curves for Farmington Reservoir (as shown in Figure 9).
- Unregulated frequency curves for Littlejohn Creek at Farmington, CA (as shown in Figure 10).
- Unregulated-regulated flow transform for Farmington Reservoir (as shown in Figure 13).
- Regulated flow-frequency curve and associated volumes for Farmington Reservoir (as shown in Table 7 and in Table 8).
- Unregulated-regulated flow transform for Littlejohn Creek at Farmington, CA (as shown in Figure 15).
- Regulated flow-frequency curve and associated volumes for Littlejohn Creek at Farmington, CA (as shown in Table 9 and in Table 10).
- Expected hydrograph properties for Farmington Reservoir (as shown in Table 11).

In addition, these intermediate data are included with the original report on DVD:

- HEC-DSS time series of the floods-of-records.
- HEC-DSS time series of the scaled historical floods.
- HEC-DSS time series of developed local flows below Farmington Reservoir (detailed in Attachment 2).
- The tabulated event maxima datasets for the 2 analysis sites.
- Simulated reservoir releases and routed flows from the HEC-ResSim reservoir simulation model.
- Tabulated unregulated-regulated flow transforms for the 2 analysis sites.
- Tabulated families of regulated characteristic curves for the 2 analysis sites.

References

Beard, Leo R. (1962). *Statistical methods in hydrology.* Hydrologic Engineering Center, US Army Corps of Engineers, Davis, CA.

Bradley, Allen A. Jr., and Potter, Kenneth W. (2004). *PVSTATS, user manual version 3.1.* University of Wisconsin-Madison, Department of Civil and Environmental Engineering, Madison, WI.

Cleveland, William S. (1979). "Robust locally weighted regression and smoothing scatter plots." *Journal of the American Statistical Association*, 74(368) 829-836.

Cleveland, William S. (1985). Lowess.f [Fortran file]. Bell Laboratories. Murray Hill, NJ.

Cohn, Tim. (2007). PeakfqSA, version 0.937 [Software]. <http://www.timcohn.com/TAC_Software/PeakfqSA/>.

Goldman, David M. (2001). *"Quantifying uncertainty in estimates of regulated flood frequency curves." State of the practice – proceedings of the World Water and Environmental Resources Congress,* ASCE, Reston, VA.

Helsel, D. R., and Hirsch, R. M. (2002). *Statistical methods in water resources,* US Geological Survey, Reston, VA.

Interagency Advisory Committee on Water Data (IACWD). (1982). *Guidelines for determining flood flow frequency, Bulletin 17B.* US Geological Survey, Reston, VA.

US Army Corps of Engineers (USACE). (1983). *New Hogan Dam and Lake, Littlejohn Creek, California, Water control manual, Appendix III to Master water control manual, San Joaquin River Basin, California,* Sacramento District, Sacramento, CA.

USACE. (1990). *Littlejohn Creek, California: Reconnaissance report*, Sacramento District, Sacramento, CA.

USACE. (1993). *Hydrologic frequency analysis, EM 1110-2-1415*, Washington, D.C.

USACE. (1994). *Engineering and design-hydrologic engineering studies design, EP 1110-2-9*, Washington, D.C.

USACE. (1997). *Hydrologic engineering requirements for reservoirs, EM 1110-2-1420*, Washington, D.C.

USACE. (2002). *Sacramento and San Joaquin river basins comprehensive study, December 2002 interim report* ("Comp study"), USACE, Sacramento District, Sacramento, CA.

USACE. (2004). *Farmington Dam and Reservoir, Littlejohn Creek, California, Water control manual, Appendix IV to Master water control manual, San Joaquin River Basin, California,* Sacramento District, Sacramento, CA.

USACE. (2009a). *Central Valley hydrology study (CVHS): Technical procedures document* ("Technical procedures document"), prepared by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (2009b). *CVHS product uses* ("Uses document"), prepared by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (2009c). *Sacramento and San Joaquin river basins: Procedures for hydrologic analysis ("Procedures document")*, prepared by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (2010). *Hydrologic engineering management plan for the Lower San Joaquin River feasibility study, prepared by David Ford Consulting Engineers, Inc., Sacramento, CA.*

USACE. (Forthcoming 1). *Central Valley hydrology study (CVHS): Technical procedures document*, in preparation by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (Forthcoming 2). *Central Valley hydrology study (CVHS) technical procedures document Attachment B: Unregulated time series development*, in preparation by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (Forthcoming 3). *Central Valley hydrology study (CVHS) Technical procedures document Attachment C: Regulated time series development*, in preparation by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (Forthcoming 4). *Central Valley hydrology study (CVHS) Technical procedures document Attachment D: Flow frequency analysis*, in preparation by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (Forthcoming 5). *Central Valley hydrology study (CVHS) Technical procedures document Attachment E, Development of flow and stage transforms*, in preparation by David Ford Consulting Engineers, Inc., Sacramento, CA.

Attachment 1: Correspondence of procedural steps

Table 12 shows how the procedural steps in this document correspond to the steps in the *Procedures document* and the *Technical procedures document*.

Table 12. Correspondence of procedural steps for the LSJR FS, the CVHS "Procedures document," and the CVHS "Technical procedures document"

This step in the hydrologic analysis at Farmington Reservoir… (1)	**Corresponds to this action in the *Procedures document*… (2)**	**And / or this action in the *Technical procedures document*… (3)**
Develop unregulated flow time series	Task 3.0	Attachment B: Unregulated flow time series development
• Estimate local flows	Task 3.2	• Application and distribution of local flows
• Route and complete unregulated flow time series at analysis locations	Task 3.3	• Procedures for routing flows through the system
Develop unregulated frequency curves	Task 4.0	Attachment D: Frequency analysis
Develop regulated flow time series	Task 5.0	Attachment C: Regulated time series development
• Identify floods-of-record • Scaling of historical reservoir inflows	Task 6.2	Attachment E: Development of flow and stage transforms • Determination of historical event scaling for extrapolating unregulated-regulated flow transform
• Simulation of reservoir releases for historical and scaled events	Task 5.1, Task 6.2	• Procedures for routing regulated flows through the system
Develop flow transforms	Task 6.0	Attachment E: Development of flow and stage transforms
• Identify annual maximum series	Task 6.1	—
• Assess reservoir critical duration	—	Attachment E: Development of flow and stage transforms • Identification of critical duration at analysis points Attachment F: Procedure for critical duration calculation

This step in the hydrologic analysis at Farmington Reservoir… (1)	**Corresponds to this action in the *Procedures document*… (2)**	**And / or this action in the *Technical procedures document*… (3)**
• Fit unregulated-regulated flow transform • Fit family of regulated characteristic curves	Task 6.3	Attachment E: Development of flow and stage transforms • Procedure for fitting a "most likely" transform through the datasets
• Apply flow transforms to develop regulated-flow-frequency curves	Task 6.4	—
Develop expected hydrographs[1]	—	—

Notes:

1. Options for expected hydrograph development using study products were submitted by Ford Engineers to the Corps on June 23, 2010. From that memorandum, the Corps selection Option 1: Selected event-based reservoir release hydrographs.

Attachment 2: Littlejohn Creek local flow development

Overview

For Littlejohn Creek, we estimated local flows for the area between Farmington Reservoir and Farmington, CA, shown in Figure 8. For this area, we used 2 options to estimate local flow:

- Option 1. Direct calculation of local flow.
- Option 2: Estimation of local flow as a function of Farmington Reservoir inflow. Note: the Corps currently estimates local flow as 0.04 times reservoir inflow.

Option 1 is the most accurate option for local flow estimation. To determine which of the other 2 options for local flow estimation is more appropriate to use, we:

- Reviewed the streamgage and reservoir inflow data provided by the Corps. In Table 13 we list the streamgages that were used in estimating local flows on Littlejohn Creek. Column 1 lists the streamgage ID whose corresponding name is listed in column 2, column 3 lists the data type (e.g., daily or hourly), column 4 lists the applicable time period of the streamgage data, and column 5 lists notes on the data.
- Coordinated with Corps staff regarding streamgage data quality.
- Identified the data type (e.g., daily or hourly) of the provided data.
- Identified the overlapping time periods for each streamgage by time step.
- Estimated local flow by direct calculation (Option 1).
- Compared the directly calculated local flow time series to Farmington Reservoir inflows.
- Identified, for Option 2, alternative functions for estimating local flow including:
  - Direct multipliers based on ratios of peak flows for selected large events.
  - Direct multipliers based on drainage area ratios.
  - Linear functions determined by regression analysis.
  - Exponential functions determined by regression analysis.
  - Linear functions of logarithmic transforms of flow determined by regression.
- Estimated local flow time series using the possible functions identified.
- Estimated a local flow time series using the reservoir inflow and the 0.04 multiplier used by the Corps.
- Compared the estimated local flow time series to the directly calculated local flow time series.

- Identified the function for each option that most reasonably estimates local flows.

Table 13. Streamgages reviewed for use in estimating local flows on Littlejohn Creek: data were provided by Corps on 6/22/2010 as part of the CVHS.

USGS or CDEC ID (1)	Streamgage name (2)	Data type (3)	Time period (water year) (4)	Notes (5)
—	Farmington Reservoir unregulated inflow	Daily	1951-2009	Values computed by Corps.
FRM	Farmington Dam (reservoir outflow)	Daily	1951-2009	
		Hourly	1995-2009	Data starts January 1, 1995.
FRG	Littlejohn Creek at Farmington, CA	Daily	1948-2008	Streamgage data is influenced by regulation.
		Hourly	1995-2008	Data starts January 1, 1995. Streamgage data is influenced by regulation.
—	Duck Creek Diversion	Daily	1952-2009	Diversion began operation in 1951.
		Hourly	1995-2009	Data starts January 1, 1995.
—	Duck Creek near Farmington	Daily	1979-2009	Data starts January 1, 1979.
		Hourly	1995-2009	Data starts January 1, 1995.
—	Rock Creek at Farmington	Daily	1950-2010	Streamgage data is influenced by regulation.
		Hourly	1995-2010	Data starts January 1, 1995. Streamgage data is influenced by regulation.

Event selection for local flow estimation analysis

As previously noted, local flows developed were used to support the development of an unregulated-regulated flow transform and a family of regulated characteristic curves. A key aspect in the development of these was the scaling of the largest events, i.e., the 15 events previously indentified for Littlejohn Creek.

Thus, the local flows estimated for these large events needed to be reasonable and as accurate as possible. To assess this, we used the local flows calculated directly corresponding to the largest events possible as a basis of comparison. Specifically, we used the 1997, 1998, and 2006 water year events whenever possible.

Local flow estimation Option 1: Calculate local flows directly

The preferred option for estimating local flows was to calculate directly flows using streamgage data. In general, this was completed on Littlejohn Creek using known releases from Farmington Reservoir and the observed flows at Farmington, CA. This was completed only for the time periods when data overlap. On Littlejohn Creek this corresponds to all floods events in the period of record, except for the 1969, 1970, and 1973 water year events.

In the case of daily data, local flows were calculated directly by subtracting the reservoir releases and observed diversion diversions from Duck Creek from the gaged flows. Any resulting negative values were then set to 0. Routing of the daily observed outflows (using the 1-hour hydrologic routing model of Littlejohn Creek) was not necessary because the total travel time between Farmington Reservoir and Farmington, CA, is less than 1-day.

Accepted travel time estimates between Farmington Reservoir and Farmington, CA, are: (1) 3 hours as indicated in the Farmington Reservoir water control manual (Corps 2004), and (2) 1.7 hours as indicated by the sum of Muskingum K value from the HEC-ResSim model provided by the Corps. This shorter travel time was attributed to the availability of hourly streamgage data after 1995 used to calibrate the reservoir simulation and hydrologic routing flood model of Littlejohn Creek, and was adopted for this analysis.

In the case of hourly data, reservoir releases were first routed from Farmington Reservoir downstream to the gage at Farmington, CA. These routed releases and the observed diversions from Duck Creek were then subtracted from the observed flows to calculate local flow directly. Any resulting negative values are then set to 0. We used hydrologic routing to estimate local flows on Littlejohn Creek. Specifically, we used HEC-DSS math utilities and the Muskingum routing parameters from the CVHS HEC-ResSim model as shown in Table 14. In Table 14, column 2 lists the reach, column 3 lists the Muskingum K values in hours, column 4 lists the Muskingum X, and column 5 the number of subreaches.

In Table 15 we summarize how local flows were calculated directly by time period and data type. In Table 15, column 2 lists the data type, column 3 the overlapping time period, and column 4 the components for calculating local flows.

In Figure 18 through Figure 20 we compared the daily and hourly inferred local flows for the 1997, 1998, and 2006 water year events. (These events are the 3 largest of the overlapping time period for which we could calculate both daily and hourly local flows.) In Figure 18 through Figure 20 the daily local flows are shown in red, the hourly local flows in blue, and the daily differences in their volumes (daily local flows minus hourly local flows) in green. From these comparisons we see (1) that the timing of the hourly and daily local flows are similar, and (2) the differences in volume appear to be greatest around the largest local flows associated with the event. These differences in volumes are small compared to the total volume of unregulated inflow to Farmington Reservoir.

Table 14. Littlejohn Creek Muskingum routing parameters between Farmington Reservoir and Farmington, CA

Reach (1)	**Muskingum K (hours) (2)**	**Muskingum X (3)**	**Number of subreaches (4)**
Farmington Reservoir to Farmington, CA	1.7	0.2	1

Table 15. Summary of direct calculation of local flows on Littlejohn Creek

ID (1)	**Data type (2)**	**Overlapping time period[1] (water year) (3)**	**Calculate local flows directly by:[2] (4)**
1	Daily	1951-1968 1971-1972 1974-2008	Subtracting (1) known outflows from Farmington Reservoir and (2) observed flows from Duck Creek, via the Duck Creek diversion, from observed flows on Littlejohn Creek at Farmington, CA
2	Hourly	1996-2008	Routing known outflows from Farmington Reservoir, then subtracting (1) the routed outflows and (2) observed flows from Duck Creek, via the Duck Creek diversion, from observed flows on Littlejohn Creek at Farmington, CA

Notes:

1. Because of missing values, local flow may not be calculated directly for the entire period listed. In such cases flows are either interpolated using the directly calculated flow, or Option 2 or Option 3 depending on data availability.
2. Any resultant negative values were set to 0.

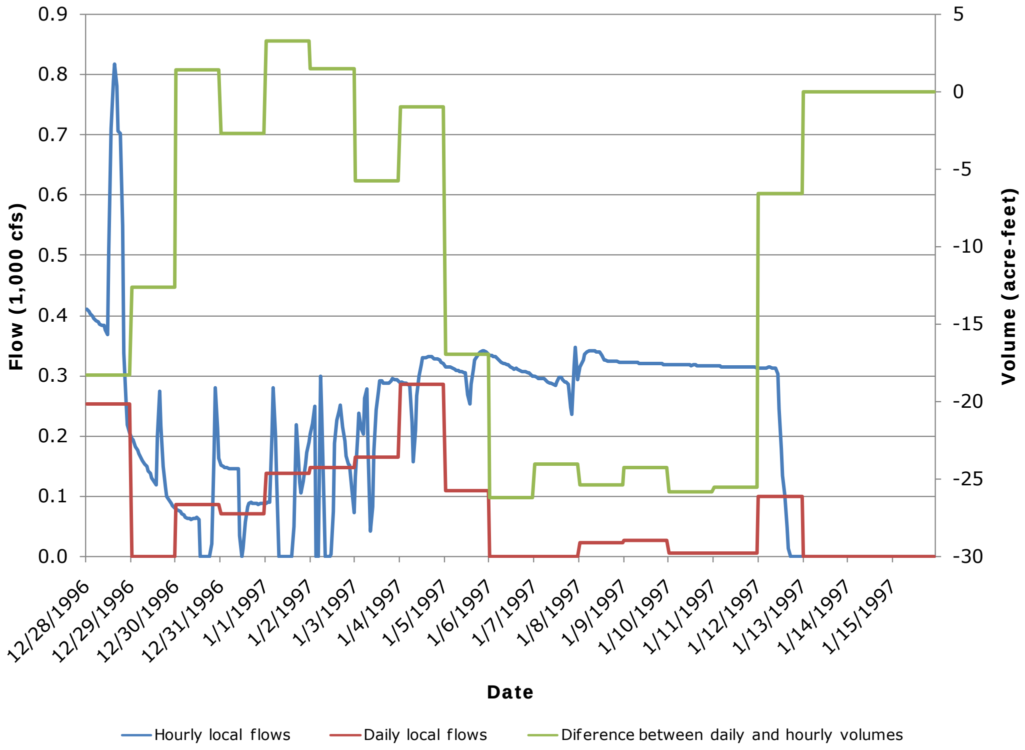


Figure 18. Littlejohn Creek 1997 event directly calculated local flows

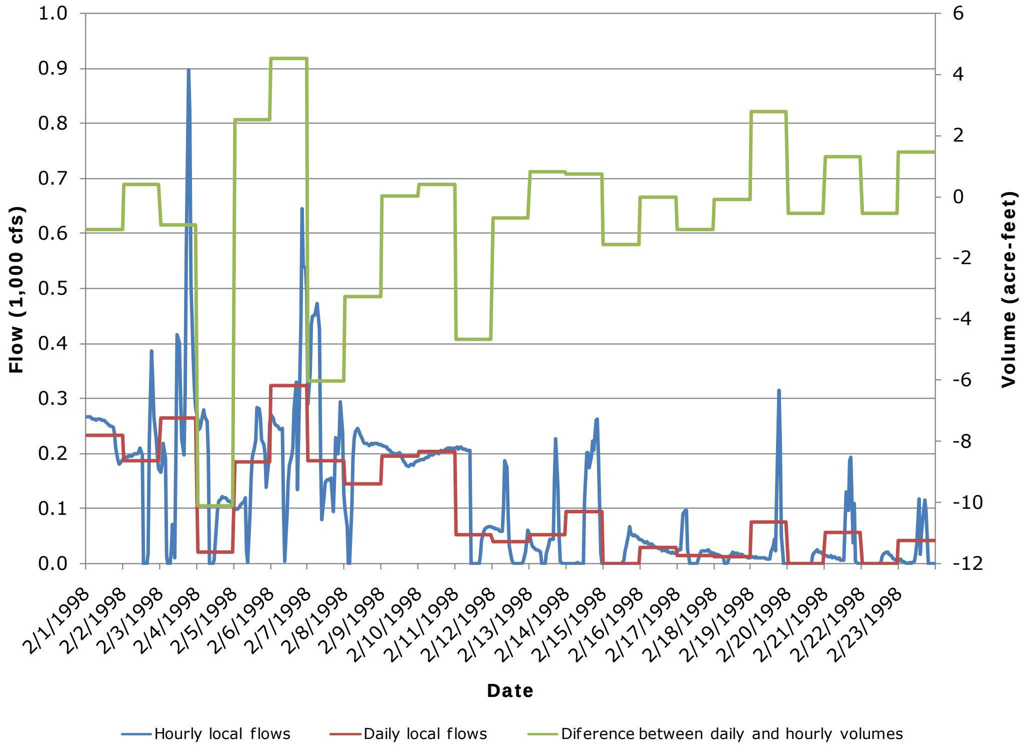


Figure 19. Littlejohn Creek 1998 event directly calculated local flows

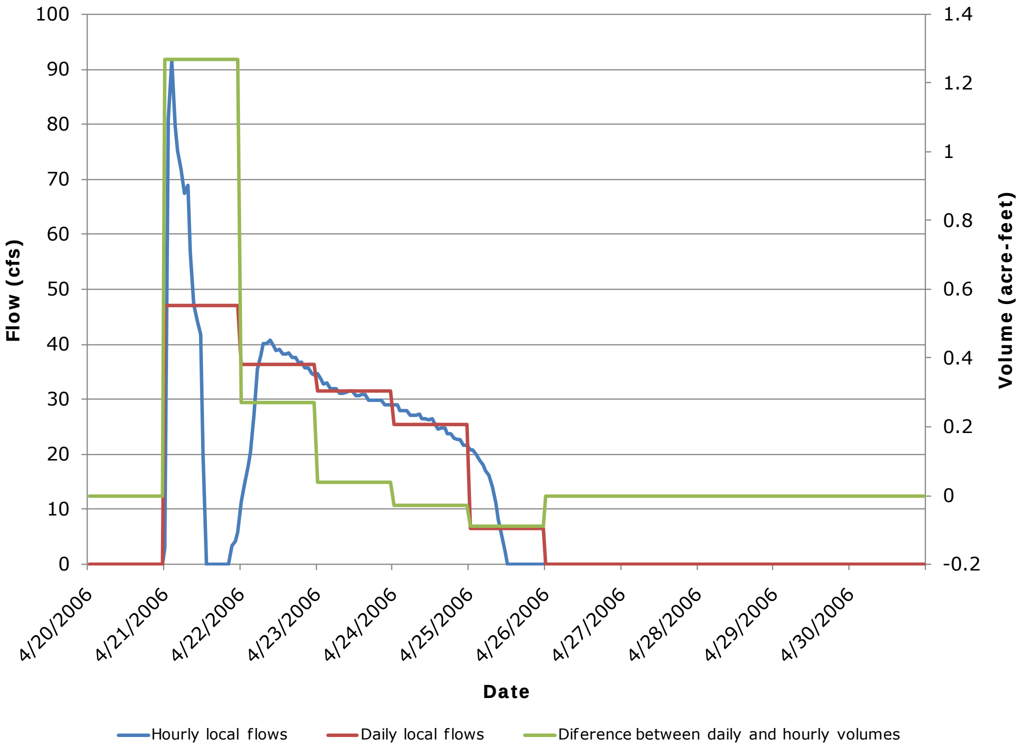


Figure 20. Littlejohn Creek 2006 event directly calculated local flows

Local flow estimation Option 2: Estimate local flows as a function of unregulated inflow to Farmington Reservoir

In the cases where local flows could not be calculated directly, we estimated local flows using reservoir inflows. As noted above, the Corps already estimates local flows using coefficients for reservoir operations on Littlejohn Creek as 0.04 times the reservoir inflow. Because the estimation of local flows is important to simulate accurately reservoir operations we need to either (1) verify the coefficients used by the Corps to estimate such flows, or (2) adopt new coefficients. We completed this task by:

- Calculating local flows directly as detailed in the previous subsection.
- Comparing the directly calculated local flow time series to observed flows on reservoir inflows for selected large events occurring in the overlapping period of record.
- Identifying an average ratio of maximum 1-day inflows to directly calculated peak local flows for selected large events.
- Estimating local flow time series using the average ratio identified as a multiplier of unregulated reservoir inflow.
- Estimating local flow time series using a drainage area ratio between the local flow area and watershed above the reservoir as a multiplier to reservoir inflows.
- Completing regression analyses that relate the directly calculated local flows to the reservoir inflow for the overlapping periods of record.

- Identifying the best fitted functions from the regression analysis for estimation of local flows.
- Estimating local flow time series using the identified functions.
- Estimating a local flow time series using the unregulated reservoir inflow and the 0.04 multiplier used by the Corps.
- Comparing the estimated local flow time series to the directly calculated local flow time series.
- Identifying the function that most reasonably estimates local flows.

Based on this analysis, we identified the best relation for estimating local flows using reservoir inflow to be the function currently used by the Corps. Thus, we estimated local flows as:

$$Q_{Local} = 0.04\left(Q_{FRM}\right) \tag{2}$$

where Q_{Local} is the local flow estimate for a given time, and Q_{FRM} is the unregulated inflow to Farmington Reservoir. The Corps estimates local flows for the purpose of real-time reservoir operations using this option (John High, personal communication, 11/9/2009) and this is the option used to estimate local flows in the Comp Study (USACE 2002).

All estimated local flows using this option were on a daily basis. We did not lag or route the estimated flows because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the Farmington Reservoir and Farmington, CA, is approximately 2 hours, which is less than the 1-day time step of the reservoir inflows.

Local flow estimation details

The selected estimation approaches, in order of best estimate of local flow, are:

- Option 1. Calculate local flow directly using known releases from Farmington Reservoir and the observed flows at Farmington, CA, routing hourly flows as necessary. Note in the case of missing streamgage data, local flows values were interpolated as needed.
- Option 2. Estimate local flow as 0.04 times the unregulated inflow to Farmington Reservoir.

We detail the development of the local flow time series for Farmington Reservoir in Table 16. Column 1 notes the time period for which the option listed in column 3 will be used to estimate local flow, and column 2 lists the time step (hourly or daily) of the developed local flow time series. We interpolated local flows using other estimated local flows as appropriate. The hourly and daily time series were combined and these finalized time series stored as hourly data in HEC-DSS.

Table 16. Local flow time series calculation details by time period

Period (date) (1)	Time step (2)	Approach to be used[1] (3)
10/1/1950—9/30/1951	Daily	Option 1: directly calculate local flow. Note that the Duck Creek diversion was not in operation during this time period.
10/1/1951-1/6/1969	Daily	Option 1: directly calculate local flow
1/7/1969-3/29/1969	Daily	Option 2: 0.04 times reservoir inflow.
3/30/1969-1/10/1970	Daily	Option 1: directly calculate local flow
1/11/1970-3/31/1970	Daily	Option 2: 0.04 times reservoir inflow.
4/1/1970-1/7/1973	Daily	Option 1: directly calculate local flow.
1/8/1973-4/5/1973	Daily	Option 2: 0.04 times reservoir inflow.
4/6/1973-5/3/1978	Daily	Option 1: directly calculate local flow.
5/4/1978-9/30/1978	Daily	Assume 0 local flow.
10/1/1978-10/31/1978	Daily	Option 1: directly calculate local flow.
11/1/1978-1/10/1979	Daily	Assume 0 local flow.
1/11/1979-4/5/1979	Daily	Option 1: directly calculate local flow.
4/6/1979-9/24/1979	Daily	Assume 0 local flow.
9/25/1979-9/30/1991	Daily	Option 1: directly calculate local flow.
10/1/1991-12/31/1991	Daily	Assume 0 local flow.
1/1/1992-12/31/1994	Daily	Option 1: directly calculate local flow.
1/1/1995-9/27/1995	Hourly	Option 1: directly calculate local flow.
9/28/1995-12/18/1995	Daily	Option 1: directly calculate local flow.
12/19/1995-12/28/2008	Hourly	Option 1: directly calculate local flow.

Attachment 3: Annual maximum series for unregulated frequency curves

Here we list the series of annual maximum unregulated volume values that we used in development of the unregulated frequency curves for Farmington Reservoir and at Farmington, CA. In addition, we include here the unregulated peak inflow annual maximum series for Farmington Reservoir. Development of a peak flow-frequency curve is not required for development of the regulated flow-frequency curves. However, we developed such curves for completeness.

Annual maximum series

For the Farmington Reservoir, the unregulated reservoir inflow time series was used as the basis of the unregulated frequency analysis. The Corps provided the finalized unregulated inflow time series for Farmington Reservoir on 7/12/2010. From this time series, we extracted the 1-, 3-, 7-, 15-, and 30-day volume data. We list these values for Farmington Reservoir in Table 17. In the table, column 1 lists the water year, and columns 2 through 11 list the date, if available, and the volume, as average flow for the given duration, in cfs. The dates listed in Table 17 correspond to the start of the duration.

To develop annual maximum series for Farmington Reservoir’s operation point on Littlejohn Creek at Farmington, CA, we combined the unregulated inflow time series with the estimated local flows by adding the 2 time series together using HEC-DSS math utilities. Note that we did not route the unregulated reservoir inflows because the travel time between the reservoir and the operation point is less than the time step of the inflows: 1 day.

Using these data, we computed the 1-, 3-, 7-, 15-, and 30-day volume-duration data using HEC-SSP version 1.1. We list these values for Farmington, CA, in Table 18. In the table, column 1 lists the water year, and columns 2 through 11 list the date, if available, and the volume, as average flow for the given duration, in cfs. The dates listed in Table 18 correspond to the start of the duration.

In addition, we reviewed the computed values for consistency. Specifically, we checked that the extracted value for a given duration is less than the values associated with each shorter duration in a given water year. For both analysis locations, we found that the computed values for each water year decrease as duration increases.

Table 17. *Farmington Reservoir annual maximum series for unregulated volume-frequency analysis*

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1951	12/8/1950	5,284	12/9/1950	4,045	12/10/1950	2,762	12/17/1950	1,605	12/17/1950	1,057
1952	3/15/1952	5,019	1/27/1952	3,351	1/29/1952	2,219	1/28/1952	1,418	1/28/1952	1,013
1953	1/14/1953	725	1/15/1953	450	1/20/1953	398	1/21/1953	316	1/28/1953	210
1954	3/17/1954	723	3/19/1954	417	3/23/1954	290	3/31/1954	166	4/14/1954	97
1955	1/1/1955	3,556	1/18/1955	1,945	1/21/1955	1,245	1/24/1955	701	1/30/1955	530
1956	12/24/1955	8,497	12/25/1955	7,413	12/28/1955	3,765	1/6/1956	2,100	1/21/1956	1,582
1957	3/5/1957	2,232	3/7/1957	1,086	3/11/1957	523	3/17/1957	263	3/30/1957	135
1958	4/3/1958	7,272	4/3/1958	6,913	4/6/1958	3,945	4/4/1958	2,234	4/12/1958	1,470
1959	2/16/1959	1,419	2/18/1959	1,218	2/22/1959	851	2/25/1959	541	3/12/1959	307
1960	2/10/1960	1,402	2/12/1960	665	2/13/1960	459	2/21/1960	286	3/2/1960	157
1961	2/2/1961	102	2/4/1961	78	2/8/1961	61	2/15/1961	38	2/15/1961	19
1962	2/15/1962	5,086	2/15/1962	2,914	2/16/1962	2,439	2/23/1962	1,370	3/10/1962	911
1963	2/13/1963	3,205	2/13/1963	1,518	2/16/1963	1,028	2/15/1963	729	4/26/1963	467
1964	1/22/1964	898	1/24/1964	749	1/27/1964	486	1/26/1964	327	2/10/1964	172
1965	12/26/1964	8,760	12/26/1964	6,357	12/28/1964	4,162	1/6/1965	2,462	1/20/1965	1,447
1966	1/30/1966	2,071	12/31/1965	1,246	1/4/1966	643	2/13/1966	438	2/27/1966	252
1967	1/22/1967	4,324	4/20/1967	2,392	4/24/1967	1,956	4/21/1967	1,368	4/29/1967	948
1968	2/21/1968	1,241	2/22/1968	699	2/23/1968	424	3/2/1968	240	3/17/1968	162
1969	1/21/1969	3,707	1/23/1969	3,459	1/27/1969	2,898	1/27/1969	2,383	2/11/1969	1,565
1970	1/21/1970	3,953	1/22/1970	3,689	1/25/1970	3,284	1/28/1970	2,577	2/6/1970	1,399
1971	11/29/1970	2,624	12/1/1970	1,482	12/5/1970	1,133	12/12/1970	590	12/28/1970	408
1972	12/25/1971	1,267	12/27/1971	891	12/31/1971	649	1/8/1972	328	1/22/1972	170
1973	2/11/1973	5,368	1/16/1973	3,565	1/18/1973	2,260	1/23/1973	1,361	2/11/1973	961
1974	3/2/1974	4,749	4/4/1974	1,931	4/7/1974	1,673	4/13/1974	1,220	4/14/1974	621

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1975	3/22/1975	2,742	2/14/1975	1,273	2/14/1975	911	3/27/1975	704	4/5/1975	495
1976	9/11/1976	10	8/25/1976	5	9/11/1976	2	9/6/1976	2	9/17/1976	2
1977	10/1/1976	-	10/1/1976	-	10/1/1976	-	10/1/1976	-	10/1/1976	-
1978	2/9/1978	3,447	2/9/1978	2,760	2/13/1978	1,534	2/14/1978	850	3/7/1978	788
1979	2/21/1979	5,080	2/23/1979	3,581	2/24/1979	2,450	3/4/1979	1,589	3/14/1979	923
1980	1/12/1980	4,921	1/14/1980	3,899	1/17/1980	2,449	1/25/1980	1,289	2/8/1980	667
1981	1/29/1981	3,890	1/30/1981	1,783	2/2/1981	933	3/30/1981	496	4/4/1981	325
1982	3/31/1982	6,522	2/17/1982	4,434	1/6/1982	2,498	4/12/1982	1,499	4/12/1982	1,202
1983	11/30/1982	6,620	1/24/1983	4,727	1/28/1983	3,243	2/1/1983	2,093	2/15/1983	1,539
1984	12/25/1983	5,755	12/26/1983	3,764	12/28/1983	1,883	1/1/1984	941	1/7/1984	554
1985	2/8/1985	2,411	2/10/1985	1,367	2/10/1985	639	2/10/1985	345	12/20/1984	237
1986	2/19/1986	9,555	2/19/1986	7,662	2/20/1986	4,420	2/24/1986	2,195	3/16/1986	1,522
1987	3/6/1987	2,891	3/7/1987	1,389	3/11/1987	643	3/19/1987	345	4/3/1987	202
1988	1/18/1988	63	1/20/1988	34	1/23/1988	16	1/23/1988	8	1/23/1988	4
1989	3/4/1989	45	3/5/1989	35	3/9/1989	16	3/16/1989	13	4/1/1989	9
1990	4/16/1990	25	4/18/1990	25	4/21/1990	25	4/29/1990	24	3/22/1990	19
1991	3/26/1991	2,718	3/26/1991	2,013	3/30/1991	1,264	4/1/1991	820	4/11/1991	434
1992	2/15/1992	4,517	2/15/1992	2,115	2/17/1992	1,363	2/25/1992	681	3/11/1992	410
1993	1/13/1993	2,697	1/15/1993	1,797	1/18/1993	1,528	1/22/1993	1,236	2/10/1993	721
1994	2/20/1994	281	2/22/1994	162	2/25/1994	104	3/4/1994	60	3/10/1994	37
1995	1/27/1995	4,854	3/12/1995	3,641	1/29/1995	2,128	3/24/1995	1,602	2/2/1995	906
1996	2/21/1996	3,941	2/22/1996	3,054	2/25/1996	1,599	3/2/1996	792	2/23/1996	765
1997	1/2/1997	7,777	1/3/1997	4,344	1/27/1997	2,448	1/4/1997	1,598	1/28/1997	1,127
1998	2/3/1998	11,270	2/4/1998	5,253	2/8/1998	4,628	2/16/1998	2,861	2/10/1998	1,831
1999	2/9/1999	4,517	2/10/1999	2,677	2/13/1999	1,423	2/21/1999	891	2/22/1999	519

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
2000	1/25/2000	5,137	2/14/2000	3,934	2/18/2000	2,049	2/26/2000	1,309	3/11/2000	940
2001	3/5/2001	1,390	3/6/2001	770	3/7/2001	376	3/7/2001	258	3/7/2001	167
2002	1/3/2002	2,653	1/4/2002	1,679	1/4/2002	1,355	1/11/2002	657	1/27/2002	390
2003	1/19/2003	254	3/26/2003	200	3/29/2003	177	3/29/2003	105	3/31/2003	70
2004	2/26/2004	1,170	2/28/2004	834	3/3/2004	567	3/3/2004	305	11/26/2003	182
2005	3/23/2005	4,597	3/24/2005	2,436	1/13/2005	1,539	1/13/2005	1,062	1/29/2005	694
2006	4/4/2006	9,912	4/5/2006	6,096	4/6/2006	3,353	1/3/2006	2,048	4/6/2006	1,273
2007	2/27/2007	869	2/28/2007	670	3/1/2007	504	2/28/2007	411	3/7/2007	266
2008	2/3/2008	3,314	1/29/2008	1,949	1/29/2008	1,346	2/5/2008	957	2/20/2008	584

Table 18. Littlejohn Creek at Farmington, CA, annual maximum series for unregulated volume-frequency analysis

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1951	12/8/1950	5,333	12/9/1950	4,091	12/10/1950	2,828	12/17/1950	1,636	12/17/1950	1,076
1952	3/15/1952	5,019	1/27/1952	3,375	1/29/1952	2,234	1/28/1952	1,435	1/28/1952	1,024
1953	1/14/1953	725	1/15/1953	457	1/20/1953	403	1/21/1953	320	1/28/1953	215
1954	3/17/1954	723	3/19/1954	429	3/23/1954	301	3/31/1954	173	4/14/1954	100
1955	1/1/1955	3,556	1/18/1955	2,034	1/21/1955	1,286	1/24/1955	741	1/30/1955	558
1956	12/24/1955	9,011	12/25/1955	7,994	12/28/1955	4,097	1/6/1956	2,284	1/21/1956	1,697
1957	3/5/1957	2,232	3/7/1957	1,086	3/11/1957	523	3/17/1957	263	3/30/1957	136
1958	4/3/1958	7,553	4/3/1958	7,006	4/6/1958	3,985	4/4/1958	2,281	4/12/1958	1,501
1959	2/17/1959	1,652	2/18/1959	1,388	2/22/1959	1,020	2/25/1959	663	3/12/1959	368
1960	2/10/1960	1,402	2/12/1960	706	2/13/1960	496	2/21/1960	303	3/6/1960	166
1961	2/2/1961	102	2/4/1961	78	2/8/1961	61	2/15/1961	38	2/15/1961	19
1962	2/15/1962	5,097	2/15/1962	2,973	2/16/1962	2,464	2/23/1962	1,386	3/10/1962	932
1963	2/13/1963	3,205	4/16/1963	1,626	2/16/1963	1,036	2/15/1963	752	4/26/1963	541
1964	1/23/1964	1,624	1/24/1964	1,308	1/27/1964	788	1/26/1964	463	2/10/1964	254
1965	12/26/1964	8,760	12/26/1964	6,362	12/28/1964	4,182	1/6/1965	2,476	1/20/1965	1,456
1966	1/30/1966	2,110	12/31/1965	1,246	1/4/1966	656	2/13/1966	469	2/27/1966	267
1967	1/22/1967	4,324	4/20/1967	2,392	4/24/1967	1,999	4/21/1967	1,406	4/29/1967	978
1968	2/21/1968	1,241	2/22/1968	699	2/23/1968	424	3/2/1968	240	3/17/1968	162
1969	1/22/1969	5,299	1/23/1969	5,221	1/27/1969	4,543	1/29/1969	3,713	2/11/1969	2,617
1970	1/23/1970	5,075	1/23/1970	4,886	1/24/1970	4,578	1/28/1970	3,612	2/12/1970	1,968
1971	11/29/1970	2,624	12/1/1970	1,482	12/5/1970	1,149	12/12/1970	641	12/28/1970	448
1972	12/25/1971	1,267	12/27/1971	900	12/31/1971	661	1/8/1972	334	1/22/1972	173
1973	2/11/1973	6,244	1/16/1973	4,240	2/17/1973	3,445	2/20/1973	2,298	3/10/1973	1,639
1974	3/2/1974	4,749	4/4/1974	1,931	4/7/1974	1,679	4/13/1974	1,223	4/14/1974	628

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1975	3/22/1975	2,742	2/14/1975	1,278	2/14/1975	916	3/27/1975	721	4/5/1975	515
1976	9/11/1976	43	9/13/1976	27	10/5/1975	25	10/13/1975	25	10/28/1975	25
1977	8/8/1977	75	8/9/1977	63	9/4/1977	48	9/6/1977	46	9/5/1977	40
1978	2/9/1978	3,517	2/9/1978	2,829	2/13/1978	1,586	2/15/1978	883	3/7/1978	807
1979	2/21/1979	5,163	2/22/1979	3,664	2/24/1979	2,493	3/4/1979	1,609	3/14/1979	933
1980	1/12/1980	4,980	1/14/1980	3,967	1/17/1980	2,486	1/24/1980	1,307	2/8/1980	705
1981	1/29/1981	3,985	1/30/1981	1,871	2/2/1981	995	2/10/1981	533	4/3/1981	354
1982	3/31/1982	6,522	2/17/1982	4,461	1/6/1982	2,610	4/12/1982	1,532	4/12/1982	1,225
1983	11/30/1982	6,876	1/24/1983	4,813	1/28/1983	3,299	2/1/1983	2,137	2/15/1983	1,565
1984	12/25/1983	5,755	12/26/1983	3,894	12/29/1983	2,036	1/6/1984	1,083	1/8/1984	688
1985	2/8/1985	2,419	2/10/1985	1,479	2/14/1985	751	2/22/1985	441	12/23/1984	287
1986	2/17/1986	9,786	2/19/1986	7,897	2/20/1986	4,612	2/21/1986	2,343	3/16/1986	1,634
1987	3/6/1987	3,228	3/7/1987	1,841	3/11/1987	975	3/19/1987	589	4/3/1987	395
1988	1/20/1988	204	1/22/1988	183	1/25/1988	148	2/2/1988	109	8/26/1988	102
1989	3/4/1989	123	3/6/1989	91	3/10/1989	81	3/18/1989	78	4/2/1989	75
1990	3/5/1990	164	2/20/1990	109	2/24/1990	87	3/5/1990	77	3/19/1990	52
1991	3/26/1991	2,718	3/26/1991	2,013	3/30/1991	1,264	4/1/1991	820	4/11/1991	434
1992	2/15/1992	4,517	2/15/1992	2,115	2/17/1992	1,363	2/26/1992	701	3/11/1992	449
1993	1/13/1993	2,810	1/15/1993	1,964	1/18/1993	1,721	1/22/1993	1,419	2/5/1993	884
1994	2/21/1994	429	2/22/1994	414	2/26/1994	360	3/6/1994	320	10/6/1993	234
1995	1/27/1995	4,999	3/12/1995	3,683	1/29/1995	2,308	3/24/1995	1,612	2/3/1995	1,121
1996	2/21/1996	3,977	2/22/1996	3,130	2/25/1996	1,645	3/4/1996	1,001	2/23/1996	880
1997	1/2/1997	7,942	1/3/1997	4,510	1/27/1997	2,453	1/4/1997	1,788	1/28/1997	1,251
1998	2/3/1998	11,547	2/4/1998	5,455	2/8/1998	4,838	2/16/1998	3,008	2/10/1998	2,013
1999	2/9/1999	4,668	2/10/1999	2,736	2/13/1999	1,449	2/21/1999	946	3/8/1999	572

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
2000	1/25/2000	5,149	2/14/2000	3,949	2/18/2000	2,116	2/26/2000	1,366	3/11/2000	976
2001	3/5/2001	1,452	3/6/2001	833	3/10/2001	450	3/9/2001	382	3/10/2001	251
2002	1/3/2002	2,692	1/3/2002	1,752	1/4/2002	1,414	1/11/2002	737	1/26/2002	438
2003	1/1/2003	306	1/3/2003	254	3/29/2003	177	3/29/2003	117	1/21/2003	82
2004	2/26/2004	1,170	2/28/2004	834	3/3/2004	567	3/3/2004	333	3/8/2004	188
2005	3/23/2005	4,597	3/24/2005	2,436	1/13/2005	1,539	1/13/2005	1,062	1/29/2005	694
2006	4/4/2006	9,912	4/5/2006	6,096	4/6/2006	3,353	1/3/2006	2,048	4/6/2006	1,273
2007	2/27/2007	869	2/28/2007	670	3/1/2007	504	2/28/2007	411	3/7/2007	266
2008	2/3/2008	3,345	1/29/2008	1,952	1/29/2008	1,367	2/6/2008	1,004	2/20/2008	608

Peak annual maximum series

To develop the peak inflow annual maximum series for Farmington Reservoir, we reviewed the data provided by the Corps and other sources that contain annual maximum series, including:

- Littlejohn Creek stream group hydrology report (USACE 1983).
- Farmington Reservoir water control manual (USACE 2004), hereafter referred to as Farmington WCM.
- Peak flow data provided by the Corps on 6/11/2010.

We summarize in Table 19 the data we identified for use in developing flow-frequency curves for New Hogan. Column 1 lists the time period for which data were identified, and column 2 lists the source of these data.

Table 19. Data sources of peak inflow annual maximum series data identified for use in developing flow-frequency curves for Farmington Reservoir

Time period (water year) (1)	**Data source used (2)**
1903-1951[1]	Littlejohn Creek stream group hydrology report (USACE 1983)
1952-2004	Farmington WCM (USACE 2004)

Notes:

1. Intermittent historical data only. Historical information was not used to fit the unregulated inflow frequency curves consistent with current practice for peak flows at this location.

We list the peak inflow values and, where possible, their associated dates of occurrence, for Farmington Reservoir in Table 20. In the table, column 1 lists the water year; column 2 lists the date, if available; and column 3 lists the value in cfs.

We did not develop a peak flow-frequency curve for Littlejohn Creek at Farmington, CA, because a series of annual maximum peak flows at this location is not available. A peak unregulated flow-frequency curve is not required for this analysis.

Table 20. Farmington Reservoir annual maximum peak inflows

Water year (1)	**Date of peak inflow (2)**	**Peak inflow (cfs) (3)**
1952	March 1952	11,500
1953-1954	—	—
1955	—	5,700
1956	December 1955	20,000
1957	—	2,400
1958	April 1958	28,900
1959	—	2,390
1960	—	1,100
1961	—	—
1962	—	7,700
1963	—	—
1964	—	2,480
1965	December 1964	18,100
1966	—	—
1967	1/22/1967	8,110
1968	—	—
1969	1/21/1969	7,390
1970-1972	—	—
1973	2/12/1973	7,300
1974	3/2/1974	10,500
1975	3/22/1975	4,400
1976-1979	—	—
1980	1/14/1980	7,900
1981	—	—
1982	3/31/1982	14,411
1983	1/22/1983	16,500
1984	12/25/1983	9,900
1985	—	—
1986	2/19/1986	23,571
1987	3/5/1987	6,779
1988	—	—
1989	3/3/1989	71
1990	—	—
1991	3/24/1991	12,714
1992	2/15/1992	9,595
1993	—	6,823
1994	2/20/1994	807
1995	3/10/1995	12,281

Water year (1)	**Date of peak inflow (2)**	**Peak inflow (cfs) (3)**
1996	2/4/1996	10,185
1997	1/2/1997	12,929
1998	2/6/1998	24,830
1999	2/9/1999	8,302
2000	2/12/2000	10,013
2001	2/24/2001	2,465
2002	1/2/2002	6,331
2003	12/16/2002	1,550
2004	2/26/2004	1,992

Attachment 4: Fitting the unregulated frequency curves

Overview

The purpose of this attachment is to describe the steps taken to fit unregulated frequency curves to annual maximum series. We developed unregulated frequency curves following the procedures specified in *Bulletin 17B* (IACWD 1982), guidance detailed in EM 1110-2-1415 (USACE 1993), and the current standards of practice. Specially, we:

- Identified the annual maximum series.
- (Task 4.1) Calculated regional skew values for each duration of interest using relationships developed by the USGS.
- (Task 4.2) Fitted LPIII distributions to the annual maximum series following *Bulletin 17B* procedures and Corps guidance using PeakfqSA, the USGS's flow-frequency software with the expected moments algorithm (EMA) option enabled developed by Tim Cohn of the USGS (Cohn 2007).
- Reviewed and adopted the curves, checking them for consistency and comparing them to previously accepted values.

Regional skew values

Bulletin 17B recommends the use of a regional skew value in fitting LPIII distributions to maintain consistency of frequency curves. *Bulletin 17B* also states that such a value can be developed using regression techniques. For the CVHS, the USGS, in cooperation with the Corps, has developed regression equations for regional skew values (USGS 2010). In general, there are 2 equation forms, 1 for peak flows, and 1 for volumes. The coefficients for the volumes change with duration.

The regional skew associated with peak flows is calculated as:

$$\gamma = -0.62 + 1.30\left(1 - e^{\left(-\left(\frac{Elev}{6500}\right)^{2}\right)}\right) \tag{3}$$

where γ is the regional skew value *Elev* is the average basin elevation in ft (NAVD 88). The associated average variance of prediction (AVP) is 0.14. AVP is analogous to mean square error (MSE) for the purpose of weighting regional and station skew values.

The regional skew associated with volumes is calculated as

$$\gamma = \beta_0 + \beta_1\left(1 - e^{\left(-\left(\frac{Elev}{3600}\right)^{12}\right)}\right) \tag{4}$$

where γ is the regional skew value, *Elev* is the average basin elevation in ft (NAVD 88), and β_0 and β_1 are coefficients based on the duration of interest as shown in Table 21. The associated AVP also varies with duration and is also shown in Table 21.

For this analysis, we used these equations to develop regional skew values for Littlejohn Creek as shown in Table 22. We used GIS tools to compute average basin elevations for use in the regional skew computations.

Table 21. Duration skew equation parameters

Parameter (1)	1-day regional skew (2)	3-day regional skew (3)	7-day regional skew (4)	15-day regional skew (5)	30-day regional skew (6)
β_0	-0.7340	-0.6901	-0.5872	-0.6445	-0.6322
β_1	0.6778	0.6764	0.5822	0.5375	0.4277
AVP	0.0485	0.0576	0.0490	0.0521	0.0615

Table 22. Regional skew values

Location (1)	Elevation (ft) (2)	Peak flow regional skew (3)	1-day regional skew (4)	3-day regional skew (5)	7-day regional skew (6)	15-day regional skew (7)	30-day regional skew (8)
Farmington Reservoir	621.82	-0.608	-0.734	-0.690	-0.587	-0.644	-0.632
Farmington, CA	605.62	—	-0.734	-0.690	-0.587	-0.644	-0.632

Fitting the curves

As a first step, the curves were fitted using a straightforward *Bulletin 17B* procedure in which all data points were included in the analysis and low outliers were identified by the *Bulletin 17B* outlier test and the station statistics appropriately adjusted. This includes weighting the station skew and regional skew values by the inverse of their associated errors. This weighting procedure is included in *Bulletin 17B* and the weighted skew is automatically calculated by PeakfqSA, which we used here.

We found that this initial fitting of the frequency curves: (1) was sensitive to low flow values, and (2) the 1-day and 3-day flow quantiles for p= 0.01 and p=0.005 annual exceedence probabilities were uncharacteristically large on a flow-per-square mile basis.

We then refitted the frequency curves manually setting the low outlier thresholds for each duration. Specifically, we set these thresholds consistent with those used in the Comp Study. These low outlier thresholds are shown in Table 23 and Table 24.

We then reviewed the curves for appropriateness and consistency. We found the frequency curves on Littlejohn Creek were consistent between durations at each location for the frequencies of interest. However, at Farmington Reservoir the curves associated with the 3-day and 7-day volumes “crossed” for annual exceedence probabilities less than approximately p=0.95. We therefore adjusted the 1-day and 3-day standard deviations consistent with guidance specified in EM 1110-2-1415 (USACE 1993). Specifically, we fit a line to the pairs of mean of the logs and standard deviation of the logs by duration using least squares regression through the data point associated with the peak flow-frequency curve. This relation is shown in Figure 21. We then set the standard deviation of the 1-day and 3-day volumes equal to that specified by this regression. We then reviewed these curves and found that they do not "cross," as would be expected.

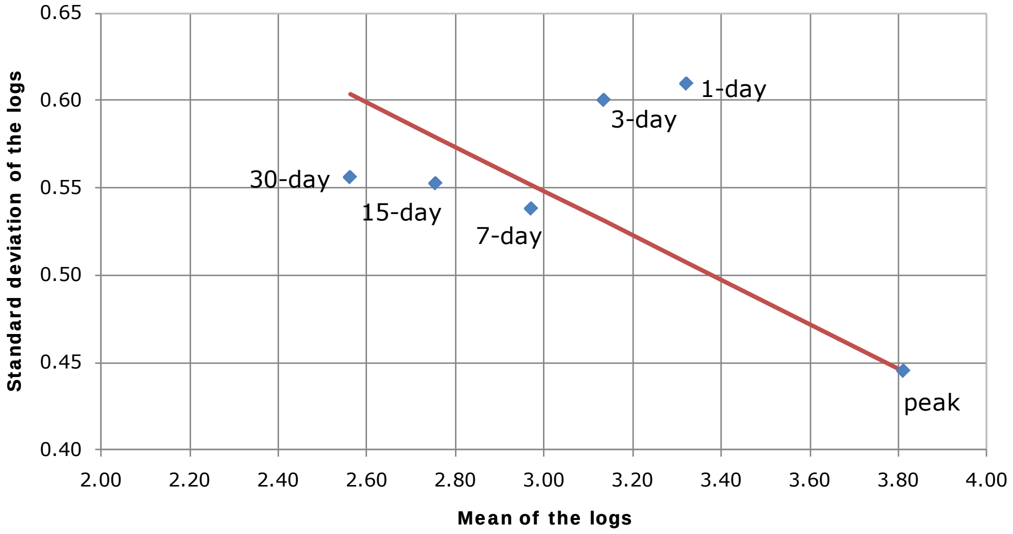


Figure 21. Relationship used to adjust standard deviations at Farmington Reservoir

In addition, we found in our review of the curves at Farmington, CA, that the curve associated with the 30-day volume is inconsistent with the 30-day curve associated with the upstream analysis location at Farmington Reservoir. We therefore set the standard deviation of the 30-day curve at Farmington, CA, equal that of the 30-day curve at Farmington Reservoir. This is consistent with Corps guidance in EM 1110-2-14-15 (USACE 1993). We then reviewed these curves and found that they do not "cross," and flow quantiles for a given duration at the downstream location are greater than those of the upstream location, as would be expected.

As a comparison, we considered the volume-frequency curves developed for Farmington Reservoir in the Comp Study (USACE 2002). The annual maximum series in the Comp Study ended in 1998.

Results

The final parameters and statistics used to fit LPIII distributions to develop the unregulated frequency curves at Farmington Reservoir (shown in Figure 9) are shown in Table 23.

The final parameters and statistics used to fit LPIII distributions to develop the unregulated frequency curves at Farmington, CA, (shown in Figure 10) are shown in Table 24.

Table 23. Unregulated frequency curves parameters and statistics: Farmington Reservoir

Statistic (1)	**Peak flows (2)**	**1-day volumes (3)**	**3-day volumes (4)**	**7-day volumes (5)**	**15-day volumes (6)**	**30-day volumes (7)**
Station mean[1]	3.810	3.301	3.114	2.948	2.733	2.540
Station standard deviation[1]	0.449	0.668	0.661	0.601	0.612	0.615
Station skew[1]	-0.978	-1.410	-1.410	-1.410	-1.410	-1.410
Station skew associated MSE[2]	0.370	0.276	0.275	0.274	0.274	0.273
Regional skew[3]	-0.608	-0.734	-0.690	-0.587	-0.644	-0.632
Regional skew associated AVP[4]	0.140	0.049	0.058	0.049	0.052	0.062
Adopted mean[5]	3.811	3.321	3.135	2.970	2.754	2.561
Standard deviation[5]	0.445	0.610	0.601	0.538	0.553	0.556
Adopted standard deviation	0.445	0.507	0.531	0.538	0.553	0.556
Weighted skew[5,6]	-0.692	-0.858	-0.812	-0.675	-0.733	-0.721
Number of systematic events	34	58	58	58	58	58
Number of high outliers	0	0	0	0	0	0
Number of EMA iterations	2	1	1	1	1	1
Specified low outlier threshold (cfs)	—	282	201	178	105	71
Number of low outliers	0	8	8	8	8	8
Number of zero events	0	0	0	0	0	0
Number of missing events	19	0	0	0	0	0
Number of EMA censored observations	1	8	8	8	8	8
Corresponding censored events[7]	1). 1977	1.) 1977 2.) 1976 3.) 1990 4.) 1989 5.) 1988 6.) 1961 7.) 2003 8.) 1994	1.) 1977 2.) 1976 3.) 1990 4.) 1988 5.) 1989 6.) 1961 7.) 1994 8.) 2003	1.) 1977 2.) 1976 3.) 1989 4.) 1988 5.) 1990 6.) 1961 7.) 1994 8.) 2003	1.) 1977 2.) 1976 3.) 1988 4.) 1989 5.) 1990 6.) 1961 7.) 1994 8.) 2003	1.) 1977 2.) 1976 3.) 1988 4.) 1989 5.) 1990 6.) 1961 7.) 1994 8.) 2003
Record length	53	58	58	58	58	58

Notes:

1. Statistic calculated using the series of logarithmic transforms and EMA without regional skew; rounded to nearest thousandth.
2. Mean square error; rounded to nearest thousandth.
3. Regional skew values calculated using relationships developed by the USGS; rounded to nearest thousandth.
4. Average variance of prediction, analogous to MSE; rounded to nearest thousandth.
5. Statistic calculated using the series of logarithmic transforms and EMA with regional skew; rounded to nearest thousandth.
6. Skew value calculated by weighting the station and regional skew values inversely proportional to their associated errors: (MSE and AVP) and EMA; rounded to nearest thousandth.
7. Events are listed by water year in order of increasing flow or volume.

Table 24. Unregulated frequency curves parameters and statistics: Farmington, CA

Statistic (1)	**1-day volumes (2)**	**3-day volumes (3)**	**7-day volumes (4)**	**15-day volumes (5)**	**30-day volumes (6)**
Station mean[1]	3.339	3.169	2.992	2.797	2.628
Station standard deviation[1]	0.621	0.593	0.579	0.573	0.539
Station skew[1]	-1.410	-1.410	-1.410	-1.410	-1.268
Station skew associated MSE[2]	0.278	0.276	0.276	0.276	0.251
Regional skew[3]	-0.734	-0.690	-0.587	-0.644	-0.632
Regional skew associated AVP[4]	0.049	0.058	0.049	0.052	0.062
Adopted mean[5]	3.356	3.186	3.011	2.815	2.639
Standard deviation[5]	0.573	0.545	0.525	0.523	0.507
Adopted standard deviation	0.573	0.545	0.525	0.523	0.556
Weighted skew[5,6]	-0.849	-0.786	-0.670	-0.722	-0.695
Number of systematic events	58	58	58	58	58
Number of high outliers	0	0	0	0	0
Number of EMA iterations	1	1	1	1	1
Specified low outlier threshold (cfs)	307	254	178	117	82
Number of low outliers	7	7	7	7	7
Number of zero events	0	0	0	0	0
Number of missing events	0	0	0	0	0
Number of EMA censored observations	7	7	7	7	6
Corresponding censored events[7]	1.) 1976 2.) 1977 3.) 1961 4.) 1989 5.) 1990 6.) 1988 7.) 2003	1.) 1976 2.) 1977 3.) 1961 4.) 1989 5.) 1990 6.) 1988 7.) 2003	1.) 1976 2.) 1977 3.) 1961 4.) 1989 5.) 1990 6.) 1988 7.) 2003	1.) 1976 2.) 1961 3.) 1977 4.) 1990 5.) 1989 6.) 1988 7.) 2003	1.) 1961 2.) 1989 3.) 1990 4.) 1977 5.) 1989 6.) 2003
Record length	58	58	58	58	58

Notes:

1. Statistic calculated using the series of logarithmic transforms and EMA without regional skew; rounded to nearest thousandth.
2. Mean square error; rounded to nearest thousandth.
3. Regional skew values calculated using relationships developed by the USGS; rounded to nearest thousandth.
4. Average variance of prediction, analogous to MSE; rounded to nearest thousandth.
5. Statistic calculated using the series of logarithmic transforms and EMA with regional skew; rounded to nearest thousandth.
6. Skew value calculated by weighting the station and regional skew values inversely proportional to their associated errors: (MSE and AVP) and EMA; rounded to nearest thousandth.
7. Events are listed by water year in order of increasing volume.

Attachment 5: Unregulated-regulated flow transforms and critical duration assessment

Fit unregulated-regulated flow transforms

We developed the unregulated-regulated flow transforms for the 2 analysis locations by fitting transform curves through data pairs from the event maxima datasets. Specifically, we fitted transforms to pairs of unregulated volumes (as average flows) and regulated peak flows. For this analysis, we used unregulated volumes associated with the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations. We fitted these curves to the data pairs of historical and scaled events using the robust locally weighted scatterplot smoothing (LOWESS) regression technique. (The LOWESS procedure is detailed in the *Technical procedure document*.)

Here, we used the LOWESS algorithm developed by William Cleveland (Cleveland 1985). We complied an executable of the algorithm, implemented in Fortran. This executable was tested using example data included in the Fortran file.

We used an iterative process to fit these transforms. Specifically we:

- Fitted a candidate transform using the LOWESS regression technique.
- Calculated the mean squared error (MSE) associated with the candidate transform.
- Modified the LOWESS parameters using guidance provided in the literature (Bradley and Potter 2004, Cleveland 1979).
- Fitted another candidate transform and calculated the associated MSE.
- Compared this new transform to the old transform(s) visually and based on MSE.
- Repeated the previous steps until the parameters resulting in the best fit, as determined visually and based on MSE, were identified.

Determine critical duration

For a regulated system, the critical duration is the unregulated flow duration-frequency curve that best characterizes the peak regulated flow-frequency curve at a downstream point. To determine critical duration for each location, we:

- Fitted flow transforms to the event maxima datasets as detailed in the previous subsection.
- Applied these flow transforms to develop hypothetical regulated flow-frequency curves.
- Identified the duration of the unregulated annual maximum series that estimates the largest flow for each probability of interest, as shown in column 1 of Table 25. Here, we considered2 criteria: (1) the "goodness of fit" of each transform, and (2) which duration estimates the greater peak regulated flows

Table 25. Synthesis of information used to determine critical duration

Annual exceedence probability (1)	Unregulated flow duration (in days) that estimates the largest flow quantile at Farmington Reservoir (2)	Farmington, CA[1] (3)
0.500	15	10
0.200	2.5	3.5
0.100	2.5	3.5
0.050	15	1
0.020	15	10
0.010	15	10
0.005	10	10
0.002	10	10

Notes:

1. For Farmington, CA, we list the duration equal or less than 15 days that estimates the largest flow.

After considering all the durations noted above, for Farmington Reservoir we focused on durations of 15 days or less because: (1) the typical unregulated inflow event duration is less than 15 days, and (2) the flow transforms for durations of 15 days or less better fit the event maxima data pairs based on MSE and visual inspection. In addition, the scaled historical event unregulated volumes associated with the longer durations tend to include volumes of additional flood waves after the peak reservoir release. These later flood waves do not contribute to the inflow volumes that drive the reservoir releases, unlike multiple flood waves prior to the peak reservoir releases that are considered. Here, we defined a flood event as the time from when the pool elevation rises from and returns to the top of conversation pool (bottom of flood control pool). For Farmington, CA, we looked at durations equal or less than the critical duration at Farmington Reservoir because the addition of unregulated local flows will not cause the critical duration to increase.

In selection of the critical duration, we gave more weight to the durations that estimated the largest flow quantiles for the p=0.01, p=0.005, and p=0.002 annual exceedence events. We used these probabilities because Farmington Reservoir has large flood storage volume, and regulated peak flows associated with more common events are driven by local flow peaks, not reservoir inflow volumes for a given duration.

From this analysis we determined that the critical duration at Farmington Reservoir and at Farmington, CA, is 10 days. Thus, the appropriate unregulated-regulated flow transforms used in this analysis were associated with this duration. The critical duration associated with the downstream operation point is shorter than that of the reservoir because of the effects of local flow.

As a "reality check" on our critical duration values, we simulated events, with the HEC-ResSim model, that corresponded to specific volumes associated with a given duration and annual exceedence probability. This is an alternative option for assessing critical duration as detailed in Attachment F of the *Technical Procedures document* as "Method 2: Limited sample, specific volume-duration event scaling." For this check, we scaled reservoir inflows for

4 event patterns (1969, 1986, 1998, and 2006) to the 1-, 3-, 7-, and 10-day unregulated flows for the p=0.01, and p=0.005 annual exceedence probabilities. We found: (1) the resulting regulated peaks sensitive to hydrograph shape, and (2) the scaling to the 1-, 3-, and 10-day durations estimated largest regulated peak flows. These results are consistent with the adopted critical duration values for the 2 analysis locations.

Review and adopt transforms

After determining the critical duration associated with each analysis location, we reviewed the unregulated-regulated flow transforms initially fitted with the LOWESS procedure to: (1) check for appropriateness, and (2) identify the need for adjustments, if any. As part of this review we:

- Compared event hydrographs of the simulated events that correspond to the transitional areas of the transform (i.e., where the objective peak flows are being constrained, or where peak releases become larger than the objective).
- Fitted additional transforms omitting scaled historical events with scale factors of 2 or less.
- Identified and compared the unregulated volumes that define the “break points” where large floods-of-record and their scaled versions were not controlled by the reservoir because of (1) lack of storage capacity, or (2) local flows larger than the channel capacity.
- Split the unregulated-regulated flow transform initially fitted with LOWESS into 2 ranges using this break point.
- Calculated the MSE for these 2 ranges for each initially fitted LOWESS curve.
- Identified which LOWESS curves have the least MSE for each range.

For both analysis locations, we found that the LOWESS fitted curves with a smoothing coefficient of 0.2 had lowest MSE for ranges of unregulated 10-day volumes both larger and smaller than that associated with the “break point.”

We adjusted the unregulated-regulated flow transform at Farmington Reservoir based on our review of selected historical events and sensitivity analysis of the LOWESS fitting of the transform. Specifically, we refined the transform using linearly interpolation for regulated peak flows between 2,000 cfs and approximately 3,100 cfs.

As a final check, we re-applied the transform to compute the associated regulated flow quantiles. We compared these quantiles to those associated with the original fit, and those associated with the candidate transforms for the other unregulated volumes. For both locations, we computed: (1) small decreases for quantiles with annual exceedence probability equal or greater p=0.05, and (2) no change in quantiles with annual exceedence probability equal or less p=0.01.

Based on this review, we adopted flow transforms for Farmington Reservoir and Farmington, CA, shown in Figure 22 and Figure 23. The tabulated curves are in an MS Excel file on DVD with the original report.

In Figure 22 and Figure 23 we show the unregulated-regulated flow transforms in black dashes, the floods-of-record event maxima in red squares, the historical scaled event maxima in green triangles, and the initial LOWESS fitted flow transforms in blue for comparison. We also show in grey in Figure 22 and Figure 23 the corresponding unregulated volume-duration quantiles for annual exceedence probabilities of interest.

We show in Table 26 and Table 27 the parameters we used to fit these transforms and the resulting mean square errors. Highlighted in grey in Table 26 and Table 27 are the LOWESS fitted curves with smoothing coefficients listed in column 1 used in fitting the final unregulated-regulated flow transforms over the ranges specified in columns 4 and 5.

Table 26. LOWESS parameters and resulting errors for fitting of unregulated-regulated flow transforms: Farmington Reservoir

Smoothing coefficient[1] (1)	Number of iterations[2] (2)	Delta[3] (3)	Minimum threshold (1,000 cfs) (4)	Maximum threshold (1,000 cfs) (5)	Total number of data pairs (6)	MSE[4] (7)
0.2	2	0	0.5	10	186	964,227
			0.5	5	120	189,155
			5	10	66	2,373,450
Adopted transform			**0.5**	**10**	**—**	**973,765**

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 4 and 5.

Table 27. LOWESS parameters and resulting errors for initial fitting of unregulated-regulated flow transforms: Farmington, CA

Smoothing coefficient[1] (1)	Number of iterations[2] (2)	Delta[3] (3)	Minimum threshold (1,000 cfs) (4)	Maximum threshold (1,000 cfs) (5)	Total number of data pairs (6)	MSE[4] (7)
0.2	2	0	0.5	10.5	188	1,366,865
			0.5	5	117	335,543
			5	10.5	71	3,066,368
Adopted transform			**0.5**	**10.5**	**—**	1,385,920

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 4 and 5.

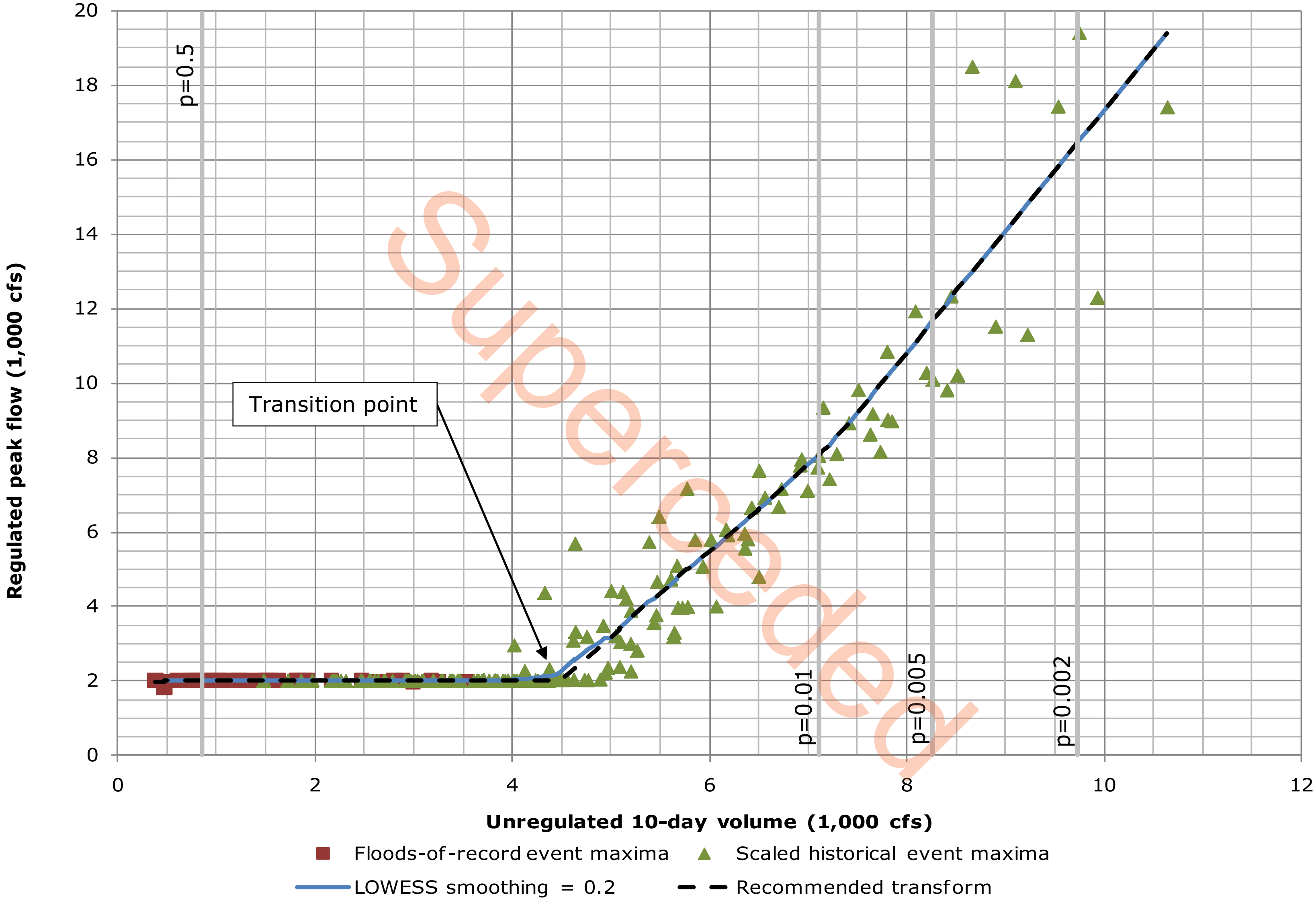


Figure 22. Unregulated-regulated flow transform and LOWESS fitted curves: Farmington Reservoir

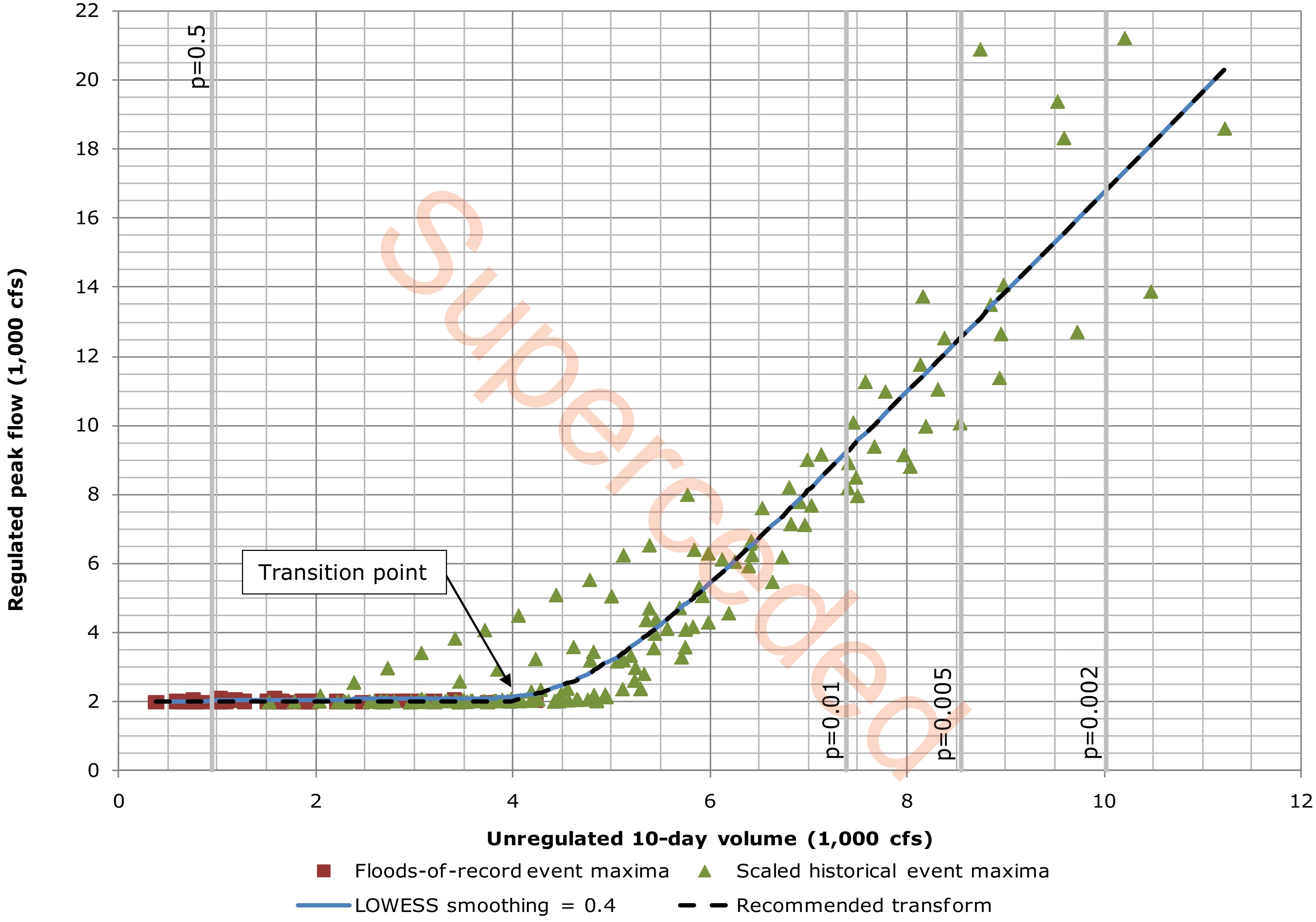


Figure 23. Unregulated-regulated flow transform and LOWESS fitted curve: Farmington, CA

Attachment 6: Family of regulated characteristic curves

Fit the characteristic curves

We used the families of regulated characteristic curves to relate a given regulated peak flow to likely associated regulated volumes at each analysis location. We developed the families of regulated characteristic curves for Farmington Reservoir and at Farmington, CA, by fitting transform curves through the pairs of event regulated volumes, as average flows, and regulated peak flows. The fitting is similar to how we developed the unregulated-regulated transforms detailed in Attachment 5. The datasets we used include both historical and scaled events to define the extreme ends of the flow transform curve.

We initially fitted these curves to the data pairs of historical and scaled events using the LOWESS regression technique and parameters shown in Table 28 and Table 29 for Farmington Reservoir and at Farmington, CA. In this initial fitting we used the entire event maxima dataset for the given analysis location. Because the flows of interest correspond to events equal or larger than the p=0.5 event, but less than or equal to the p=0.002 event, we truncated the datasets of event pairs to the minimum and maximum regulated flow thresholds specified in columns 5 and 6 of Table 28 and Table 29 for selection of the appropriate LOWESS smoothing coefficient to use in developing the characteristic curves. Highlighted in grey in Table 28 and Table 29 are the LOWESS fitted curves with smoothing coefficients listed in column 2 used in fitting the final characteristic curves for the duration specified in column 1 over the range with minimum and maximum flow thresholds specified in columns 5 and 6.

Review and adopt the characteristic curves

We reviewed and adjusted the curves initially fitted with the LOWESS procedure using the same process detailed for fitting the unregulated-regulated flow transforms. Here, the only difference is that the “break point” is defined by the downstream objective flow (2,000 cfs). Thus the mean square errors in the LOWESS fitted curves were compared over these 2 ranges for each characteristic curve.

From this review we found:

- The families of regulated characteristic curves were consistent between durations at both locations. That is, they do not cross.
- The fit of the curves at Farmington, CA, was sensitive to large diversions from Duck Creek such as those in the 1995 event and its corresponding scaled events.
- The characteristic volume at Farmington, CA, for a given annual exceedence and duration may be less than the characteristic volume associated with Farmington Reservoir for the same annual exceedence probability because this effect of diversions. However, the regulated peak flow at Farmington, CA, is always equal or larger than the peak at Farmington Reservoir for the same exceedence probability.

Based on this review, we adopted the adjusted families of curves.

We show in Figure 24 through Figure 28 the regulated characteristic curves corresponding to Farmington Reservoir. In addition, we include tabulations of this family of regulated characteristic curves in an MS Excel file on the DVD included with the original report.

We show in Figure 29 though Figure 33 regulated characteristic curves corresponding to Farmington, CA. In addition, we include tabulations of this family of regulated characteristic curves in an MS Excel file on the DVD included with the original report.

In Figure 24 through Figure 33 we show the characteristic curves in black, the floods-of-record event maxima in red squares, the historical scaled event maxima in green triangles, and the initial LOWESS fitted flow curves in blue for comparison.

Table 28. LOWESS parameters for fitting the family of regulated characteristic curves and resulting errors: Farmington Reservoir

Duration (days) (1)	**Smoothing coefficient[1] (2)**	**Number of iterations[2] (3)**	**Delta[3] (4)**	**Minimum threshold (1,000 cfs) (5)**	**Maximum threshold (1,000 cfs) (6)**	**Total number of data pairs (7)**	**LOWESS curve MSE[4] (8)**	**Characteristic curve MSE (9)**
1	0.2	2	0	2	16.5	182	7,606	7,687
3							99,693	100,058
7							270,829	279,316
15							276,837	339,035
30							183,572	290,625

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 5 and 6.

Table 29. LOWESS parameters for fitting the family of regulated characteristic curve and resulting errors: Farmington, CA

Duration (days) (1)	**Smoothing coefficient[1] (2)**	**Number of iterations[2] (3)**	**Delta[3] (4)**	**Minimum threshold (1,000 cfs) (5)**	**Maximum threshold (1,000 cfs) (6)**	**Total number of data pairs (7)**	**LOWESS curve MSE[4] (8)**	**Characteristic curve MSE (9)**
1	0.2	2	0	2	17	185	83,489	83,473
3							174,784	174,806
7							334,875	334,900
15							303,171	309,865
30							176,684	185,114

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 5 and 6.

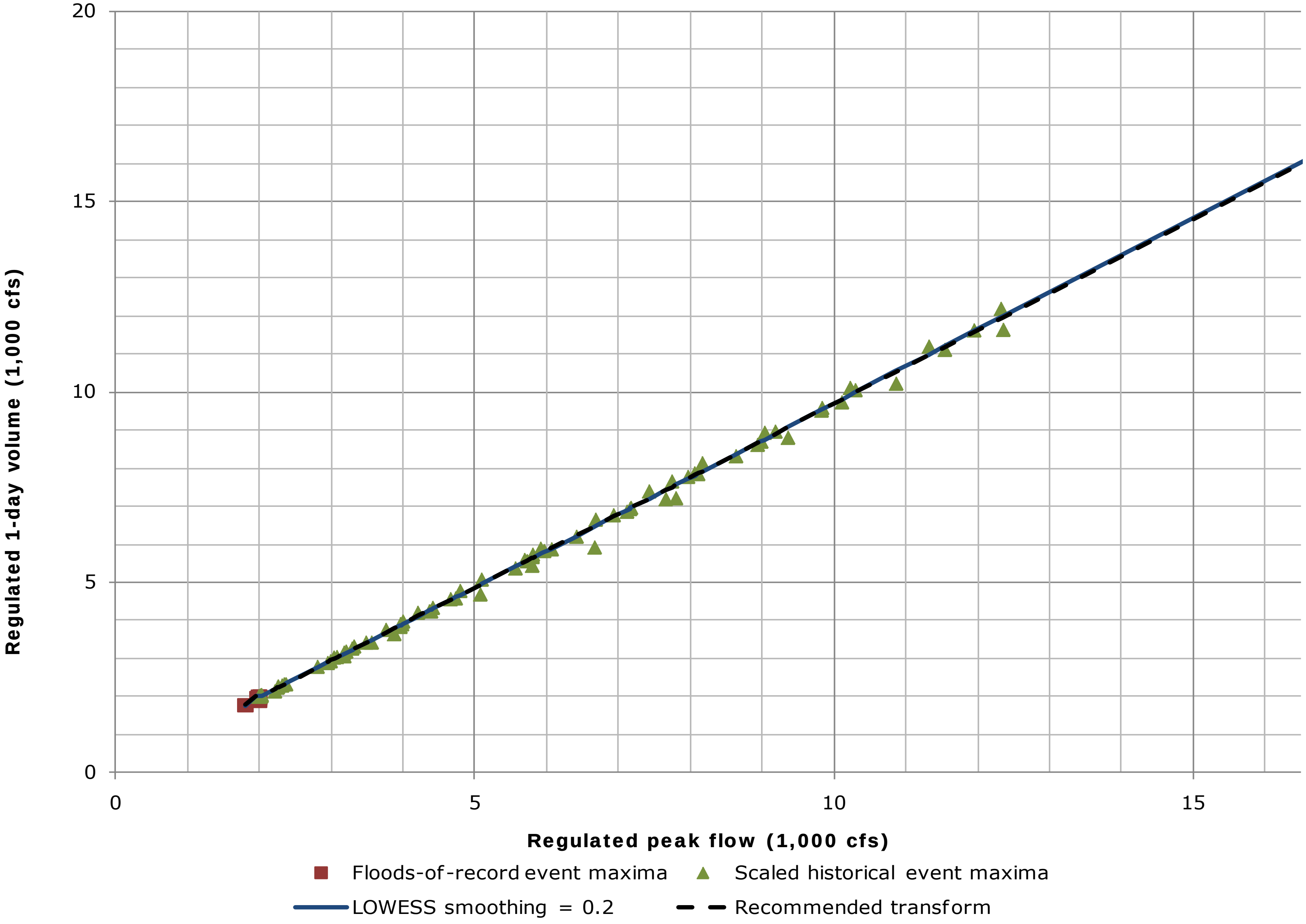


Figure 24. Farmington Reservoir regulated characteristic curve: 1-day duration

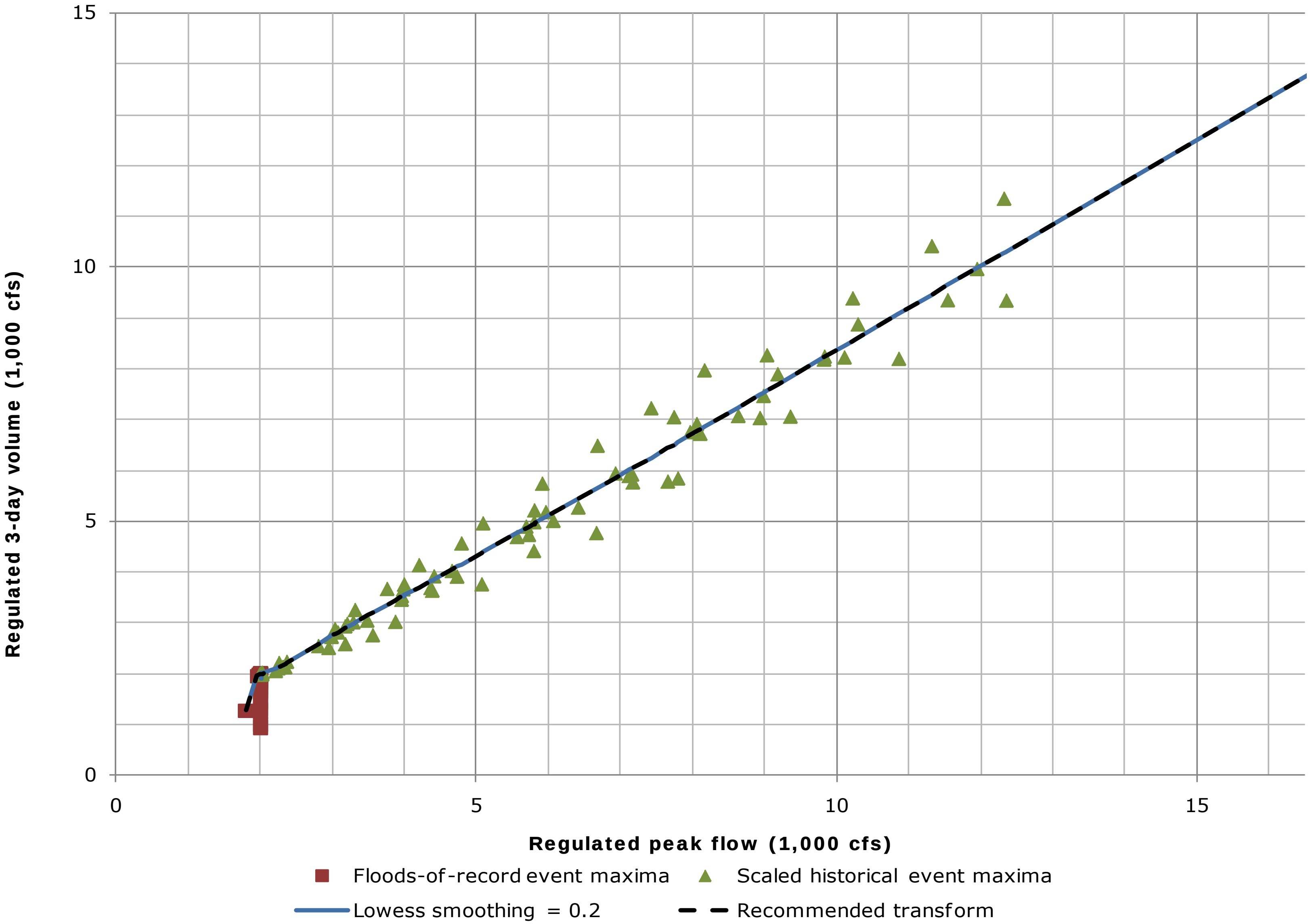


Figure 25. Farmington Reservoir regulated characteristic curve: 3-day duration

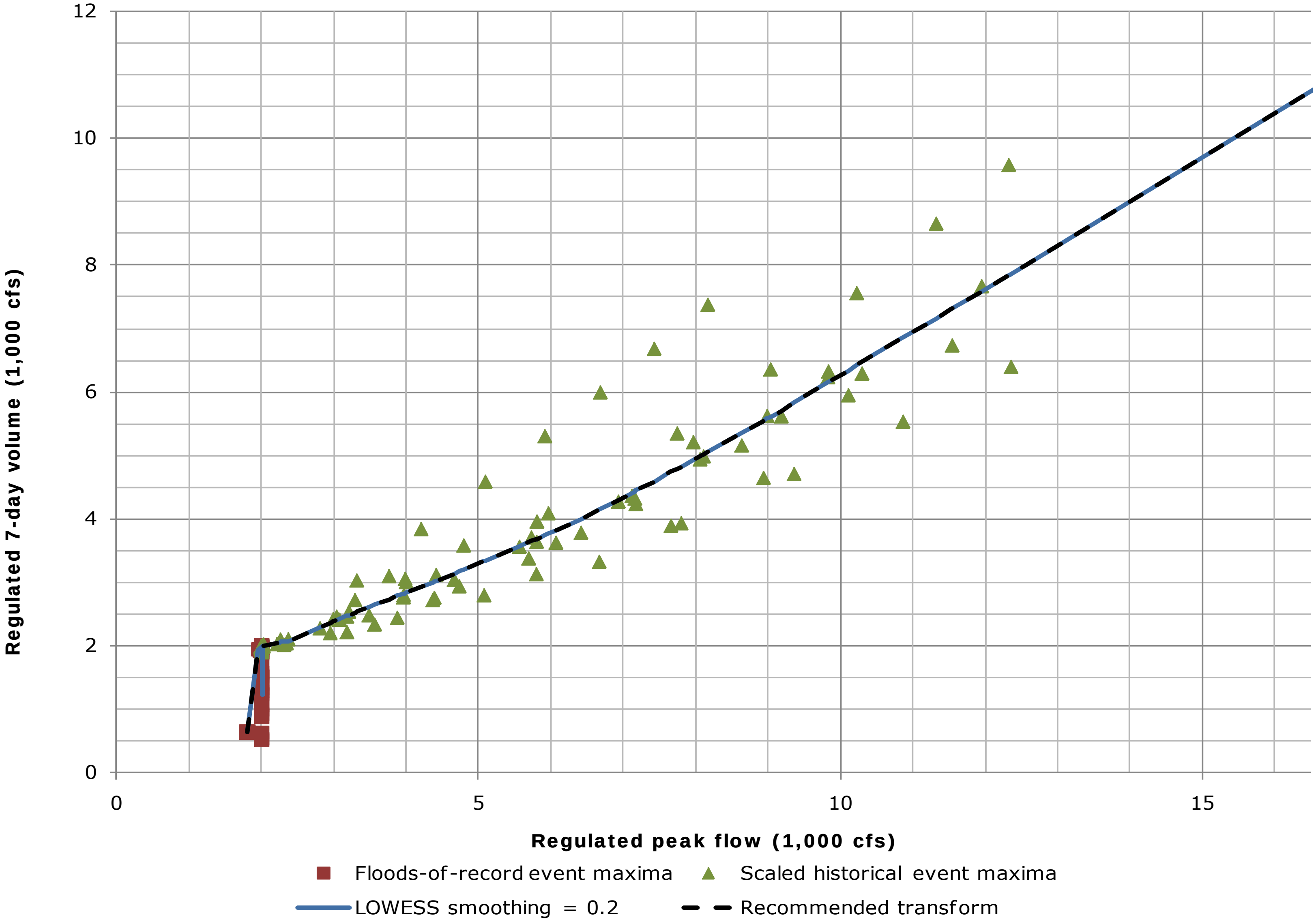


Figure 26. Farmington Reservoir regulated characteristic curve: 7-day duration

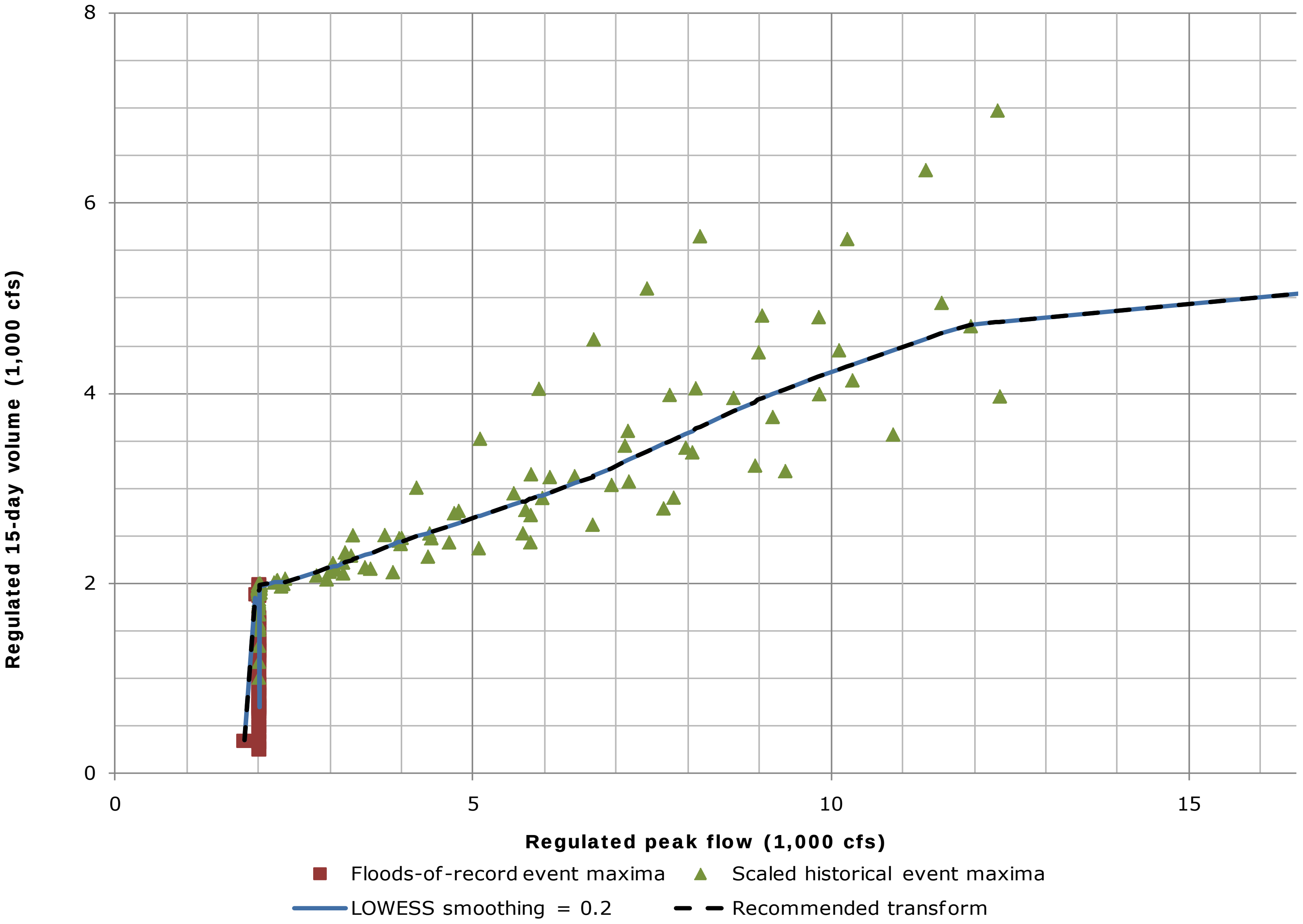


Figure 27. Farmington Reservoir regulated characteristic curve: 15-day duration

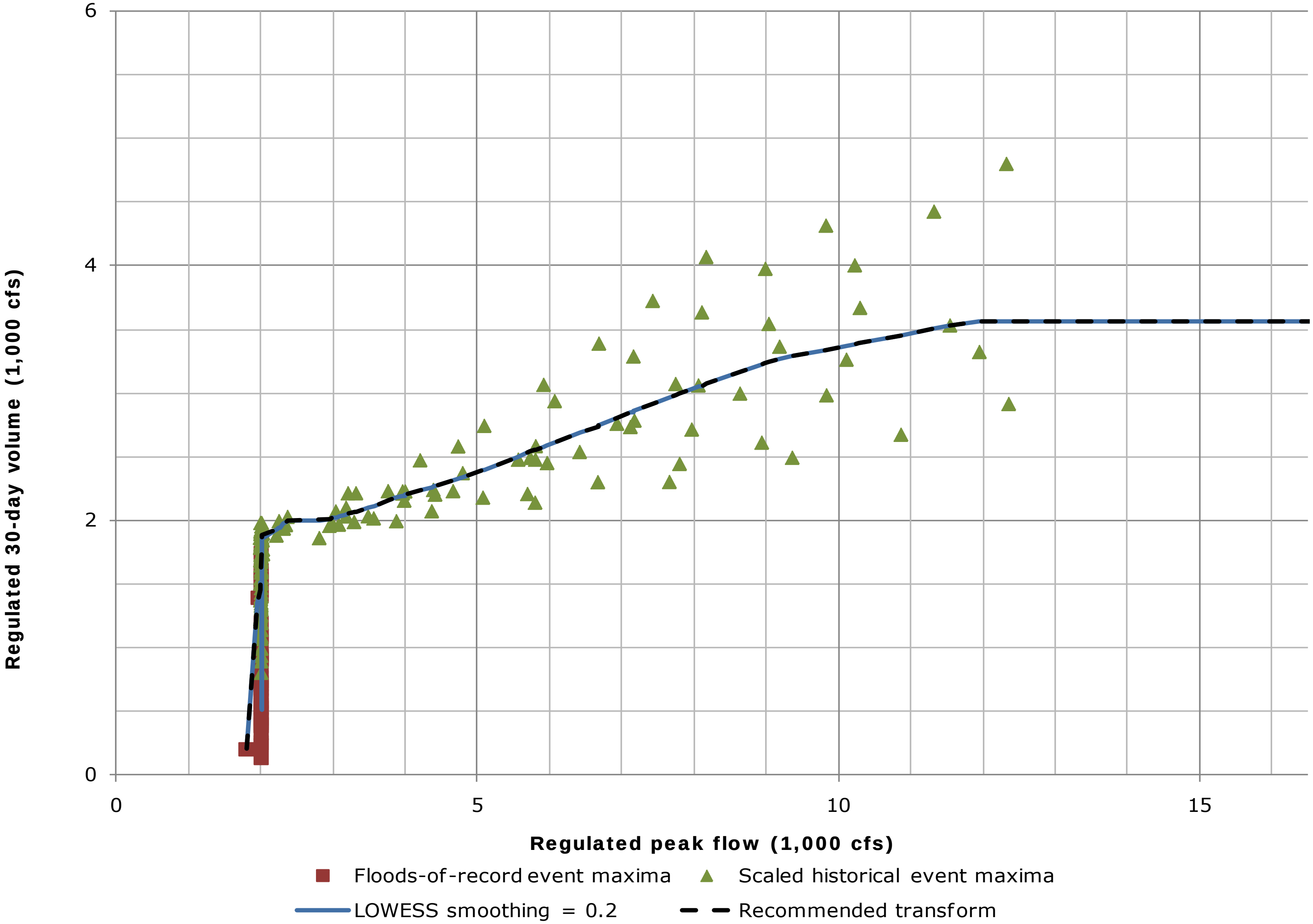


Figure 28. Farmington Reservoir regulated characteristic curve: 30-day duration

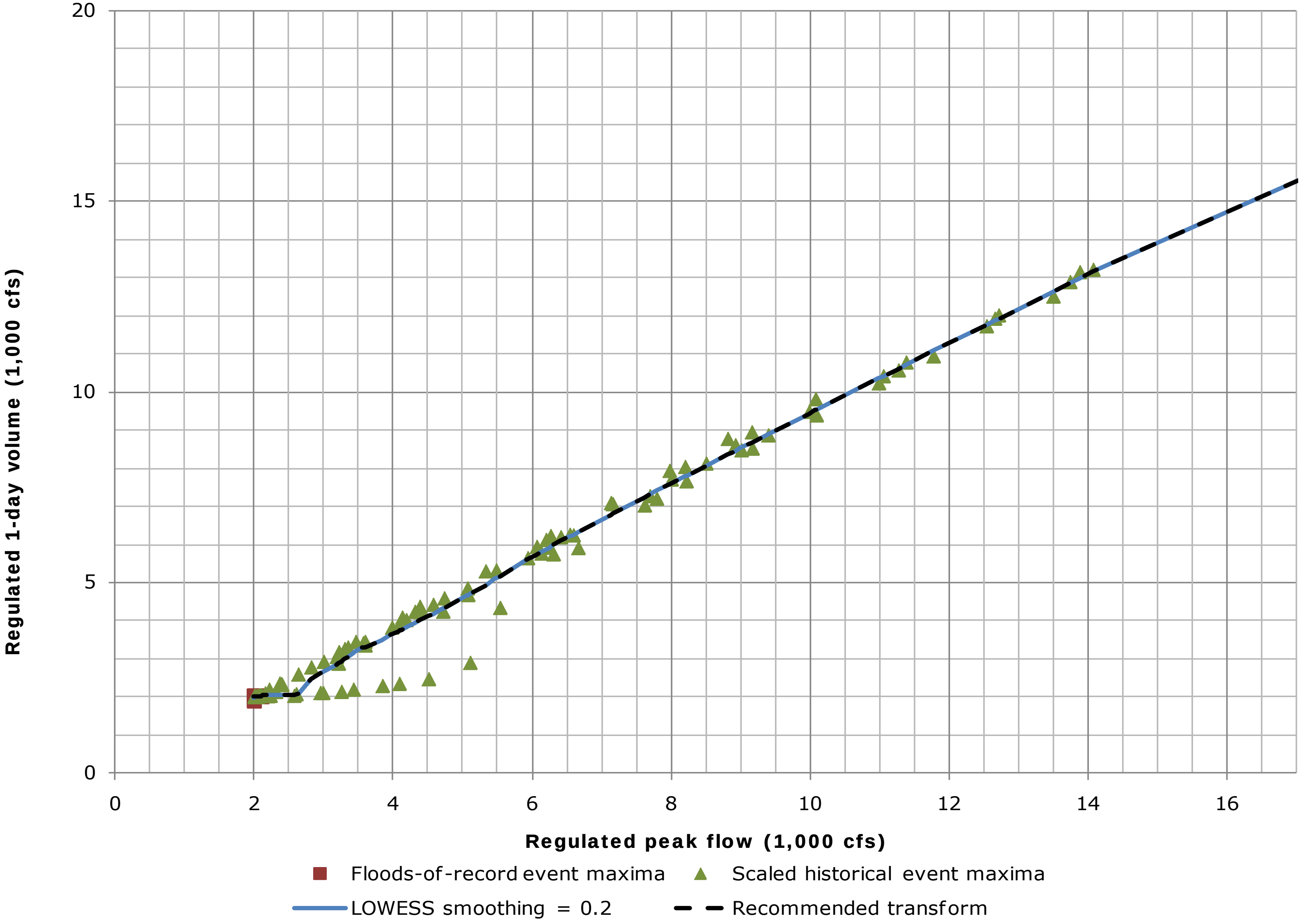


Figure 29. Littlejohn Creek at Farmington, CA, regulated characteristic curve: 1-day duration

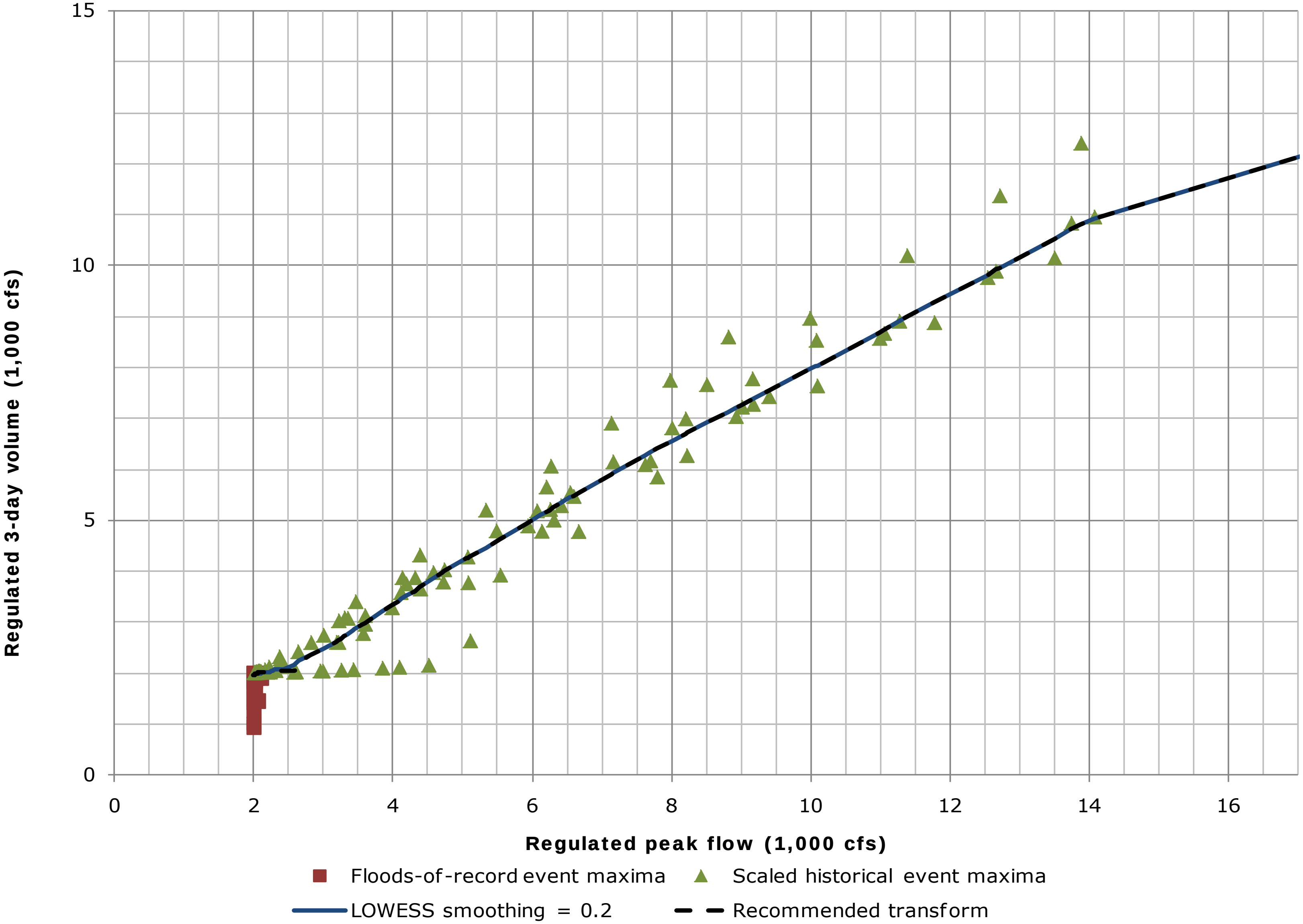


Figure 30. Littlejohn Creek at Farmington, CA, regulated characteristic curve: 3-day duration

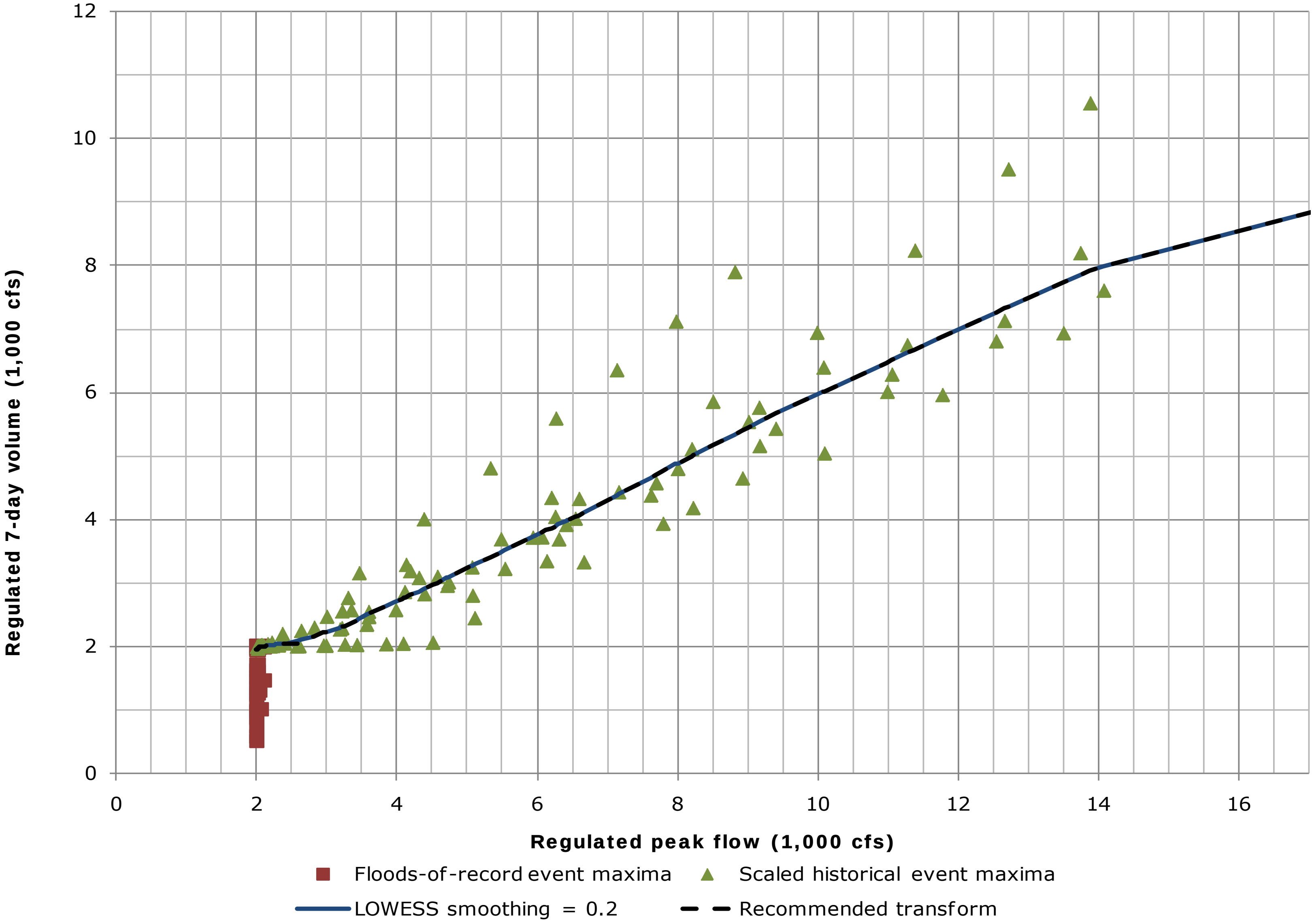


Figure 31. Littlejohn Creek at Farmington, CA, regulated characteristic curve: 7-day duration

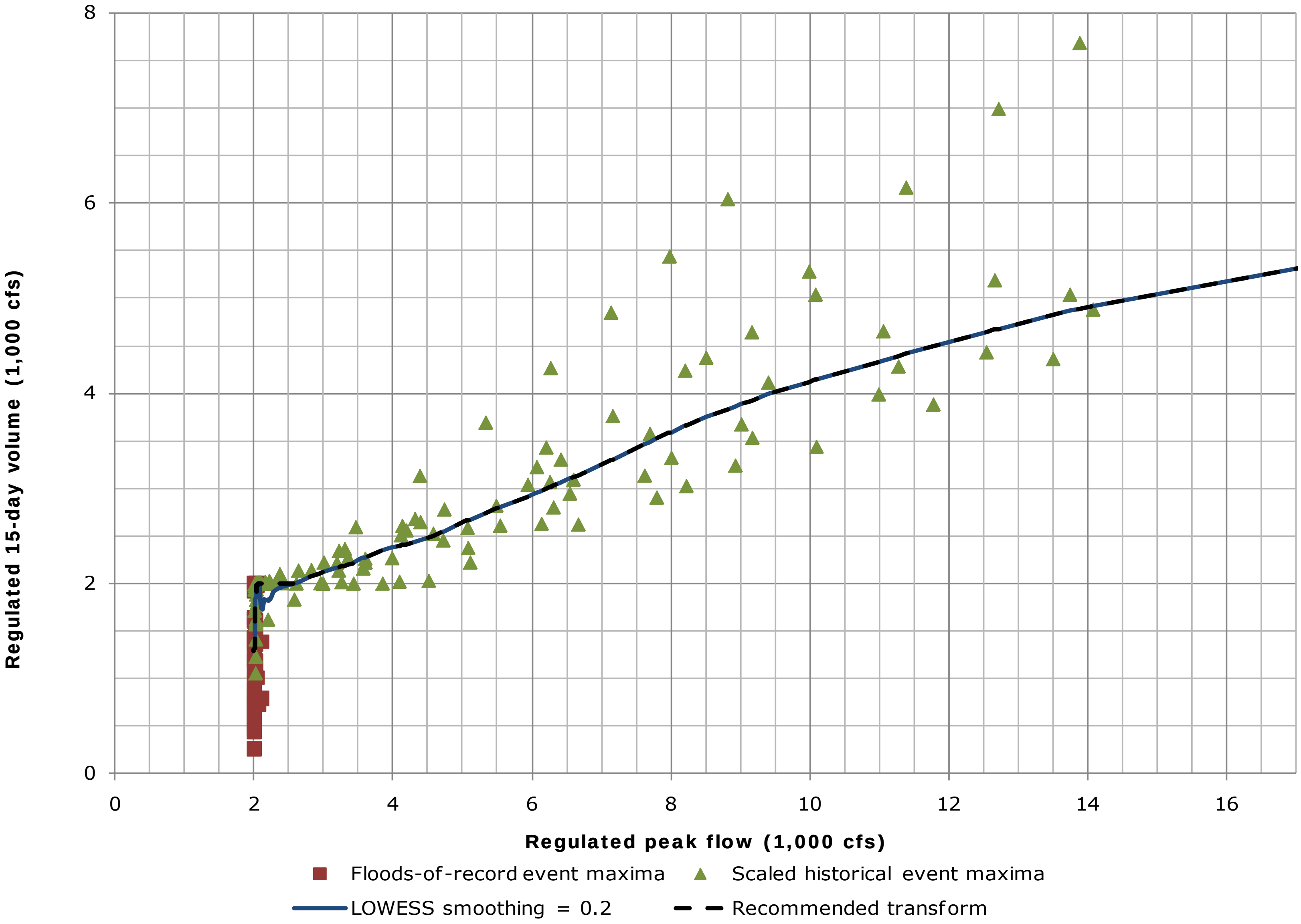


Figure 32. Littlejohn Creek at Farmington, CA, regulated characteristic curve: 15-day duration

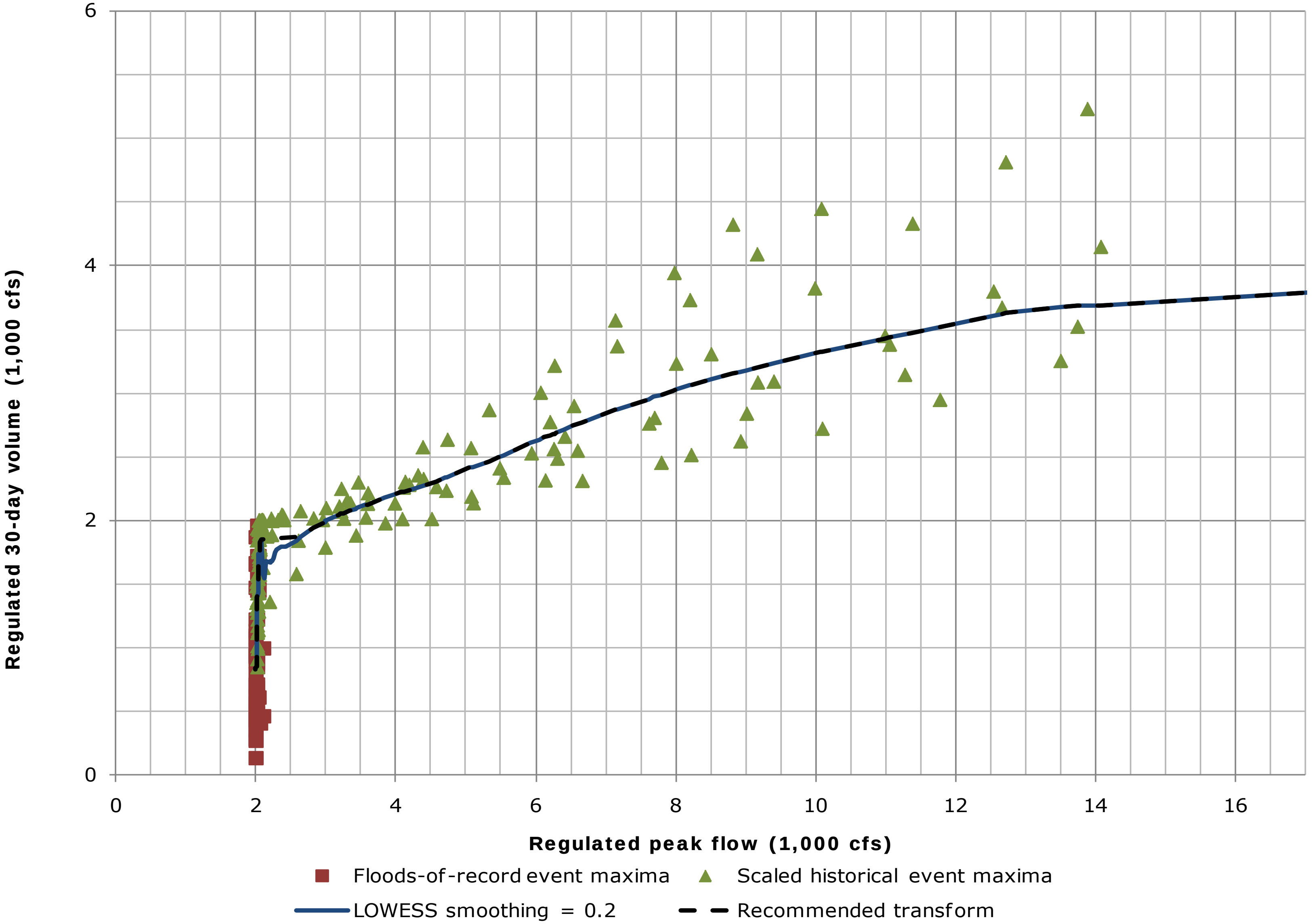


Figure 33. Littlejohn Creek at Farmington, CA, regulated characteristic curve: 30-day duration

Attachment 7: Quality control certification

David Ford Consulting Engineers, Inc. completed Task 3, development of flow frequency curves, expected hydrographs, and documentation of procedures for contract W91238-09-D-0004—Lower San Joaquin River Feasibility Study, San Joaquin County, CA including Stockton City and nearby communities.

Notice is hereby given that all quality control activities of the technical memorandum prepared by the firm have been completed, appropriate to the level of risk and complexity inherent in the project, as defined in the Quality Control Plan. Compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This includes review of assumptions; methods, procedures, and material used in the analyses; the appropriateness of data used and level of data obtained; and reasonableness of the results, including whether the product is consistent with law and existing Corps policy.

REGISTERED PROFESSIONAL ENGINEER
DAVID THURMAN FORD
No. 33889
Exp. 6/30/2012
CIVIL
STATE OF CALIFORNIA

3/25/2011

David T. Ford, PhD, PE, D.WRE
President
David Ford Consulting Engineers, Inc.

(date)

Appendix 3

Lower San Joaquin River Feasibility Study

Hydrologic Analysis for Bear Creek, Mosher Slough, Calaveras River watershed below Bellota, and French Camp Slough

US Army Corps
of Engineers.

April 2014

Table of Contents

Attachments

1. Attachment 1: Lower San Joaquin River Feasibility Study, F3 Hydrology Appendix by Peterson, Brustad, Inc. dated July 30, 2012.

References

1. Lower San Joaquin River Feasibility Study, F3 Hydrology Appendix by Peterson, Brustad, Inc. dated July 30, 2012.

1.0 Background

This Appendix covers the hydrologic analysis for Bear Creek, Mosher Slough, Calaveras River watershed below Bellota, and French Camp Slough. Peterson Brustad, Inc. (PBI) studied these watershed areas (Ref 1) while the Sacramento District of the US Army Corps of Engineers (SPK) and David Ford Consulting Engineers (DFC) analyzed: 1) New Hogan Dam down to the downstream control point (Mormon Slough at Bellota) and 2) Farmington Dam down to the downstream control point (Littlejohn Creek at Farmington, Ca). PBI studied the portions of the watershed that required rainfall runoff models due to a lack of sufficient gaged flow data; while SPK and DFC analyzed the largely regulated portions of the study area that could be analyzed via measured flows and reservoir simulation models.

The first part of this appendix describes multiple analyses performed jointly by PBI and SPK after the initial ATR review. These were meant to address address concerns about 1) the calibration of the lower Calaveras River HMS model and 2) the nature of the design storms. The concerns about the storm include: a) the design storm was not balanced to multiple durations (PBI balanced a 1997 pattern hydrograph to only one duration – the 72-hour NOAA14 depth) b) only one areal reduction factor was applied to the storm (72-hour) and c) the adopted storm centering approach for the area downstream of Bellota caused a lack of clarity about what the hydrographs at downstream index points actually represented (i.e. a specific frequency flow or something else). The PBI Report for Bear Creek, Mosher Slough, Calaveras River watershed below Bellota, and French Camp Slough is attached to this Appendix 3 to provide further details on their analysis.

Significant Findings: The results of the new rainfall runoff calibration efforts indicated that the original PBI modeling parameters were appropriate and did not need modification. For the design storm concerns, SPK created a fully balanced design storm for the 1-,3-, 6-, 12-, 24-, 48-, and 72-hour depth durations from NOAA14 using the January 1997 storm pattern. The appropriate HMR 59 areal reduction factors that go along with each duration were applied. The storm centering method was to assume a storm bullseye for the whole drainage area above Mormon Slough at Bellota, with only concurrent and more common frequency rainfall occurring between Bellota and Stockton. As the Bellota hydrograph that was fed into the upper end of the HMS model was based on an unregulated flow frequency analysis plus reservoir routing simulations, it truly represents an n-year flow event. For index points downstream of Bellota, the hydrographs represent what would happen when you have an n-year event centered above Bellota and concurrent runoff downstream. For these reasons, the hydrographs produced in the HMS model are probably not significantly different than if PBI had created a specific storm centering for each and every index point: 1) the majority of runoff that gets into the levee system comes from sources above Bellota (approximately 75% or more) 2) the lower watershed is heavily leveed downstream of Bellota and only a few locations exist where water can enter into the levee channels.

2.0 HMS model calibration

2.1 Background: The firm PBI performed rainfall runoff modeling for the Lower SJQ River Feasibility Study. PBI developed an HEC-HMS model for the lower Calaveras River downstream of New Hogan Dam. This model was then integrated with a separate reservoir modeling analysis of New Hogan Dam, performed by David Ford Consulting Engineers (DFCE), in which the flow at Bellota (for the 1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, and 1/500 AEP events) would be derived from the reservoir operation analysis, and coincident local flows below the Bellota gage would be derived from the HEC-HMS model developed by PBI. Rainfall runoff model calibration was performed for the local flow areas between New Hogan Dam and the Bellota gage. Modifications to the base parameters needed to match the observed flow at Bellota for the April 2006 storm event were then applied to the ungaged watershed areas in the study. PBI calibrated their model using the recorded hourly flow at the Mormon Slough at Bellota gage. The flow at this location represents both New Hogan Dam releases and local flow runoff from the approximately 107 square mile area between the dam and the stream gage. To accomplish getting a similar hydrograph from their model, PBI took the recorded reservoir outflow hydrograph shown in figure 1 shown below and routed it from New Hogan Dam location to the Bellota index point where it was combined with the local flow hydrograph produced by their rainfall runoff simulation. For their final simulation, PBI adopted a basin “n value” of 0.15 and constant soil loss rates of 0.85 times the handbook values. The final calibration run with their adopted parameters is shown in Figure 2.

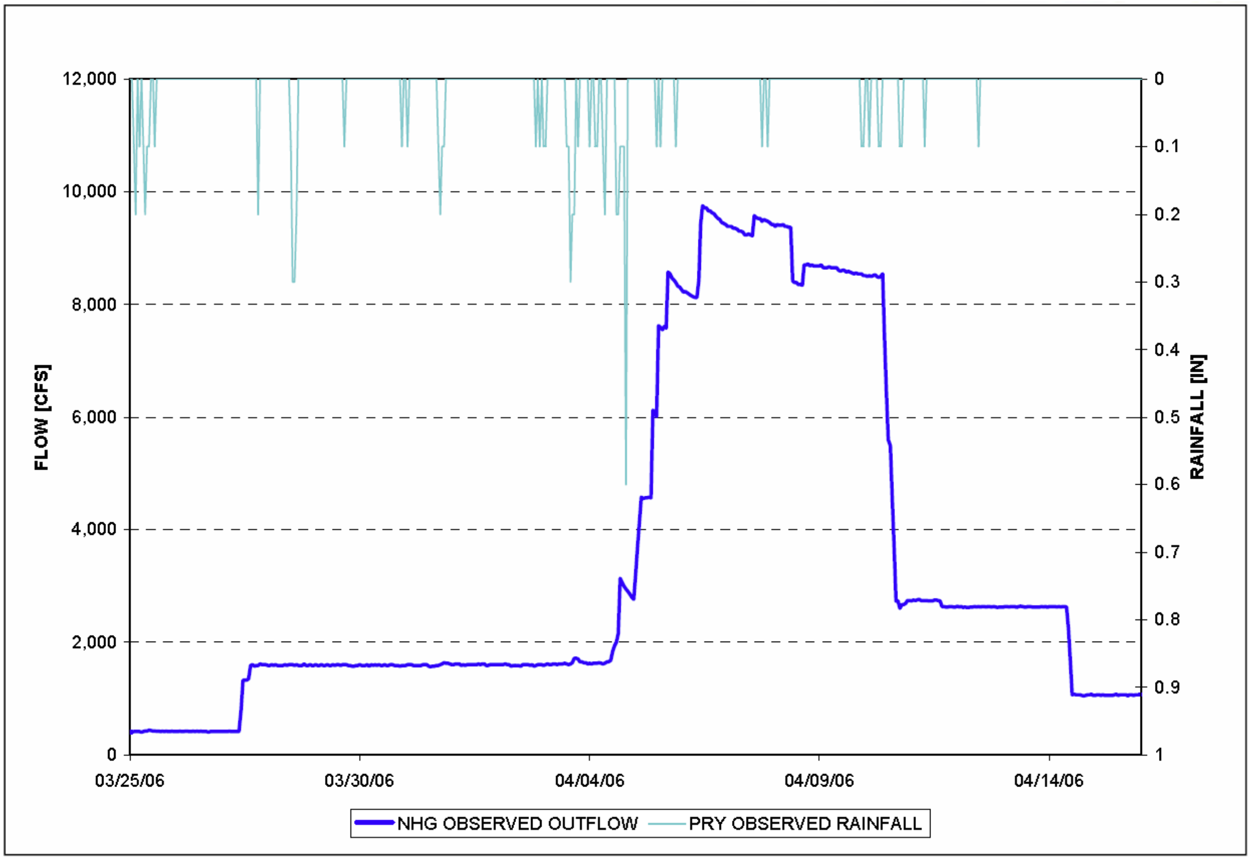


Figure 1: Observed outflow from New Hogan Dam. This hydrograph was routed downstream and added to local flow computed in HMS.

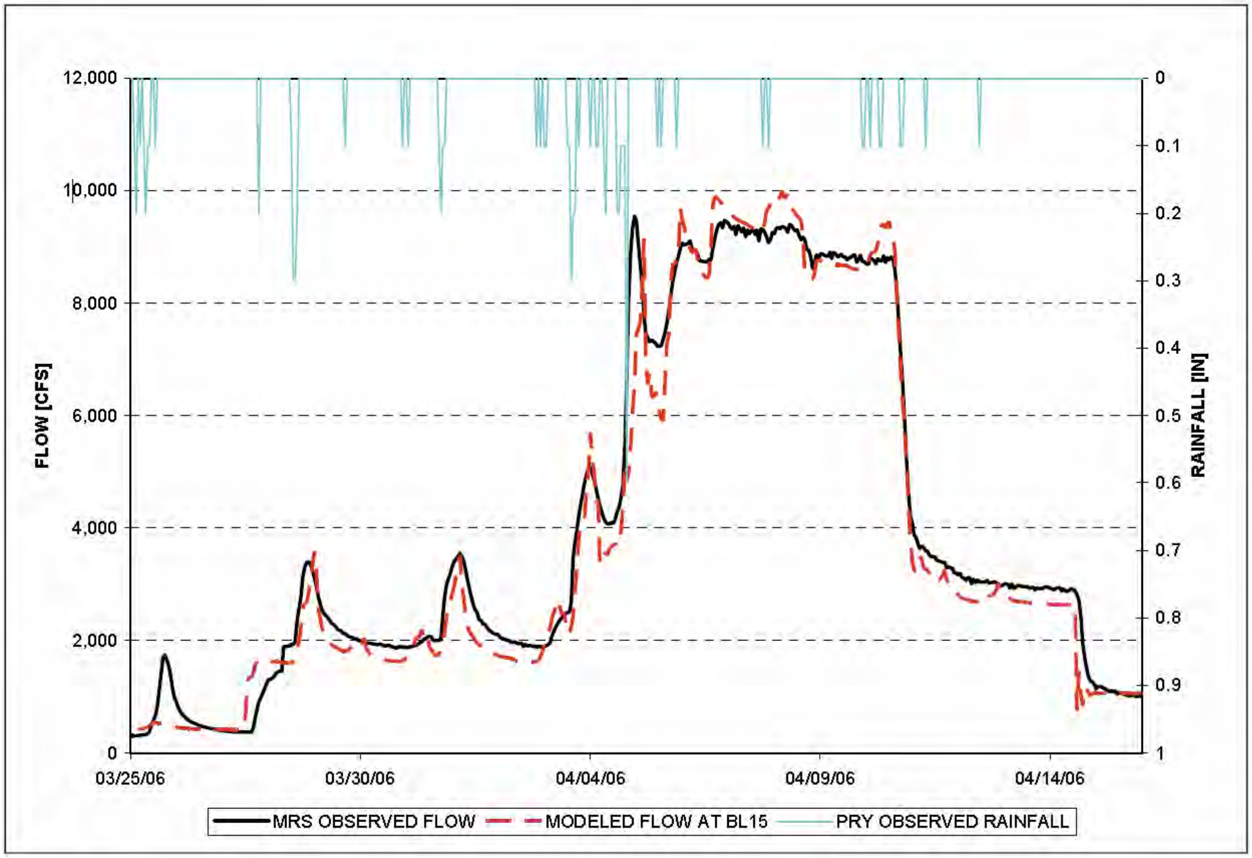


Figure 2: Observed and modeled flow at Bellota. Both hydrographs include both outflow from New Hogan Dam and additional local flow contributions.

2.2 Issue: During the ATR review of the Hydrology Appendix, it was recommended that the calibration results be compared for the local flow below New Hogan Dam only, rather than total flow at the Bellota gage (which includes New Hogan Dam outflow). DFCE had previously developed hourly local flow hydrographs by subtracting observed reservoir releases (routed downstream to Bellota) from the total flow observed at the Bellota gage. SPK provided the Ford local flow hydrographs for the 1997 and 2006 floods to PBI. PBI then performed calibration runs without the New Hogan Dam reservoir releases. Initial results for the 1997 calibration run are shown in figure 3. The model came up significantly short in peak and volume. An attempt was made to lower the basin n (shorten lag) but the timing of the peak became too early as shown in figure 4. The basin n was then restored back to 0.15. Next, several attempts were made to adjust which precipitation gages were assigned to each rainfall zone, and by dropping soil loss rates down to the lowest range possible (per handbook guidance). The model still came up short in peak and volume! The only positive result from the calibration was confirmation of an appropriate basin n value of 0.15 as it also worked well for the 2006 calibration effort. The effort to use the 1997 event for calibration was abandoned since precipitation data was apparently too low (insufficient).

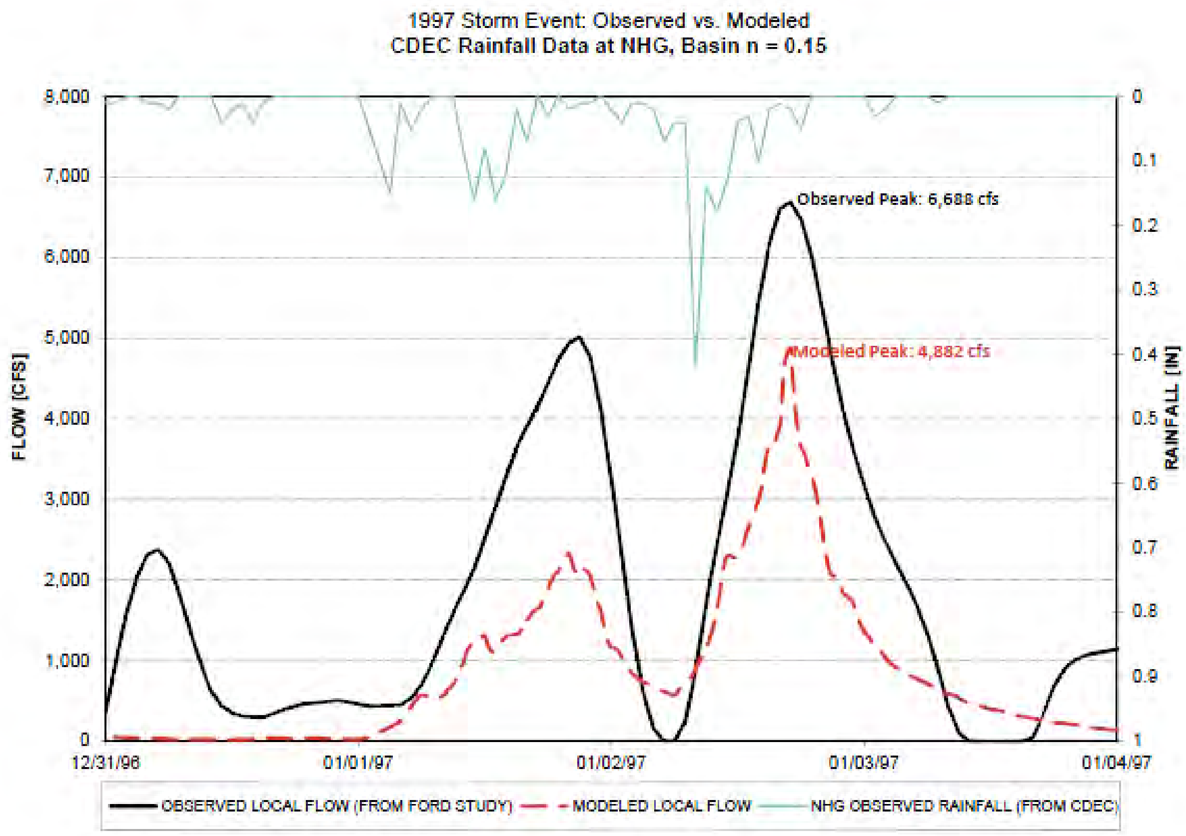


Figure 3: Initial comparison of observed (computed from gage data) to simulated local flow between New Hogan and Bellota (1997 event). Both peak and 3-day volume were found to be low.

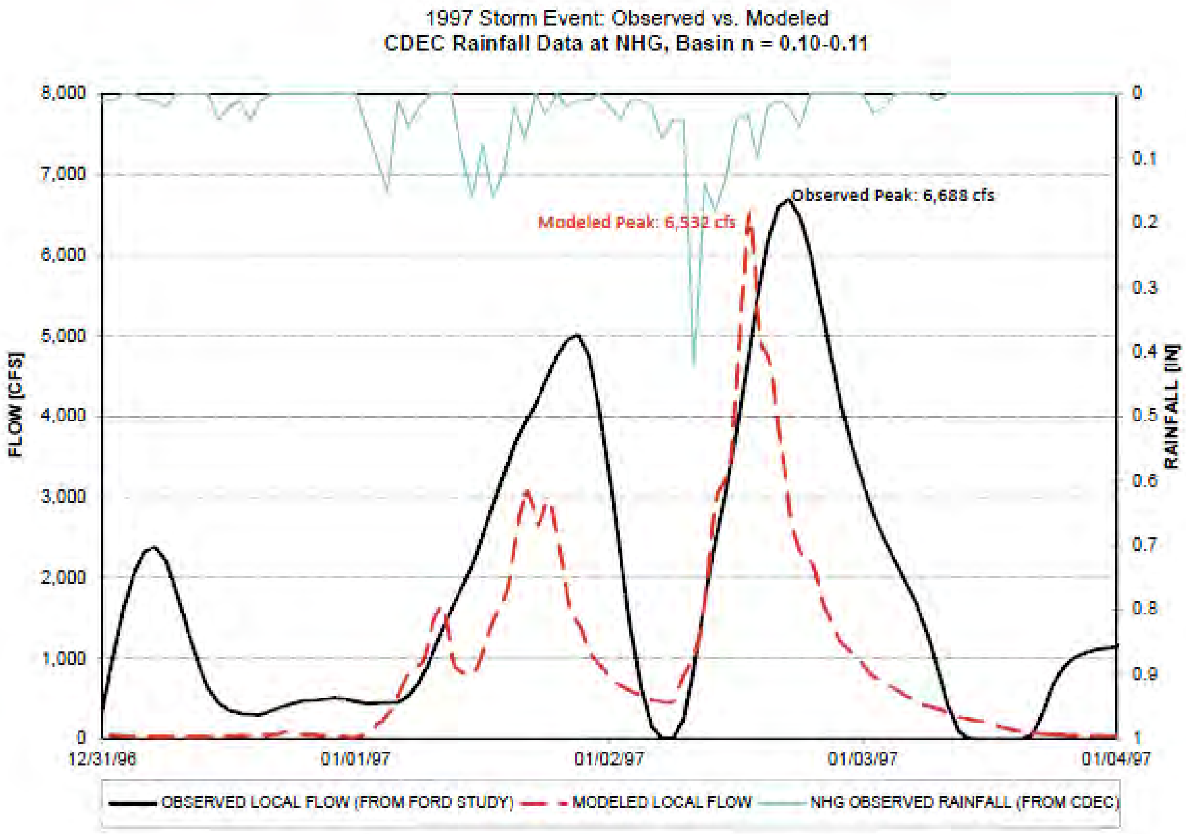


Figure 4: 1997 event calibration with reduced basin n.
Note: Timing is too early.

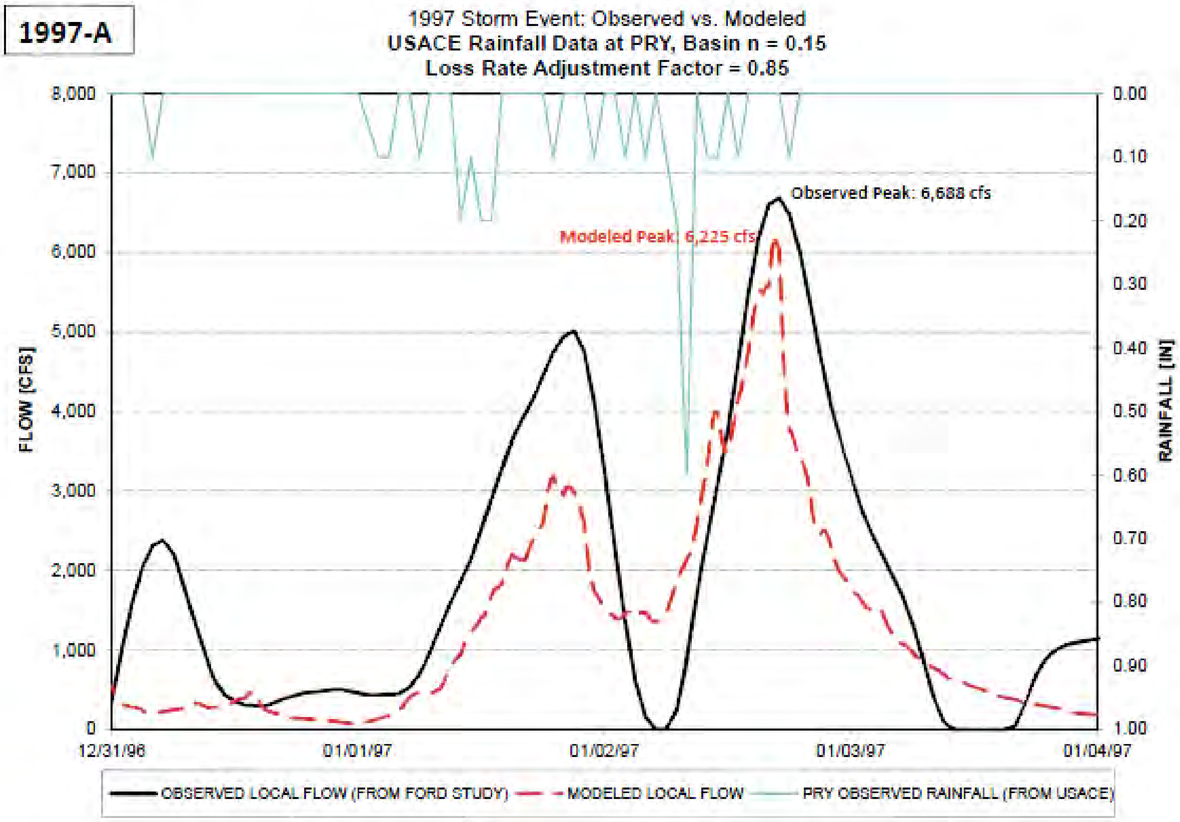


Figure 5: 1997 event calibration using adjusted rainfall ..
Note: Peak and volume still comes up short.

2.3 2006 Event Calibration. Next, the model was re-calibrated to the 2006 flood event. The intial calibration run resulted in Figure 6 below. The model came up short in peak and volume.

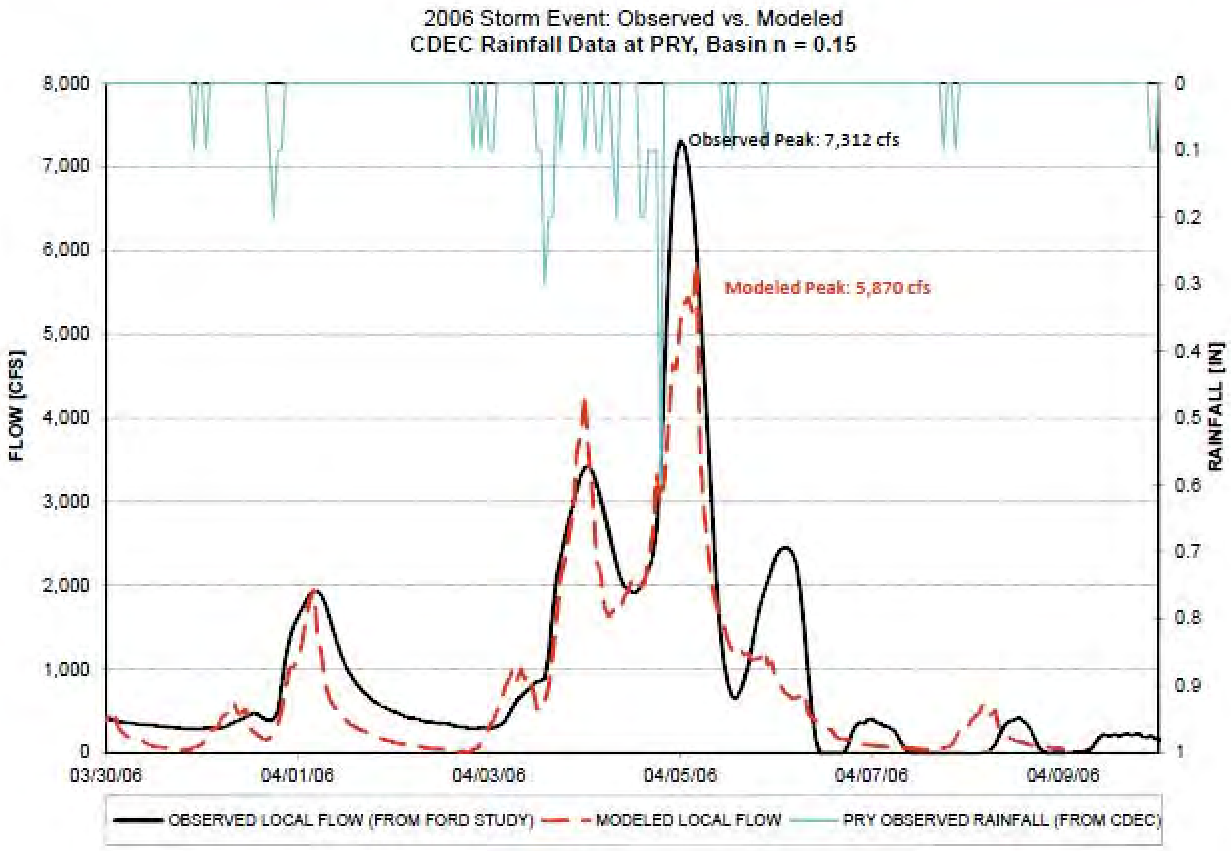


Figure 6: Initial 2006 event calibration
Initial comparison of observed (computed from gage data) to simulated local flow between New Hogan and Bellota (2006 event). Both peak and 3-day volume were found to be low.

2.4 Resolution: Several modifications to the HMS model parameters were investigated, before a final calibration was adopted:

1) The unit hydrograph parameter "basin n" was modified to create a more peaked unit hydrograph. This modification caused mixed results as shown in figure 7. The waves around either side of the main wave appear to occur too early as compared to figure 6. As this did not seem desireable, the original basin n value of 0.15 was restored.

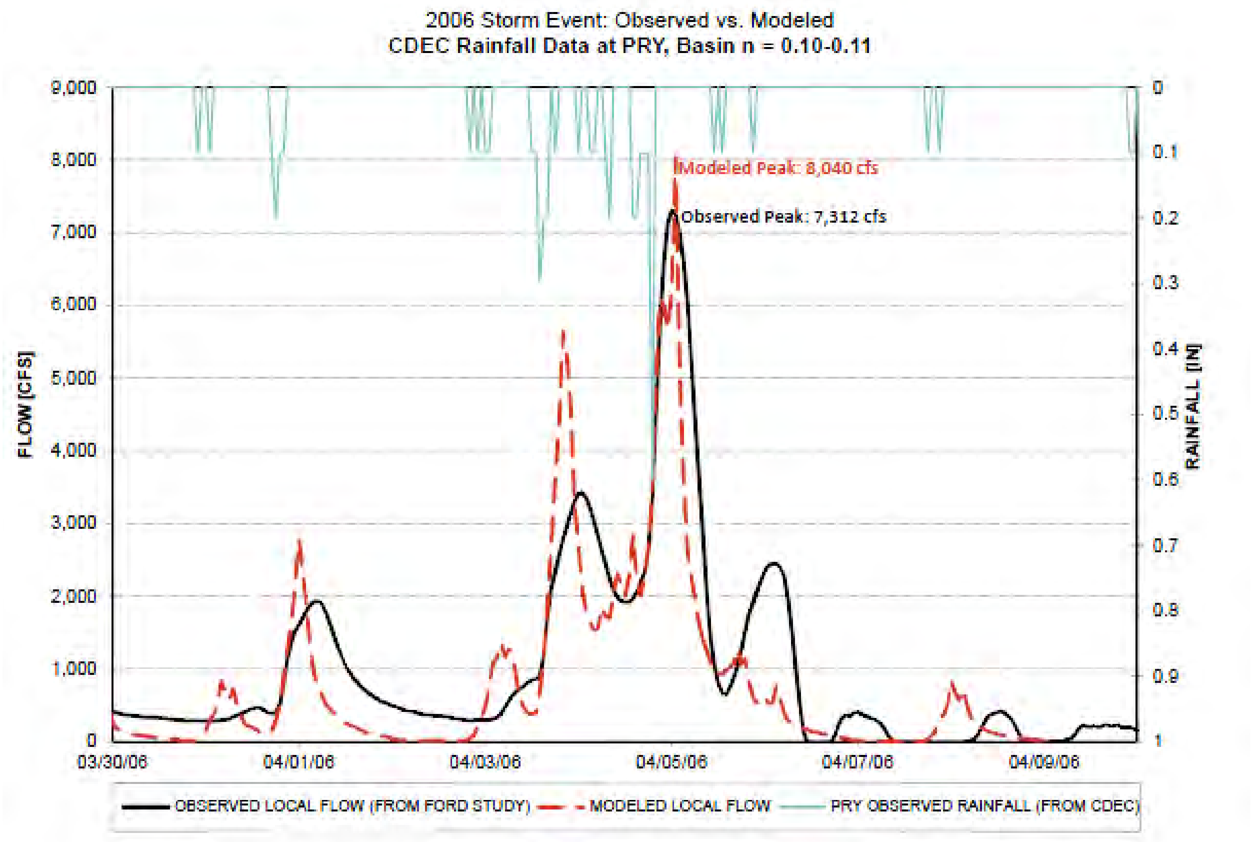


Figure 7: 2006 Event with reduced basin n.
Note: Attempts to match peak by reducing the basin n were found to provide mixed results; peak of the main wave is okay but the pre- and post-waves happen earlier than figure 6

2) The next attempt to get a better match was to lower soil loss rates. In order to get a good match to peak and volume, some soil types had to be lowered below the lower limit of the range suggested in handbooks. Consequently, this adjustment was abandoned and HEC-HMS soil loss rates adopted by PBI were restored (85% times the average soil loss rate per soil type).

3) The last step was to modify the precipitation. To do this, logically based re-assignments were made as to which observed gage hyetographs were assigned to each subbasin, on the basis of comparative proximity and representative elevation. For the 2006 calibration, the model performed very well with this adjustment. The gages used in the calibration are shown in Figure 8.

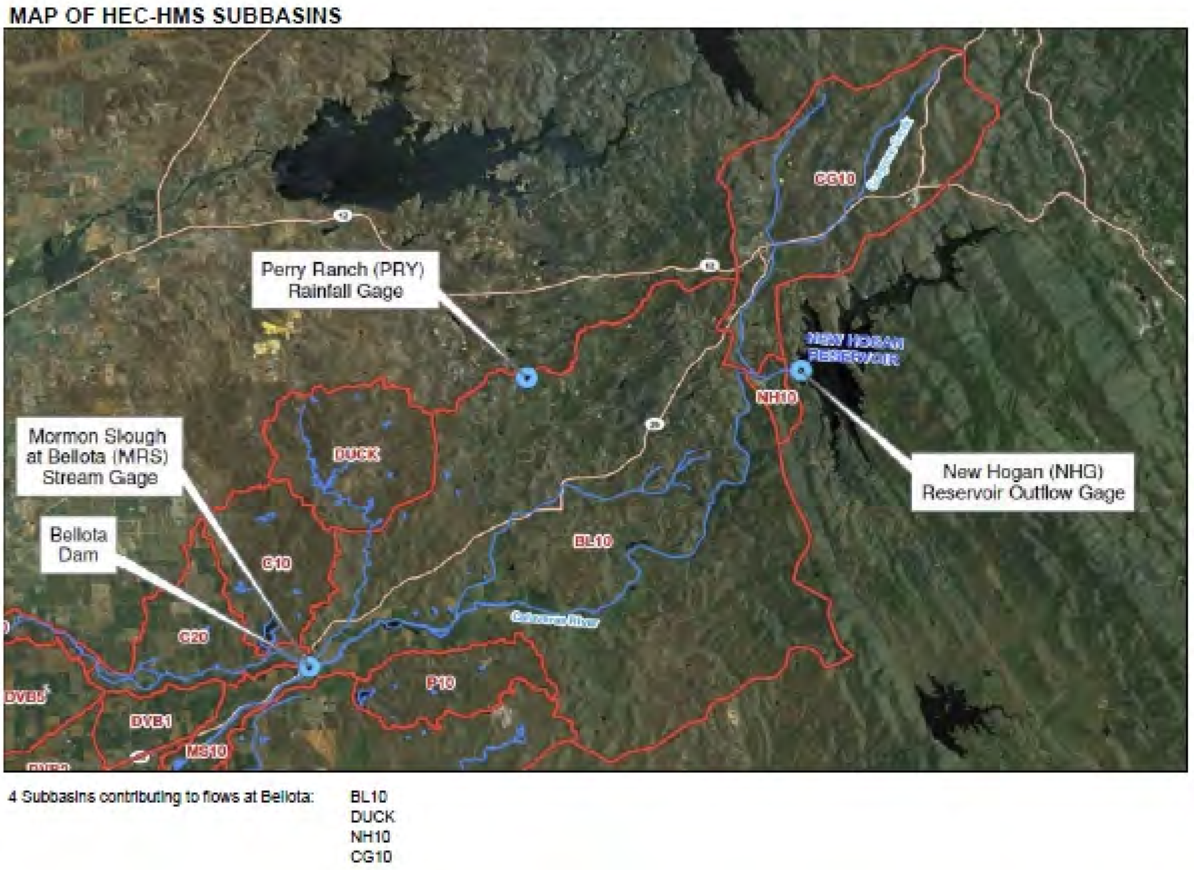


Figure 8: Precipitation gage locations in relation to the subbasins comprising the local flow between New Hogan Dam and the Mormon Slough at Bellota gage. Rainfall at the New Hogan gage is believed to be more representative of CG10 and NH10 subbasins, while an average of both Perry Ranch and New Hogan seems appropriate for the local areas downstream. Note: this applies to the 2006 event. Precipitation recorded at New Hogan appears to be significantly underreporting in relation to the volume of runoff observed.

4) The final calibration run for 2006 used the original calibration parameters of a basin n of 0.15 and average constant soil loss rates from the handbook times 0.85. The rainfall was modified per discussion under Figure 8. The final adopted calibration run is in Figure 9 below.

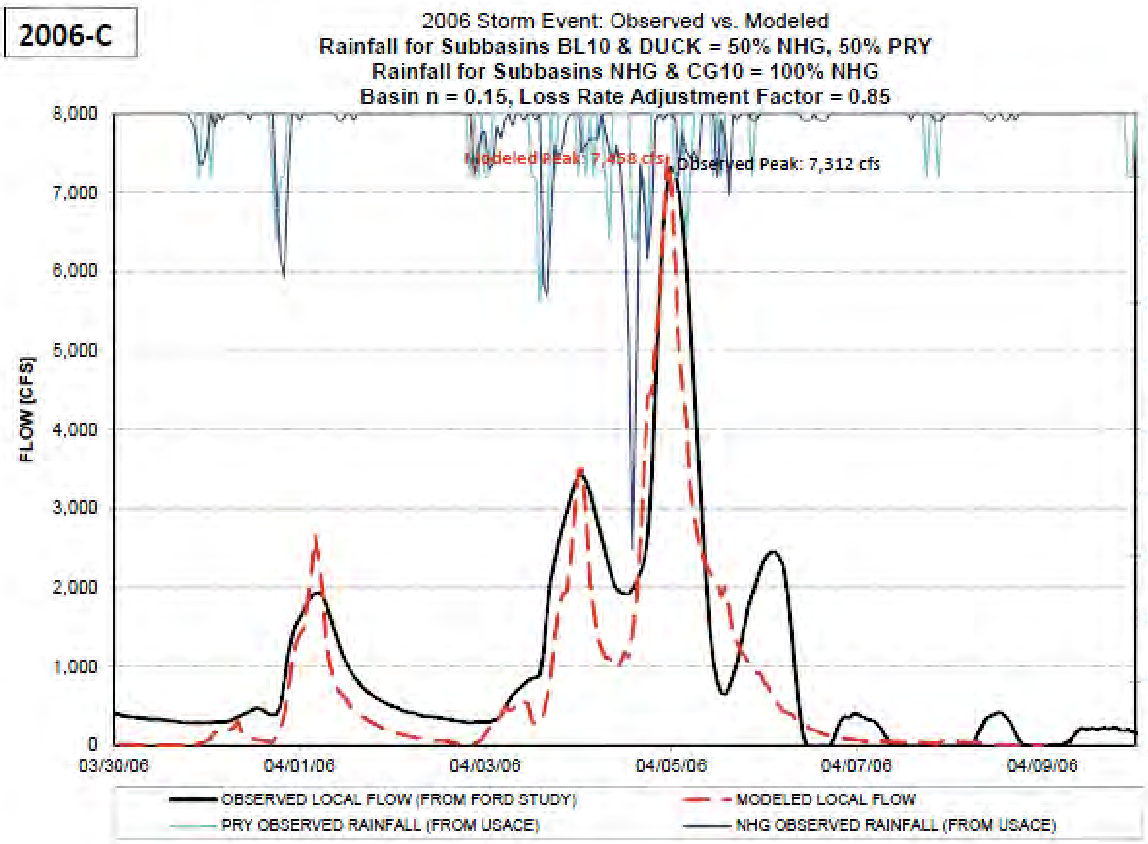


Figure 9: Calibration results for the 2006 event were significantly improved with the modified precipitation gage selection.
Note: The original PBI calibration parameters of basin n = 0.15 and constant soil loss rates = 85% of average handbook values was used for the final calibration run.

2.5 Summary: A reasonable recreation of the 2006 event was achieved by reassigning the observed precipitation gages used for each subbasin to those logically expected to be representative of their respective drainage area. For the 1997 event, no matter which gages were assigned to each subbasin, the model always came up short in peak and volume. Ultimately, this calibration was abandoned as it was realized that the rainfall data was not adequate to accurately model this event. In conclusion, the new calibration efforts reinforced to the Corps that the original model parameters that were adopted by PBI for the n-year simulations were acceptable. As such, no adjustment of the hydrology was deemed necessary.

3.0 Design Storm Sensitivity Analysis

<u>Background:</u> The original design storm created by PBI used a 1997 pattern storm that was balanced to the average 72-hour depth found in NOAA Atlas 14 for the 140 square mile area downstream of the Mormon Slough at Bellota gage. This was justified by a test that the firm performed earlier in the study. In this test, PBI used the balanced storm feature in HEC-HMS (balanced to the 1-hour through 72-hour NOAA14 depths) for a 0.005 AEP storm centered over the area between New Hogan Dam and the Bellota gage. HMS automatically applies TP 40 areal reduction factors to this storm. This was compared to HEC-HMS results when an observed 1997 hyetograph pattern was balanced only to the NOAA14 72-hour depth for the 0.005 AEP event (with HMR 59 areal reduction applied for a 72-hour duration). The resulting peak flow was 12,500 cfs in both cases.

<u>Issue:</u> During the ATR review of the Hydrology Appendix, concern was expressed about the design storm including: a) balancing the pattern hyetograph to <u>only</u> the 72-hour duration b) applying areal reduction only to the 72-hour depth of the design storm rather multiple durations and c) PBI's use of an average of two types of centerings to determine the 72-hour areal reduction factor. These two centerings were the "above New Hogan Dam centering" and the Bellota Centering (storm centered on the area between the dam and the Bellota gage.

<u>Resolution:</u> To address the above concerns, the Corps created a new 0.01 AEP balanced design storm to run in the model for comparison with the PBI design storm results. The Corps' design storm used a 1997 pattern hyetograph that was manipulated/balanced to the 1-hour through 72-hour NOAA14 depths (1-, 3-, 6-, 12-, 24-, 48-, and 72-hour durations).

The areal reduction factors applied to this new design storm were designed to produce concurrent rainfall downstream of Bellota when the entire drainage area upstream of Bellota was having a 0.01 AEP storm (storm that creates 0.01 AEP runoff at the Bellota gage). The following steps were utilized to determine the depths to use in each subbasin:

1. For each duration (i.e 1-hour, 2-hour, etc), use GIS to determine the average 0.01 AEP NOAA14 point rainfall for the entire watershed upstream of the Bellota gage.
2. Apply the appropriate HMR 59 areal reduction factors to the point precipitation depths found in step 1 (use reduction factor for drainage area above the Bellota gage) .
3. Multiply the areally reduced depths found in step 2 by the drainage area upstream of the Bellota gage to get a volume of precipitation (per duration).
4. Repeat steps 1 through 3 for the entire watershed of the Calaveras River. Compute the rainfall volumes (per duration) for the entire watershed.
5. Subtract volume found in step 3 from the volume found in step 4 for the entire watershed. This must be done for each duration (i.e. 1-hour, 2-hour, 3-hour, etc). The result is the remaining volume that can be applied to the watershed area downstream of

Bellota (i.e. when the whole watershed is incurring a 0.01 AEP event with a specific "bullseye" above Bellota).

6. To account for orographic influences (rather than apply the same depth to all subbasins), find the relative "weighting" of every subbasin that is downstream of Bellota. First, multiply each subbasin area by its mean annual precipitation (MAP). Each subbasin's MAP can be found using GIS. The multiplication will create a volume "x". Next, divide each subbasin's "x" by volume "y" (total area ds of Bellota times its MAP). This will result in a ratio/percentage which is the percentage of volume found in step 5 that is to be applied to each subbasin.
7. Finally, divide the volume allotted to each subbasin (based on step 6) by the subbasin drainage area. This is the depth (per duration) that is to be applied to the design storm for each subbasin.

Summary: The above design storm was run in the HEC-HMS model for two scenarios. One scenario included applying the Bellota hydrograph (which includes New Hogan Dam releases) at the upstream end of the model. The other scenario only looked at the differences in local runoff created by the HMS model (without the Bellota hydrograph). The table below provides a comparison of results between the PBI design storm versus the Corps revised design storm at the farthest downstream end of the Calaveras River. This comparison demonstrates the hydrographs in the HMS model are reasonable and do not need modification.

Below is a comparison summary of results at the model outlet:

	Local Flows Only		With New Hogan Outflows	
	Peak Flow [cfs]	Total Volume [AF]	Peak Flow [cfs]	Total Volume [AF]
Current LSJRFS Storm	3,208	7,947	15,603	247,331
Fully Balanced Storm	3,150	7,660	15,544	247,125
% Difference	-1.8%	-3.6%	-0.4%	-0.08%

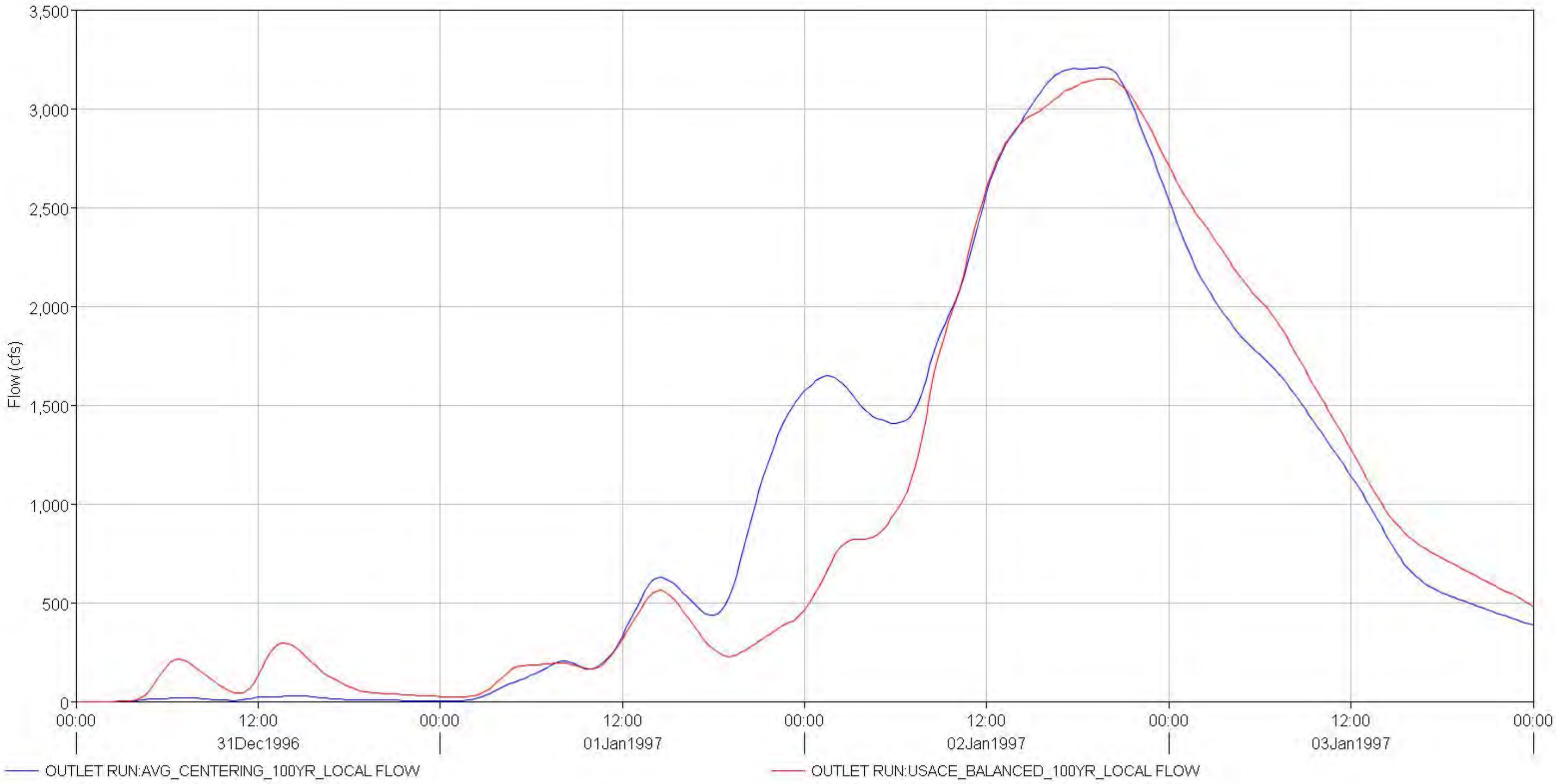


Figure 4: Comparison of local flows only. Blue: original study results using design storm scaled to 3-day duration and area reduction factors. Red: Results using a fully balanced design storm and areal reduction factors.

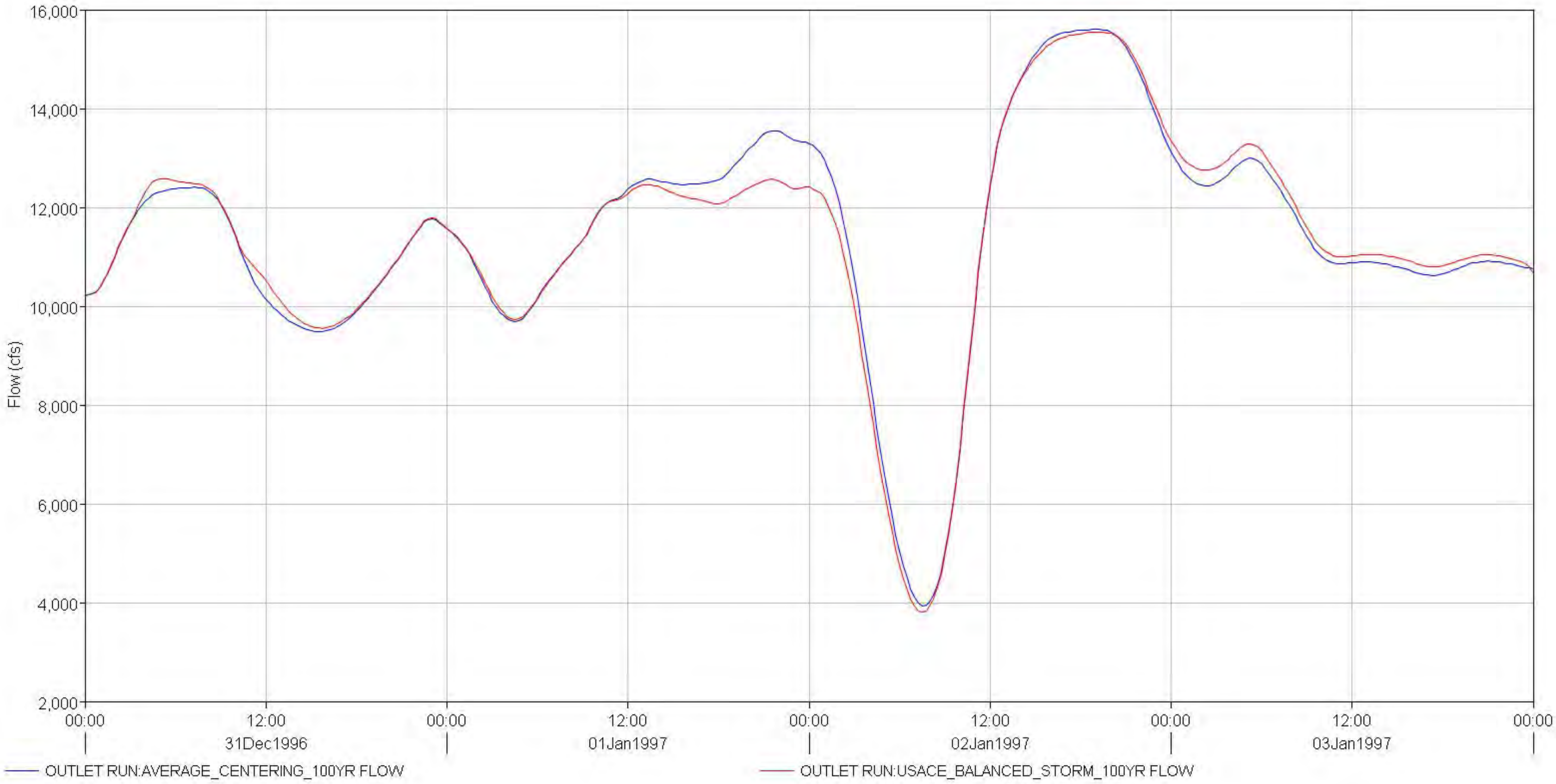


Figure 5: Comparison of total flow at model outlet. Blue: original study results using design storm scaled to 72-hour depth and area reduction factor. Red: Results using a fully balanced design storm and areal reduction factors.

3.1 Applicability to Littlejohn Creek Design Storm

The above sensitivity analyses comparing the results of a fully balanced design storm on the lower Calaveras River to the PBI design storm for the same area may indicate that the HMS modeling results for the Littlejohn Creek below Farmington, CA are reasonable. Like the Calaveras River design storm, PBI used an average of two centerings to create the design storm that was applied to the HMS model areas downstream of Farmington, Ca. These two centerings were the “upper watershed” centering (stress the foothill region) and the “Farmington” centering which stressed the watershed above Farmington Dam. The drainage area downstream of the Calaveras River at Bellota gage is 140 square miles while the drainage area downstream of Littlejohn Creek at Farmington, Ca is 182 square miles. Furthermore, the flow hydrograph on the lower Littlejohn Creek is bifurcated four times and highly attenuated in storage areas downstream of Farmington, Ca which makes the local flow below the Farmington gage less important for this watershed. At the confluence of Littlejohn Creek and Duck Creek where the French Camp Slough levees begin, specific frequency events centered on the mainstem San Joaquin River cause the highest stages due to backwater. The specific frequency flows coming down the tributary do not cause the highest stages within the French Camp Slough levees. To date, the feasibility study has not found an alternative for Littlejohn Creek due to a lack of sufficient annualized damages to justify a project. As the unregulated flow frequency curves at Farmington, Ca are probably conservative (flows on the high side) as stated in Appendix 2, the hydrology has not negatively impacted the study goals.

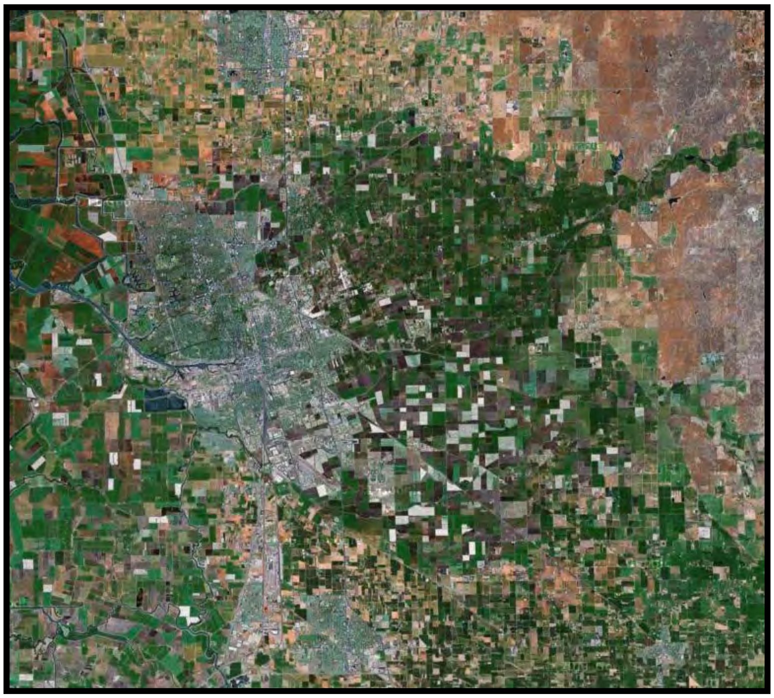
3.2 Bear Creek Design Storm

A 1997 pattern hyetograph fully balanced to multiple duration NOAA14 precipitation frequency depths was used for the study which meets USACE guidelines. The PDT team used the storm centering that caused the worst flow on the Bear Creek for its analysis (assess floodplain damages). From a statistical viewpoint, SPK agrees that an average centering is the more desireable method to provide a best estimate of a specific frequency flow at an index point. Regardless, the feasibility study found that annualized damages on Bear Creek were not high enough to justify a project. As the hydrographs used for the floodplain analysis were probably conservative (too high), the hydrology did not negatively impact the study goals.

Attachment 1

Lower San Joaquin River Feasibility Study, F3 Hydrology Appendix by Peterson, Brustad, Inc. dated July 30, 2012.

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY



F3 HYDROLOGY APPENDIX

JULY 30, 2012

US Army Corps of Engineers

SJAFCA San Joaquin Area FLOOD CONTROL Agency

DEPARTMENT OF WATER RESOURCES
STATE OF CALIFORNIA

PREPARED BY:

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260, Folsom, CA 95630
(916)608-2212

ENGINEER'S SIGNATURE PAGE

This report titled:

Lower San Joaquin River Feasibility Study:
F3 Hydrology Appendix

has been prepared by or under the direct supervision of the following registered Civil Engineers:

REGISTERED PROFESSIONAL ENGINEER
David A. Peterson
No. C 043432
Exp. 06/30/14
CIVIL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
Michael J. Rossiter
No. C 080072
Exp. 09/30/12
CIVIL
STATE OF CALIFORNIA

Table of Contents

List of Figures

List of Tables

1.0 INTRODUCTION

U.S. Army Corps of Engineers (USACE) in conjunction with the San Joaquin Area Flood Control Agency (SJAFCA) and the Central Valley Flood Protection Board is preparing the Lower San Joaquin River Feasibility Study (LSJRFS) to evaluate flood damage reduction projects within the Lathrop to Stockton urban and urbanizing corridor, which includes Bear Creek, Mosher Slough, Calaveras River, French Camp Slough, and Lower San Joaquin River watersheds. This section of the F3 Hydrology Report documents the HEC-HMS model preparation and resulting flows within the study watersheds, with the exception of the Lower San Joaquin River, which will be documented by USACE under separate cover. Hydrologic modeling was performed with a 72-hour storm for the 1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, and 1/500 annual exceedance probability (AEP) events.

2.0 DESIGN STORMS

Design storms with 72-hour durations were created for the 1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, and 1/500 AEP events as input to the LSJRFS HEC-HMS models. As discussed in Section 2.3, the 72-hour storm pattern provides a storm event that is high in both peak flow and volume which is important for levee breach scenarios.

2.1. RAINFALL ZONES

LSJRFS subbasins were aggregated into 7 rainfall zones with uniform rainfall characteristics. Seven rainfall gages were selected to form the basis of this subbasin aggregation. The selected gages are distributed throughout the study area and have available rainfall data at short-interval timesteps which can be used for storm patterning (see Section 2.3).

GIS software was used to draw Thiessen polygons around the selected rainfall gages and subbasins lying within each Thiessen polygon were aggregated to create the rainfall zones (Figure 2- 1).

2.2. DESIGN STORM DEPTHS

The National Oceanic and Atmospheric Administration (NOAA) published its *Atlas 14 Precipitation Frequency Study for California*[1] in April 2011 which includes estimates for design rainfall depths in an ASCII grid file format for use in GIS. A shapefile with 7 defined rainfall zone boundaries was projected on top of the NOAA14 ASCII grid files to calculate average point rainfall depths within each rainfall zone for 96 different frequency-duration combinations.

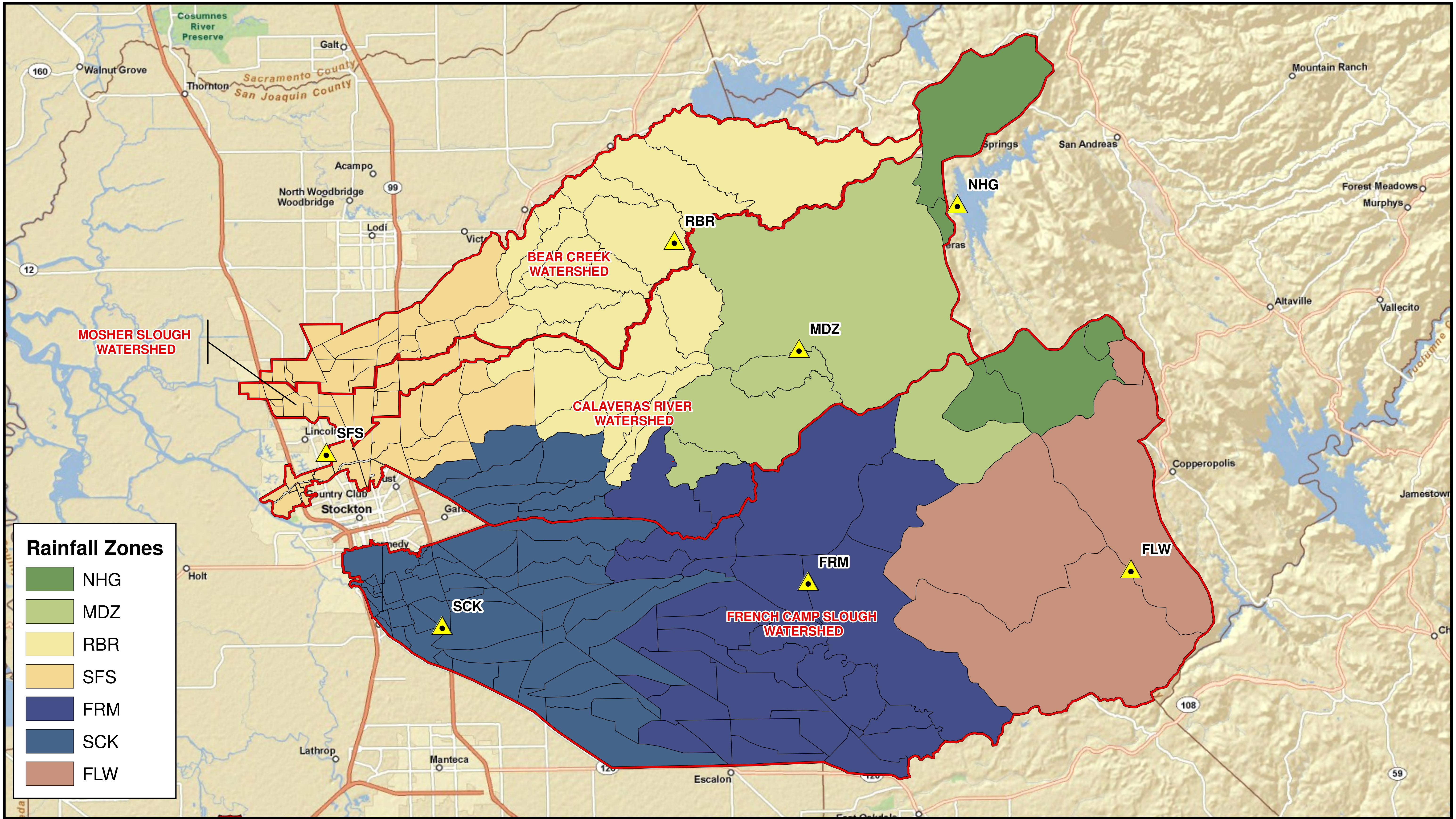
The output from the NOAA14 GIS data acquisition process includes depth-duration-frequency tables for each rainfall zone. These depth-duration-frequency tables are included for each watershed in their respective attachments.

2.3. DESIGN STORM PATTERN

The design storm pattern used for the LSJRFS is based on an observed storm event that was recorded at various rainfall gages within the study area.

The December 31, 1996-January 3, 1997 rainfall event (1997 Event) and the April 2, 2006-April 5, 2006 rainfall event (2006 Event) were considered for the basis of design storm patterning. These events represent two of the largest storms in recent history.

Data records were checked for these events at all known precipitation gages within the vicinity of the study area. Some gages only had recorded data at monthly or daily intervals and were excluded from the gage selection process based on their inadequate time step. Other gages were excluded due to lack of data for the specific dates listed; many of the available rainfall gages did not contain data for the 2006 Event.



Watershed Boundary
Subbasin Boundary
Precipitation Gage

0 5 Miles
1 : 250,000

APRIL 25, 2011

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING
1180 Iron Point Rd., Suite 260
Folsom, CA 95630
Phone: (916) 608-2212
Fax: (916) 608-2232

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

LSJRFS Rainfall Zones

FIGURE

2-1

The 1997 Event is often considered an industry standard for rainfall events and was ultimately selected as the pattern used to temporally distribute the design storms.

Data from the New Hogan (NHG) gage location represents a typical 72-hour hyetograph pattern for the 1997 Event and is shown below.

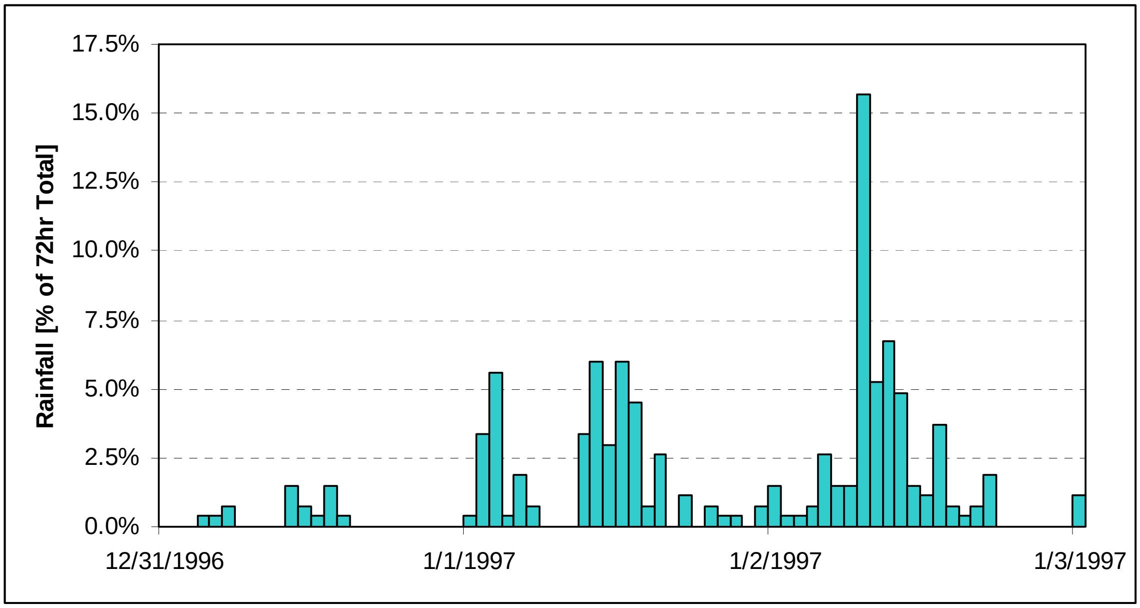


Figure 2- 2. Typical Rainfall Pattern for the 1997 Event.

The 72-hour storm pattern provides a storm event that is high in volume which is important for levee breach scenarios. For the LSJRFS, it is also desirable to preserve the high peak flows that would result from a standard, 24-hour design storm. Therefore, additional analyses were conducted to run a SCS Type 1 storm, an industry standard 24-hour storm, with the same rainfall depths to confirm that the peak flows resulting from using the 72-hour, 1997 Event hyetograph pattern are comparable to the standard, 24-hr peak flows.

All flows were comparable except for those in the Bear Creek watershed. To correct this, Bear Creek hyetographs were patterned after the 72-hour, 1997 Event and then balanced to the 3-, 6-, 12-, 24-, 48-, and 72-hour NOAA14 storm depths. After balancing the hyetographs, Bear Creek models produced high-volume hydrographs with peak flows that are comparable to those resulting from a standard 24-hour design storm.

2.4. STORM CENTERING APPROACH

The LSJRFS utilizes a storm centering approach to consider depth area reduction of design storms falling over the study area. This area reduction is typically disregarded for small watersheds where one point precipitation depth can be applied to the entire tributary area, however given the size of the watersheds in the LSJRFS it is necessary to apply area reduction factors to the point rainfall design storm depths.

Area reduction factors were calculated using a procedure that was developed by the USACE Sacramento District for the hydrology of their *Downtown Guadalupe River Project* in November 2009[2]. This procedure takes into account various storm centerings by ranking the rainfall zones according to their distance from the storm centering location and determining the cumulative drainage area for each location in the watershed. Additional details on the calculation of area reduction factors are discussed in the USACE *Guadalupe River* report provided in Attachment 2-A.

All calculated area reduction factors are included in the depth-duration-frequency tables for each watershed which are provided as attachments.

3.0 BEAR CREEK HEC-HMS MODELING

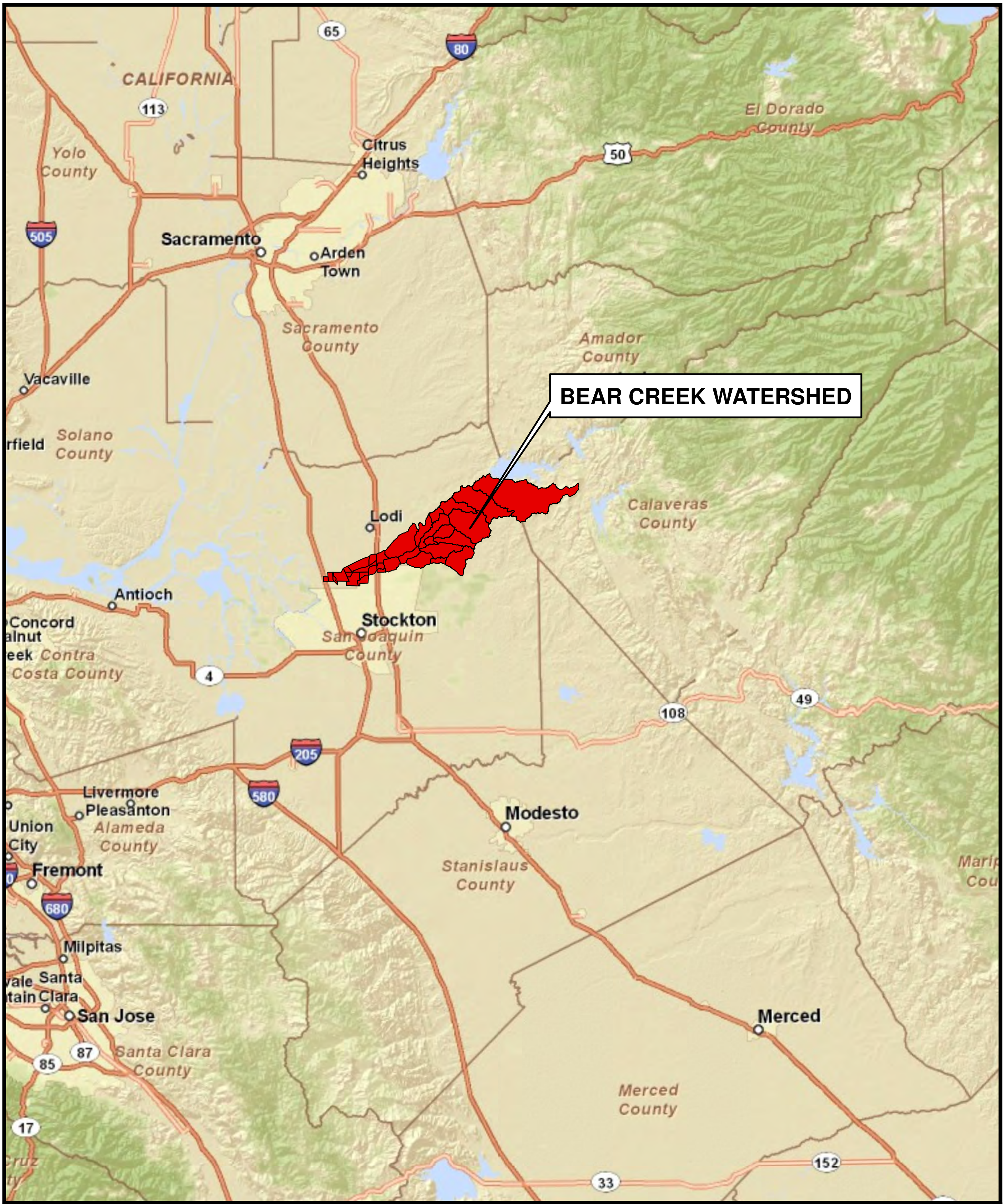
3.1. GENERAL

3.1.1. Location

Bear Creek is located near the city of Stockton in San Joaquin County, California (Figure 3-1). The watershed runs east from the city of Stockton into the Sierra Nevada foothills in Calaveras County and includes a total area of approximately 115 square miles. The uppermost portion of the watershed achieves maximum elevations of 1,000 feet and is not subject to snowmelt. It then descends through moderate slopes to the lower portion of the watershed at sea-level. The HEC-HMS model described in this memorandum has an outlet on Bear Creek at Disappointment Slough and includes Bear Creek, Upper Mosher Creek, Paddy Creek and Pixley Slough.

3.1.2. Topography

The HEC-HMS model utilized for this study is titled the PBI Bear Creek Model (PBI Model) which is georeferenced to the NAD 1983 State Plane California Coordinate System Zone III (U.S. Survey Feet). Vertical elevations are reported in the NAVD 1988 datum. Topography used for model development included United States Geological Survey (USGS) 30-meter Digital Elevation Models (DEMs)[3]. Department of Water Resources (DWR) LiDAR data[4] was also used to confirm subbasin boundaries in the lower portion of the watershed.



PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

VICINITY MAP OF THE
BEAR CREEK STUDY AREA

FIGURE

3-1

3.2. MODEL DEVELOPMENT

The PBI model was developed using HEC-HMS version 3.4[5] and HEC-GeoHMS version 4.2[6]. A summary of the tasks performed are listed below:

1. The 1998 SJAFCA HEC-1 model was imported into HEC-HMS (See Section 3.2.2).

2. Subbasin boundaries were updated using HEC-GeoHMS and United States Geological Survey (USGS) Digital Elevation Models (DEMs)[1] (See Section 3.3.1).

3. Pump stations were coded into the PBI model based on design pumping rates provided by the City of Stockton[7] (See Section 3.3.2).

4. Diversions and channel routing parameters were coded into the PBI Model (See Sections 3.3.3 and 3.3.5, respectively).

5. S-graphs and lag times were assigned to each subbasin (See Section 3.3.4).

6. Loss rates and impervious percentages were coded into the PBI Model (See Section 3.3.6 and Section 3.3.7).

7. The 1/100 AEP event hyetographs from the 1998 SJAFCA HEC-1 Model were coded into the PBI Model for debugging purposes (See Section 3.2.2).

8. The PBI Model was set up to simulate both ‘Existing’ (see Section 3.5.1) and ‘Future-Without-Project’ (see Section 3.5.2) scenario runs.

3.2.1. SJAFCA HEC-1 Model

The PBI Model is a conversion and update of the HEC-1 model developed for SJAFCA by HDR Engineering, Inc. in 1998[8].

The 1998 SJAFCA HEC-1 model utilized the San Joaquin County LA preprocessor to convert S-graphs to unit hydrographs for each subbasin. Three types of S-graphs were obtained from the San Joaquin County Hydrology Manual and used based on the surface condition classification of the subbasin: Foothill, Valley Undeveloped, and Valley Developed. Lag times were calculated by HDR using basin 'n', length of subbasin flow, flow length from the centroid, and slope of the basin.

The 1998 SJAFCA HEC-1 model used the SCS curve number method to account for subbasin losses. Curve numbers typically ranged from 78 to 85 depending on soil type and cover. Attachment 3-A lists the parameters used in the 1998 SJAFCA HEC-1 model and compares them to the parameters used in the 2010 PBI Model.

The 1998 SJAFCA HEC-1 model was calibrated by adjusting basin 'n' values such that the 1/100 AEP rainfall event from the San Joaquin County Hydrology Manual produced the 1/100 AEP peak flood flow estimated for the Bear Creek at Lockeford gage. The frequency plot and statistics for this gage are provided in Attachment 3-B.

3.2.2. Conversion from HEC-1 to HEC-HMS

The 1998 SJAFCA HEC-1 model was successfully imported into HEC-HMS as the fundamental basis for the PBI Model.

Certain features in the HEC-1 software are not supported in HEC-HMS and therefore were not properly transferred during the import process. Pump station data and meteorological data from the SJAFCA HEC-1 model were manually coded into the PBI Model so as to conform to HEC-HMS formatting.

In addition, there are computational differences between the HEC-1 and the HEC-HMS software. One such difference involves the Muskingum-Cunge stream segment routing technique used for the PBI model. In HEC-HMS, channel properties are computed based on the physical characteristics of that channel, whereas in HEC-1 the properties are computed with formulas based on a kinematic wave assumption[5]. This causes minor differences in the flows that are transferred through the routing parameters. HEC-HMS results are preferred because of the refined computational techniques that have been implemented.

For initial PBI Model testing, user-specified hyetographs were assigned to each subbasin based on 1/100 AEP storm data defined in the 1998 SJAFCA HEC-1 model's input files. This storm event was run for debugging purposes and results were made sure to match the SJAFCA HEC-1 model results. Subsequent to initial model testing, PBI modified/refined most model input elements as documented in the following sections.

3.3. MODEL FEATURES

The 1998 SJAFCA HEC-1 model was converted and modified for this study to form the PBI HEC-HMS Model. The PBI Model components are described in the following sections.

3.3.1. Subbasins

Subbasin boundaries used in the 1998 SJAFCA HEC-1 model were cross-checked with USGS 30-meter DEM datasets[3] and modified where appropriate. Subbasin boundaries were delineated using the ArcHydro and HEC-GeoHMS[6] extensions within the ArcGIS software package. These tools utilize geospatial data to interpret drainage patterns and delineate watershed boundaries accordingly. Subbasin outlet points were set similar to the locations utilized in the SJAFCA HEC-1 model. Where available, DWR LiDAR[4] data was used to confirm subbasin boundaries. In the lower portion of the watershed, west of Highway 99, subbasin boundaries were based on the City of Stockton's *Conceptual Storm Drain Master Plan*[11]. This portion of the watershed is developed and the boundaries from the City of Stockton take into account drainage improvements that have been made in the area. The Bear Creek subbasins included in the PBI Model are shown in Figure 3- 2.

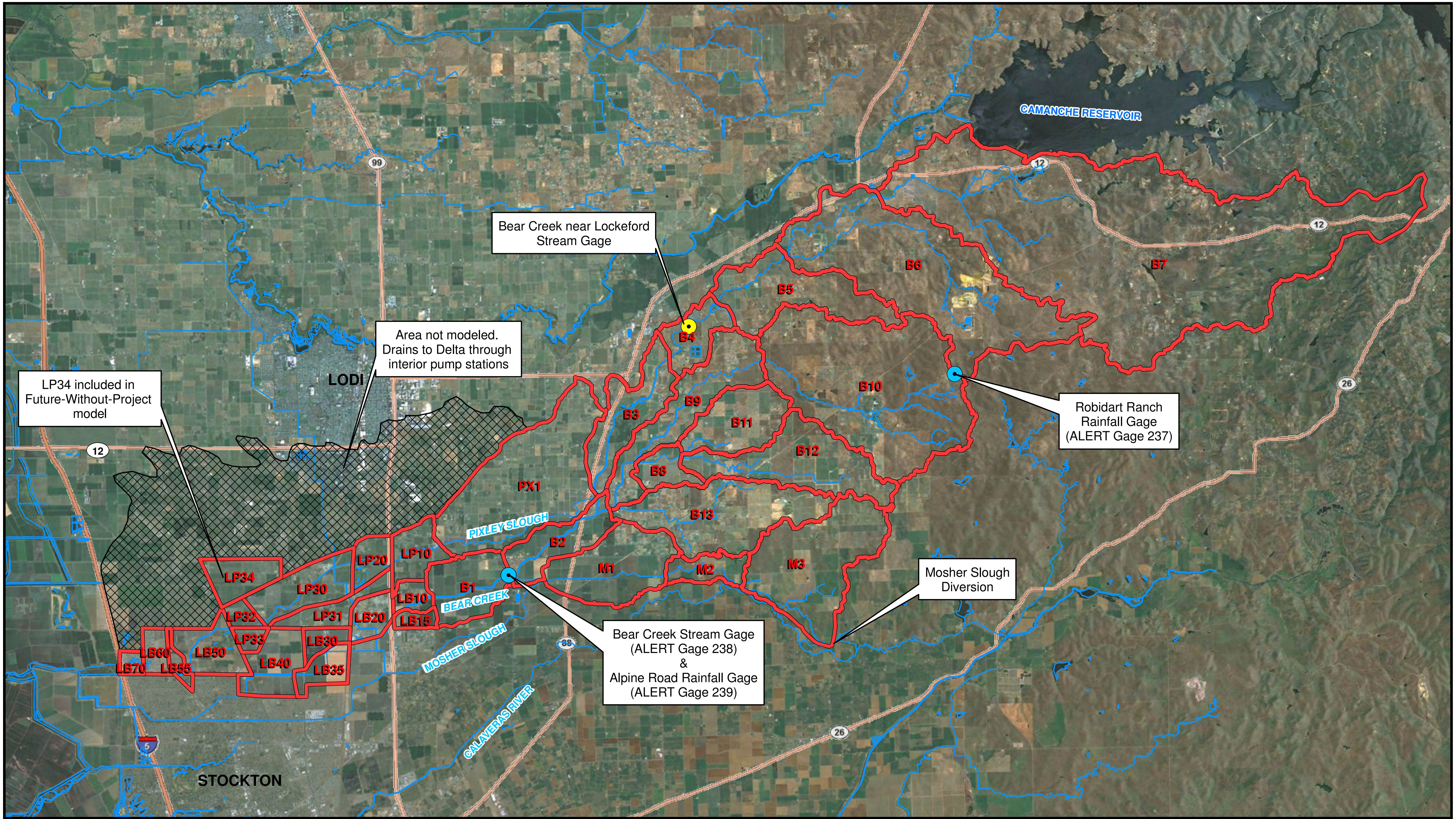
The PBI Model contains a total of 32 subbasins with drainage areas ranging from 0.26 square miles to 30.24 square miles with a total watershed area of approximately 115 square miles. An additional subbasin was added to the 'Future-Without-Project' model to account for added drainage area that is expected to be pumped into Bear Creek.

For subbasins that are on the outside of a levee which do not have pump stations, runoff is coded to enter the main channel at road crossings where there are through-levee culverts. The assumption is made that the culvert headgates will remain open and allow outside flow to enter the main channel. This assumption was made to remain conservative and to account for the potential replacement of culverts by pump stations in the future.

The GIS horizontal coordinates for each subbasin were used to georeference model elements within the PBI HEC-HMS Model. The subbasin GIS shapefile was also inserted into the PBI Model as a background map.

3.3.2. Pump Stations

Pump stations were included in the PBI Model to represent storm drain conveyance from developed subbasins to the main channels. There are three (3) pump stations included in the 'Existing Conditions' model. Multiple pumps are included at each pump station with capacities assigned based on City of Stockton records[7]. All pumps are set to discharge over the top of the levees and into the receiving channel above the highest stage expected. The exterior and interior areas at the pump stations are independent from one another.



0 0.5 1 2 Miles
1 inch = 2 miles

DECEMBER 8, 2011

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

BEAR CREEK HEC-HMS SUBBASINS

FIGURE

3-2

Ten pump stations were then added into the 'Future-Without-Project Conditions' model to represent subbasins that are expected to become developed according to the City of Stockton 2035 General Plan[12]. Pump capacities were assigned at a rate of 0.37 cfs per acre of tributary area. This rate is based on the average flow rates of existing pump stations within the City of Stockton's systems and correlates to approximately 10-year peak flows[8]. The following table provides a summary of pump stations included in the PBI Bear Creek Model.

Table 3- 1. Summary of Bear Creek pump stations.

Pump Station	Contributing Subbasin	Subbasin Area [Sq. Mi.]	Pump Station Status	Pump Station Capacity [cfs]	Pump Station Notes
PLB6070 (I-5 PS)	LB60	0.57	Existing	46.8	3 @ 15.6 cfs
	LB70	0.26			
PLB5055 (Thornton PS)	LB50	1.54	Existing	431	Based on 0.37 cfs per acre
	LB55	0.28			
PLP33 (Pixley PS)	LP33	0.32	Existing	111	3 @ 28.1 cfs 1 @ 6.5 cfs
PLB10	LB10	0.54	Future	128	Based on 0.37 cfs per acre
PLB15	LB15	0.35	Future	83	Based on 0.37 cfs per acre
PLB20	LB20	0.83	Future	197	Based on 0.37 cfs per acre
PLB30	LB30	0.50	Future	118	Based on 0.37 cfs per acre
PLB35	LB35	0.85	Future	201	Based on 0.37 cfs per acre
PLB40	LB40	1.88	Future	445	Based on 0.37 cfs per acre
PLP34	LP34	1.25	Future	296	Based on 0.37 cfs per acre
PLP30	LP30	2.09	Future	495	Based on 0.37 cfs per acre
PLP31	LP31	1.10	Future	260	Based on 0.37 cfs per acre
PLP32	LP32	0.53	Future	126	Based on 0.37 cfs per acre

3.3.3. Diversions

All flows from Upper Mosher Creek (subbasins M1, M2, and M3), which has a combined drainage area of 9.97 square miles, are diverted to the main stem of Bear Creek at a location just upstream of the Central California Traction Railroad (see Figure 3- 2). The Calaveras River has a diversion into Upper Mosher Creek, however there are no flows going over this diversion in the winter. This diversion was originally constructed by the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS), and improved by SJAFCA in 1998. Because the structure diverts all flow, Upper Mosher Creek was coded as a tributary area to Bear Creek. Lower Mosher Slough (downstream of the diversion), will be modeled using a separate HEC-HMS model.

3.3.4. S-graphs and Lag Times

As discussed in Section 3.2.1, the 1998 SJAFCA HEC-1 model utilized the San Joaquin County LA preprocessor for converting S-graphs to unit hydrographs. The PBI Model assigns Foothill, Valley Undeveloped, and Valley Developed S-graphs directly into HEC-HMS for each subbasin based on its location. S-graph data points were obtained from the San Joaquin County Hydrology Manual[10]. The S-graphs were developed based on rainfall-runoff data from Southern California catchments considered to be hydrologically similar to the local catchments. The following figures show the time versus discharge relationship for each S-graph.

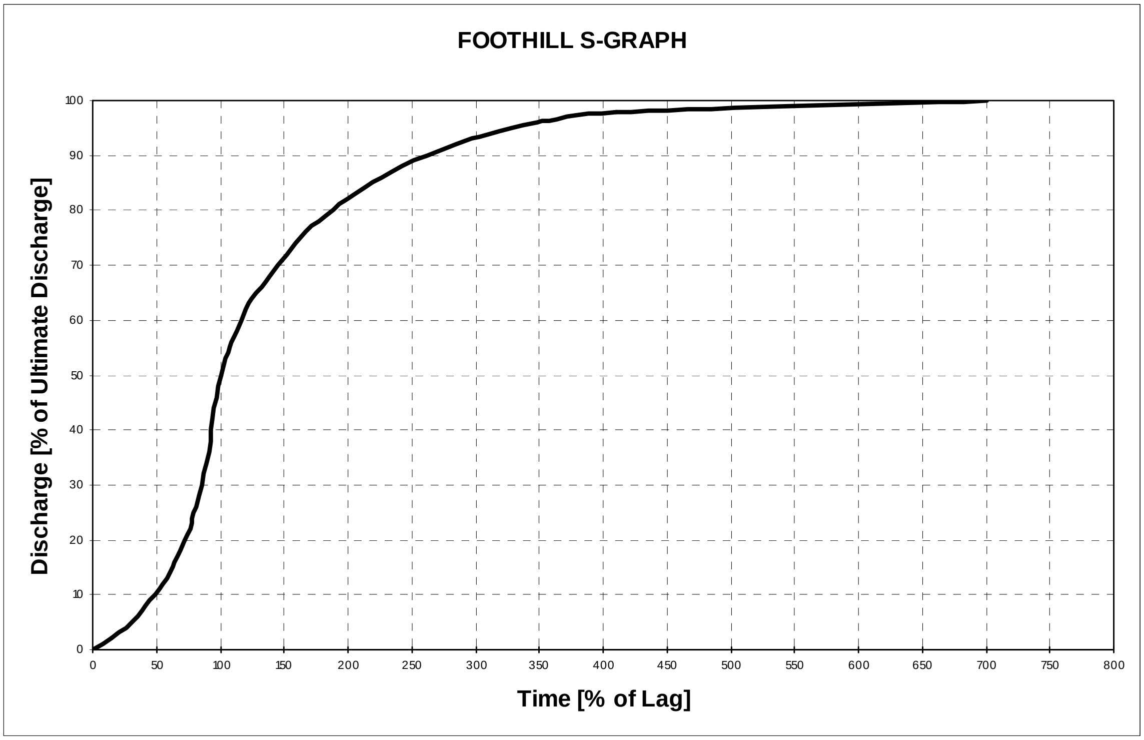


Figure 3- 3. San Joaquin County Foothill S-graph

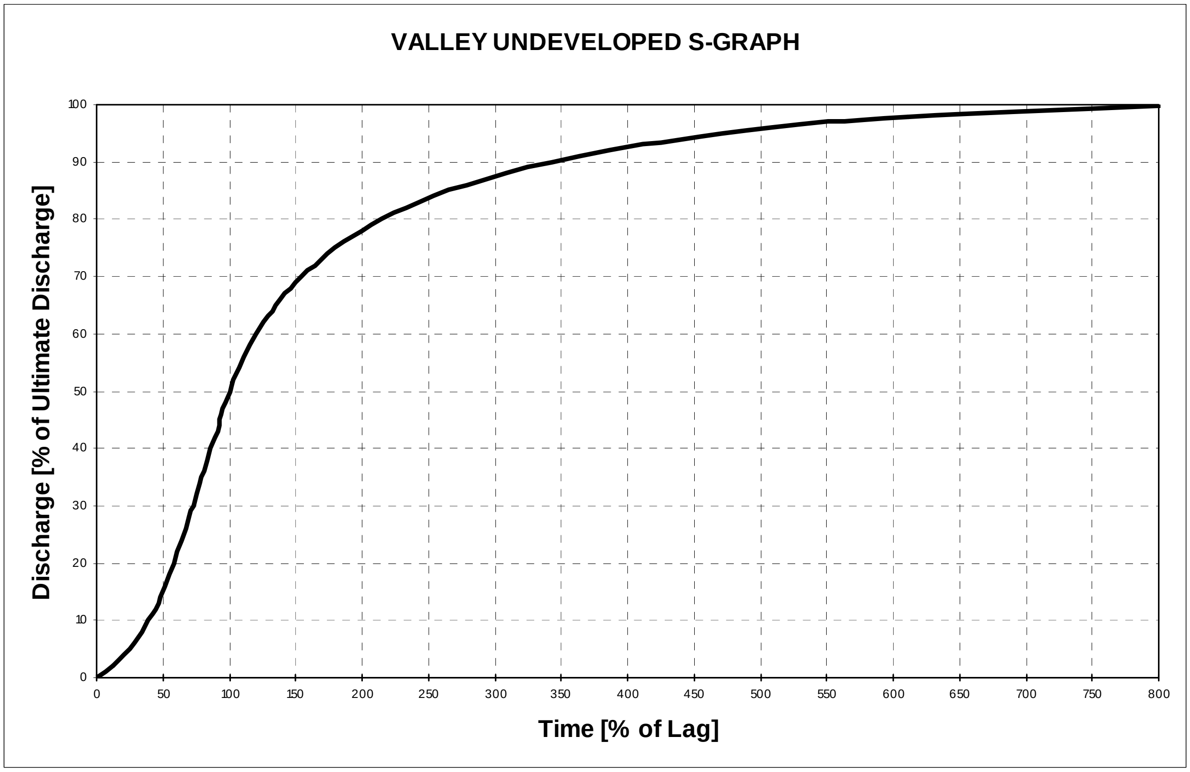


Figure 3- 4. San Joaquin County Valley Undeveloped S-graph

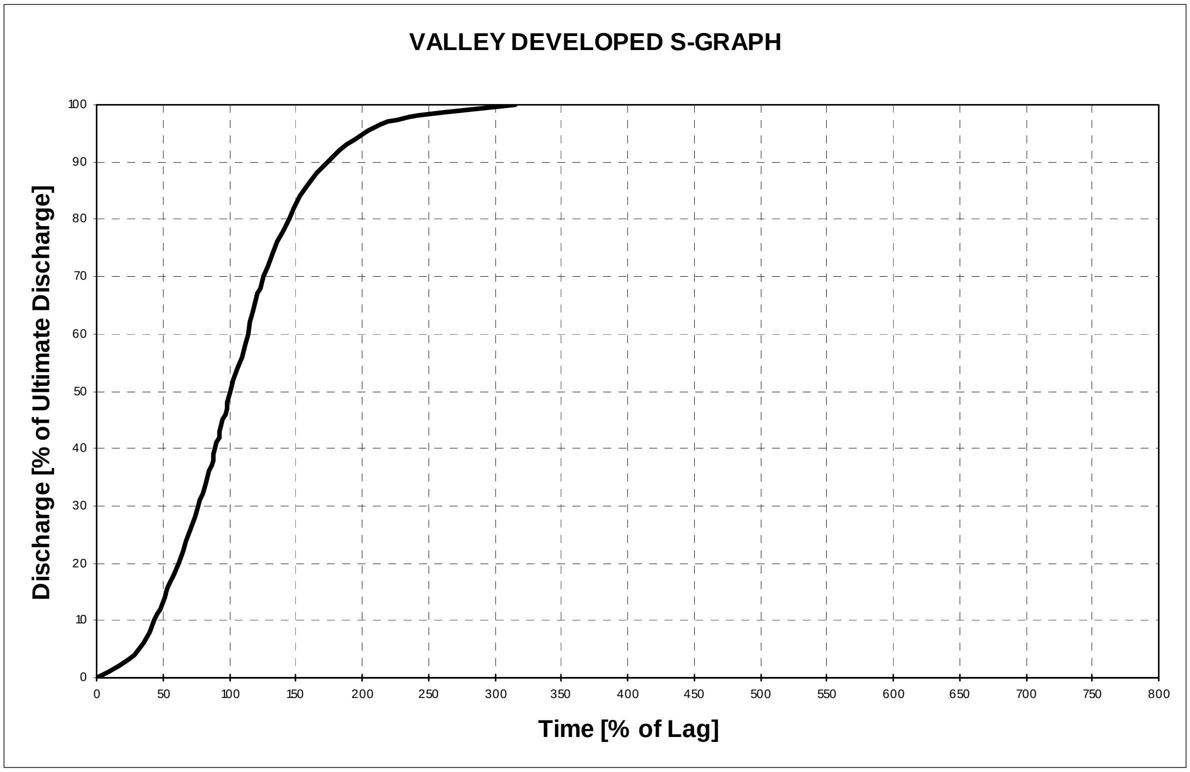


Figure 3- 5. San Joaquin County Valley Developed S-graph

Basin lag times were calculated according to guidelines set forth in the San Joaquin County Hydrology Manual[10]. The following equation was used:

$$Lg = 24n(L \cdot L_C/S^{0.50})^{0.38}$$

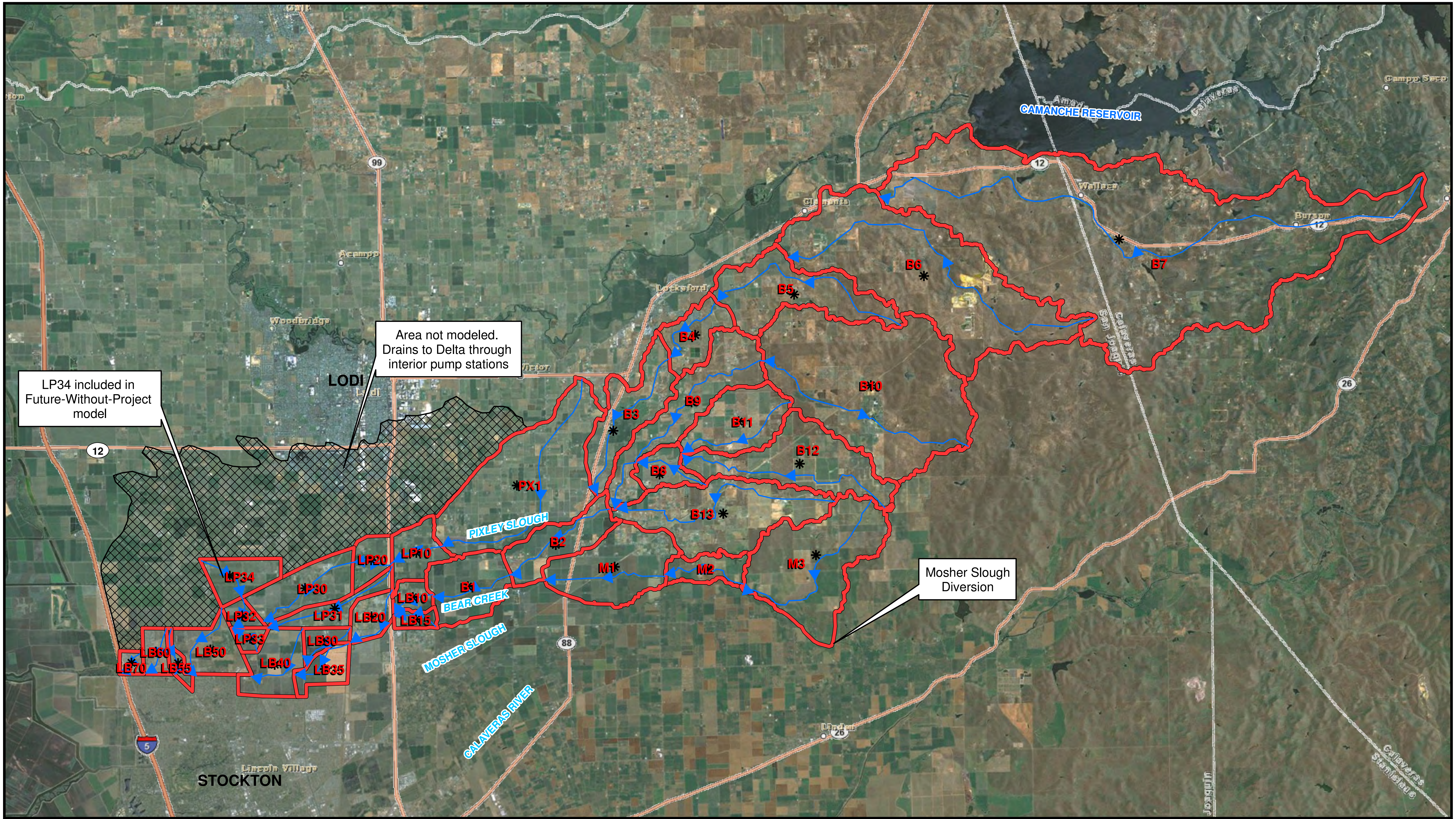
where:

Lg = Lag time [hours]
n = Average basin factor estimated using Figure E-2 in the San Joaquin County Hydrology Manual
L = Length of longest watercourse [miles]
L_C = Length of longest watercourse measured to the centroid of the basin [miles]
S = Overall slope of longest watercourse [feet/mile]

L, L_C, and S were calculated using ArcGIS software. Flowpaths identified for these calculations are shown in Figure 3- 6.

3.3.5. Channel Routing

The PBI Model utilizes the Muskingum-Cunge routing method to represent attenuation of flood waves within Bear Creek channels. Routing reach lengths and slopes were measured using ArcGIS software. Manning’s n values and channel cross-sections were imported from the 1998 SJAFCA HEC-1 model[8].



LP34 included in Future-Without-Project model

Area not modeled. Drains to Delta through interior pump stations

Mosher Slough Diversion

🟥 Subbasin Boundary
✱ Subbasin Centroid
▶ Subbasin Flowpath

N

0 0.5 1 2 Miles
1 inch = 2 miles

DECEMBER 8, 2011

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

BEAR CREEK SUBBASIN FLOWPATHS

FIGURE

3-6

The following table provides a summary of routing elements included in the PBI Model.

Table 3- 2. Summary of Bear Creek model routing elements.

			Manning's n		Description	
Routing Element	Length [ft]	Slope [ft/ft]	Main Channel	Overbank	From	To
RB7	18,670	0.0012	0.045	0.06	B7	B6
RN1222	10,190	0.0018	0.045	0.06	B6	B5
RN1210	10,010	0.0008	0.035	0.05	B5	B4
RN1209	22,300	0.0011	0.035	0.05	B4/B3	B9
RB10	27,690	0.0014	0.040	0.55	B10	B9
RTHDR	10,880	0.0015	0.035	0.05	B11/B12	B8
RN1208	1,860	0.0022	0.035	0.05	B8	B9
RN1204	13,060	0.0010	0.030	0.04	B9	B2
RMSRTN	9,820	0.0014	0.030	0.04	B2	B1
RN1203	5,260	0.0009	0.030	0.04	B1	LB15/MSDIV
RM3	11,890	0.0014	0.045	0.06	M3	M2
RNM2	14,430	0.0014	0.045	0.06	M2	M1
RN1202	980	0.0010	0.030	0.04	B1/LB15	LB10
R1020	6,530	0.0011	0.030	0.04	LB10	LB20
R2030	6,380	0.0014	0.030	0.04	LB20	LB30
R3035	1,810	0.0050	0.030	0.04	LB30	LB35
R3540	4,690	0.0009	0.030	0.04	LB35	LB40
R4050	7,080	0.0018	0.030	0.04	LB40	LB50
RPX1	7,160	0.0007	0.050	0.06	PX1	LP10
RP1020	5,470	0.0007	0.050	0.06	LP10	LP20
RP2030	13,860	0.0010	0.050	0.06	LP20	LP30
RP313	4,200	0.0012	0.050	0.06	LP30	LP32/LP33
RP325	8,370	0.0013	0.050	0.06	LP32/LP33	LB50
R5055	1,960	0.0010	0.030	0.04	LB50	LB55
R5560	6,510	0.0011	0.030	0.04	LB55	LB60/LB70

Twenty-five reaches covering a total of approximately 44 miles of the Bear Creek stream system are included in the PBI Model.

3.3.6. Loss Rates

As discussed in Section 3.2.1, the 1998 SJAFCA HEC-1 model used the SCS Curve Number method to calculate loss rates. The PBI Model differs from the SJAFCA HEC-1 model in that it uses the initial and constant loss rate method to model subbasin losses.

The Natural Resources Conservation Service (NRCS) has classified all soils into four hydrologic soil groups (A, B, C, and D) according to their infiltration rates[15]:

Table 3- 3. NRCS hydrologic soil groups.

Hydrologic Soil Group	Loss Rate Range [in/hr]	PBI's Assumed Loss Rate[a] [in/hr]
A	> 0.30	0.35
B	0.15 - 0.30	0.2
C	0.05 - 0.15	0.1
D	0.00 - 0.05	0.025

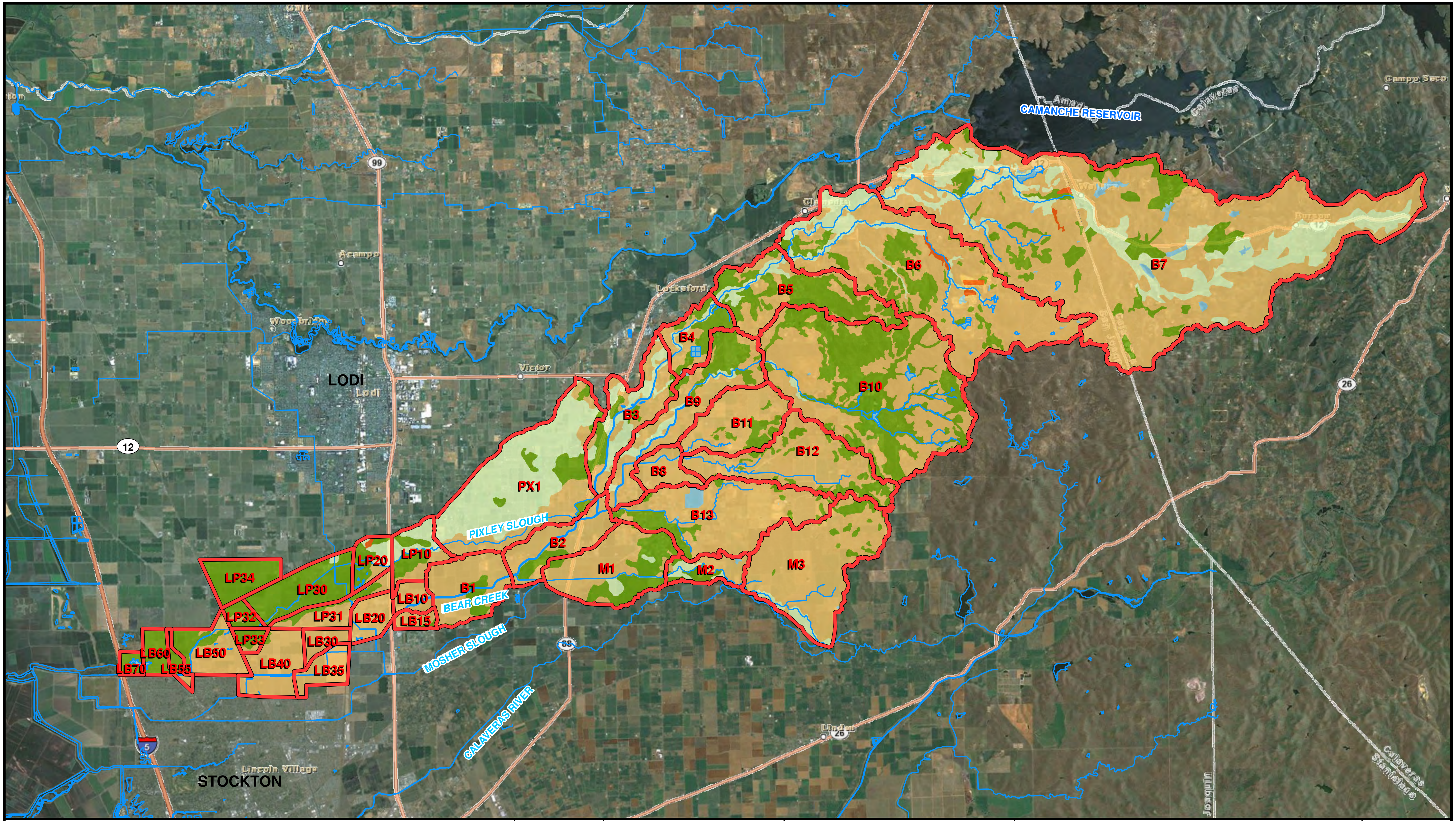
[a]This loss rate value was assigned to each soil group for initial calculations of composite loss rates. The calculated composite loss rates were then adjusted during the calibration process.

A GIS soils layer was obtained from the NRCS[13] and used to determine the proportional coverage of soil groups within Bear Creek subbasins (Figure 3- 7). NRCS GIS soils data was not available for Calaveras County. Soils data for this part of the study area was obtained from the Calaveras County Soil-Vegetation Survey[14]. A weighted average of loss rates was calculated for each subbasin and adjusted during the calibration process (See Section 3.4). After the calibration adjustment, subbasin loss rates range from 0.020 inches per hour to 0.118 inches per hour as shown in Attachment 3-C.

EM 1110-2-1417[18] recommends that initial losses are set between 0.5-1.5 inches for agricultural areas. Initial losses were set to 0.5 inches for all agricultural/rural subbasins in the foothills and to 1.5 inches for agricultural/rural subbasins in the valley. For urban subbasins, initial losses were set to 0.2 also based on guidelines listed in *EM 1110-2-1417*.

3.3.7. Impervious Percentages

Impervious percentages were assigned based on the extent of urbanization within each subbasin. Aerial photos including those contained within 2010 LiDAR datasets[4] were used to assess existing urbanization in the Bear Creek watershed. subbasins were classified into several categories with assigned impervious percentages as shown in Table 3- 4. The impervious percentages corresponding to each land use type were selected with the guidance of San Joaquin County's *Hydrology Manual*[10].



🟧 Group A

🟨 Group D

▢ Group B

🟦 Water/Other

🟩 Group C

N

0 0.5 1 2 Miles

1 inch = 2 miles

DECEMBER 8, 2011

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

FIGURE

3-7

BEAR CREEK SOILS MAP

Table 3- 4. Land use types and their corresponding impervious percentages.

Land Use Type	Impervious Percentage
Agricultural/Open Space	2%
Agricultural with Rural Residential Development	5%
Fully Developed Residential	60%

3.4. MODEL CALIBRATION

The 1998 SJAFCA HEC-1 model was calibrated using an annual exceedance probability plot at the Lockeford stream gage[8]. This plot was used to determine a 1/100 AEP flow event and it was assumed that a 1/100 AEP rainfall event would produce a 1/100 AEP streamflow event.

The PBI Model was calibrated to an observed rainfall-runoff event using gaged data retrieved from San Joaquin County's ALERT System[16]. Three gages were used for the model calibration. The Bear Creek streamflow gage (ALERT Gage 238) and Alpine Road rainfall gage (ALERT Gage 239) are both located on Bear Creek between Highway 99 and State Route 88 (see Figure 3- 2). In addition, the Robidart Ranch gage (ALERT Gage 237) provides rainfall data for the subbasins in the upper portion of Bear Creek watershed.

The storm selected to calibrate the PBI Model was the largest event recorded by the Bear Creek gage and is approximately a 1/10 AEP event. The rainfall event took place between January 29, 1998 and February 9, 1998 (12-day duration) and totaled 6.26 inches. The "effective" portion of the storm included 2.88 inches of rainfall falling in 32 hours and was responsible for the peak streamflow seen on February 3, 1998.

The Bear Creek gage location corresponds to Model Element MSRTN. During the calibration process, constant loss rates were adjusted to match the PBI Model's hydrograph at MSRTN to observed streamflow records from the Bear Creek gage. Constant loss rates were initially calculated based on the makeup of soils in each subbasin (see Section 3.3.6). The loss rates were then adjusted by a factor of 0.80 during the calibration process. The results of the calibration are shown in Figure 3- 8.

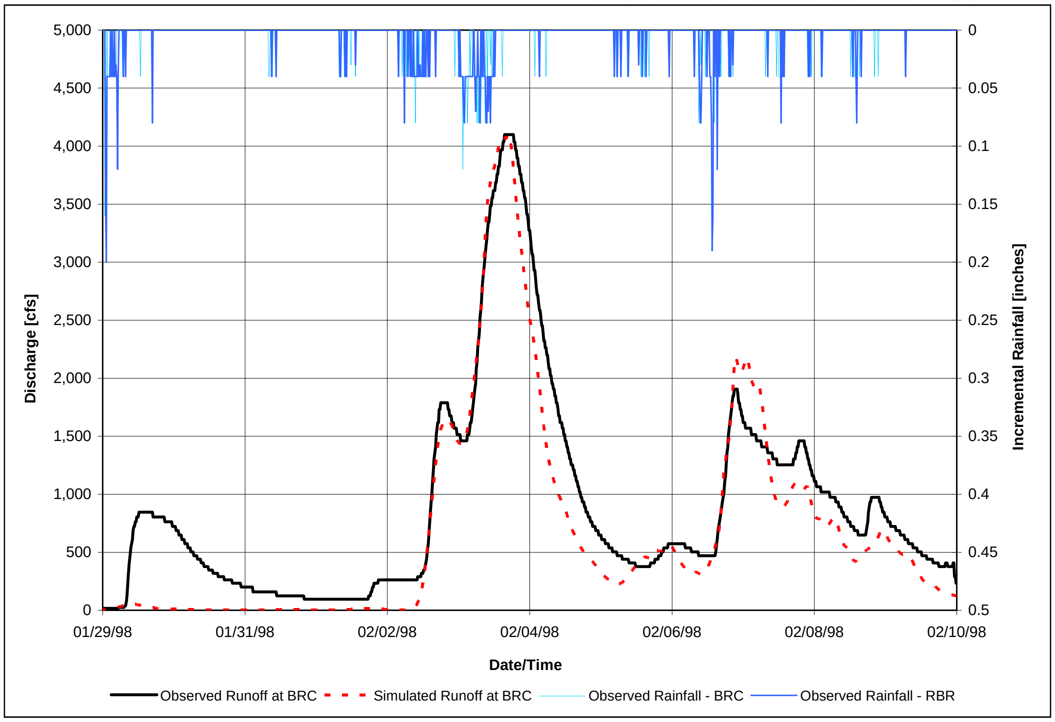


Figure 3- 8. Observed versus modeled flow for the Bear Creek calibration event.

At the onset of the storm, the initial runoff response is not picked up by the HEC-HMS model. This is due to the initial loss parameter being set to1.5 inches for the pervious areas of all subbasins (see Section 3.3.6). The subbasins upstream of the BRC gage are undeveloped and contain almost entirely pervious surfaces which are affected by the initial loss parameter. Although this runoff response could be captured by decreasing the initial losses, initial loss was held at 1.5 inches based on the ranges suggested in the Comp Study[9] and the variability in tilling practices, which have a major impact on initial losses. The emphasis of the hydrologic analysis is on peak event estimation, however, which is relatively insensitive to initial loss assumptions.

3.5. DEVELOPMENT CONDITIONS

3.5.1. Existing Conditions

An 'Existing Conditions' model run was performed to evaluate peak flows given current (2010) land use and hydrologic conditions within the Bear Creek watershed. Subbasin S-graphs, 'n' values, and impervious percentages were set according to current land cover conditions using field knowledge supplemented by aerial photos.

In general, the upstream watershed consists of natural or agricultural land whereas the lower portions of Bear Creek watershed are developed areas in and around the city of Stockton. A summary table of the subbasin characteristics used for 'Existing Conditions' model runs is provided in Attachment 3-D.

As seen in Figure 3- 9, subbasins LB50, LB55, LB60, LB70, and LP33 are considered to be developed and flows from these basins are directed through three storm water pump stations: the Spanos Park-I-5 pump station (PLB6070), the Thornton pump station (PLB5055), and the Pixley pump station (PLP33). The pump stations discharge flows up to their design capacities (see Section 3.3.2) into Bear Creek and Pixley Slough. Any subbasin flows exceeding pump station capacities would result in temporary ponding within the subbasin. This ponding would be entirely due to inadequate pump capacities and would be independent of exterior stage conditions in the receiving stream.

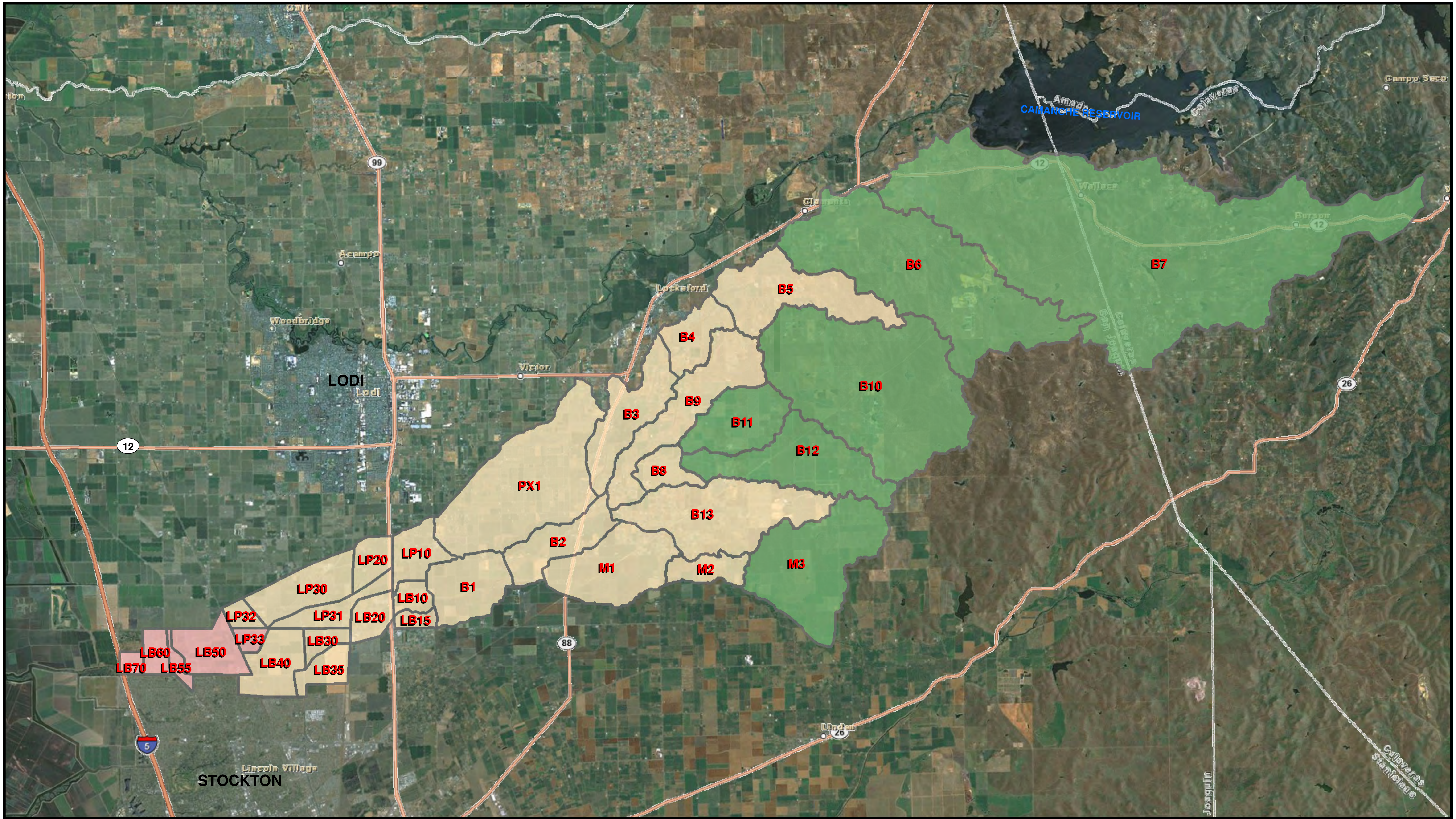
3.5.2. Future-Without-Project Conditions

A ‘Future Conditions’ model run was performed to evaluate peak flows for estimated future (2070) land use and hydrologic conditions within the Bear Creek watershed. Land use conditions are based on the City of Stockton 2035 General Plan[12] and the San Joaquin County General Plan[17].

As shown in Figure 3- 10, the upstream watershed remains unchanged and consists of natural or agricultural land whereas the lower portions of Bear Creek watershed experience an increase in development. The following 9 subbasins were previously undeveloped in the ‘Existing Conditions’ model and would be developed for the ‘Future-Without-Project Conditions’ model: LB10, LB15, LB20, LB30, LB35, LB40, LP30, LP31, LP32. As previously mentioned, subbasin LP34 was added to the ‘Future-Without-Project’ model to account for added drainage area that is expected to be pumped into Bear Creek.

In addition to updating subbasin S-graphs, ‘n’ values, and impervious percentages for the newly developed areas, storm water pump stations were also added to these subbasins. As previously mentioned, flows exceeding pump station capacities would cause temporary ponding, which was assumed to be mitigated within the subbasin through on-site detention.

A summary table of subbasin characteristics used for ‘Future-Without-Project Conditions’ model runs is provided in Attachment 3-E.



🟩 Foothill

🟨 Valley Undeveloped

🟥 Valley Developed

N

0 0.5 1 2 Miles
1 inch = 2 miles

DECEMBER 8, 2011

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

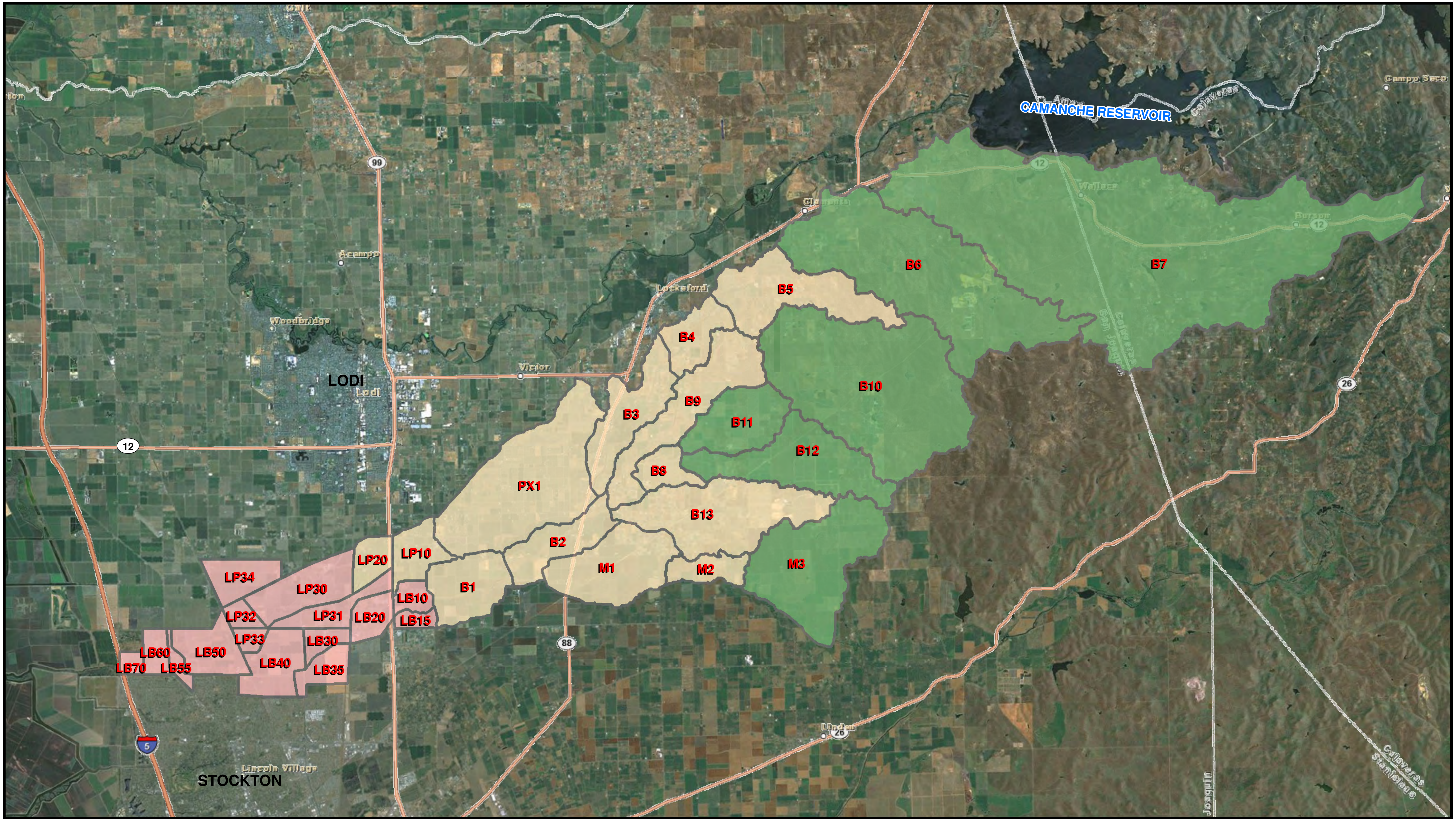
1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

EXISTING DEVELOPMENT CONDITIONS FOR BEAR CREEK WATERSHED

FIGURE
3-9



🟥 Valley Developed

🟨 Valley Undeveloped

🟩 Foothill

N

0 0.5 1 2 Miles
1 inch = 2 miles

DECEMBER 8, 2011

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING
1180 Iron Point Rd., Suite 260
Folsom, CA 95630
Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

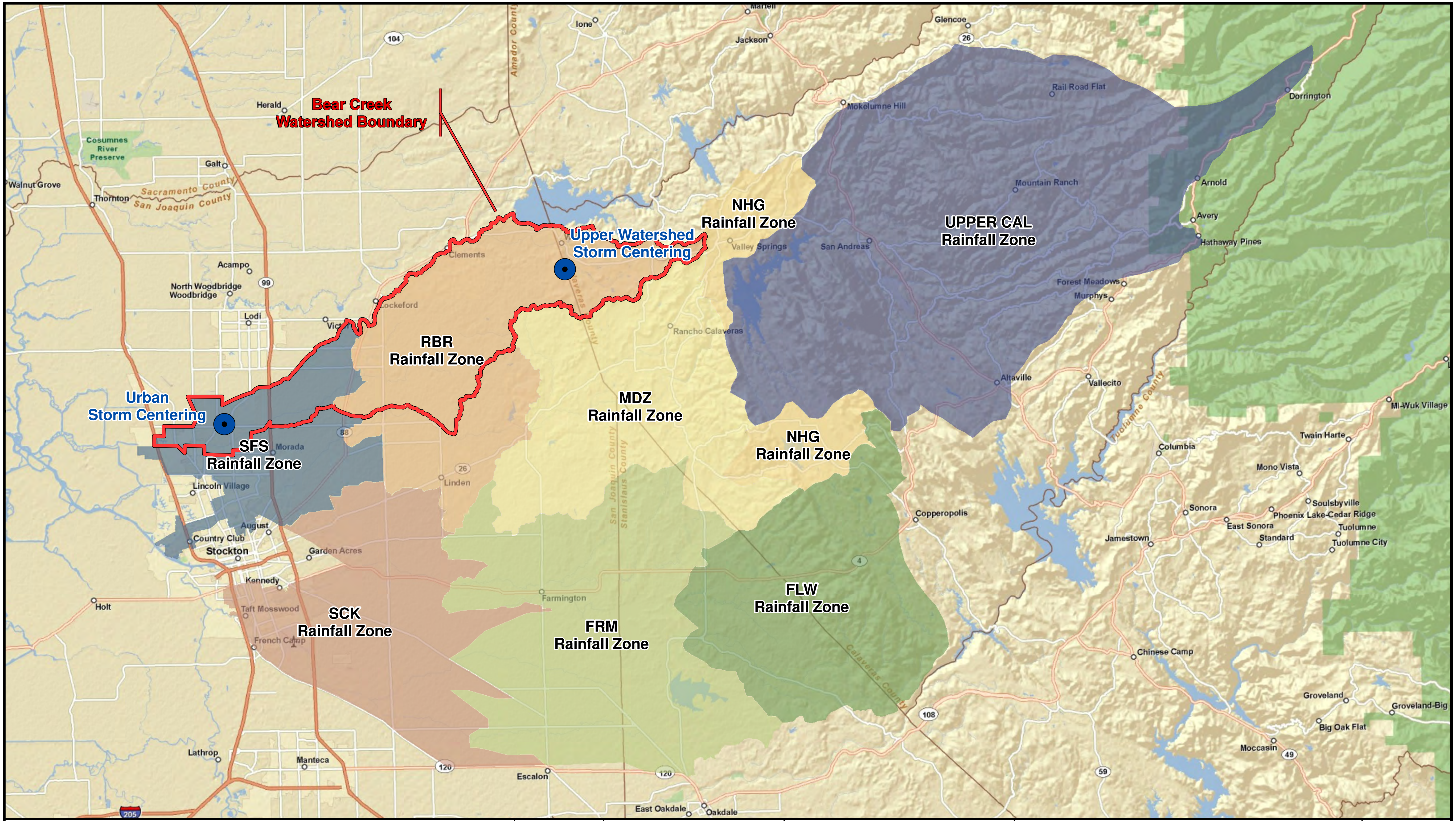
FUTURE DEVELOPMENT CONDITIONS FOR BEAR CREEK WATERSHED

FIGURE
3-10

3.6. STORM CENTERINGS

Two storm centerings were analyzed for the Bear Creek watershed (Figure 3- 11). One centering was placed over the upper portion of the watershed to create high flows in the tributary channels and concurrent inputs to the lower channel. The second centering was for interior drainage purposes and was placed over the urban areas of the watershed. The 8 AEP storm frequencies were analyzed for each centering. This selection of design storms provides a wide range of scenarios that can be used for planning purposes.

Calculated area reduction factors and resulting area-reduced rainfall depths for each rainfall zone are provided in Attachment 3-F for all frequency-duration-storm centering combinations.



N

0 5 Miles
1 : 300,000

DECEMBER 8, 2011

Peterson . Brustad . Inc
Engineering . Consulting

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

BEAR CREEK WATERSHED STORM CENTERINGS

FIGURE

3-11

3.7. MODEL SIMULATIONS

Bear Creek production runs include 32 scenarios with unique combinations of development conditions, storm frequencies, and storm centerings.

Table 3- 5. Bear Creek production run scenarios.

Development Conditions	Storm Centerings	AEP Events
Existing Conditions	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Upper Watershed	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
Future-Without-Project Conditions (2070)	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Upper Watershed	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500

3.7.1. Summary of Results

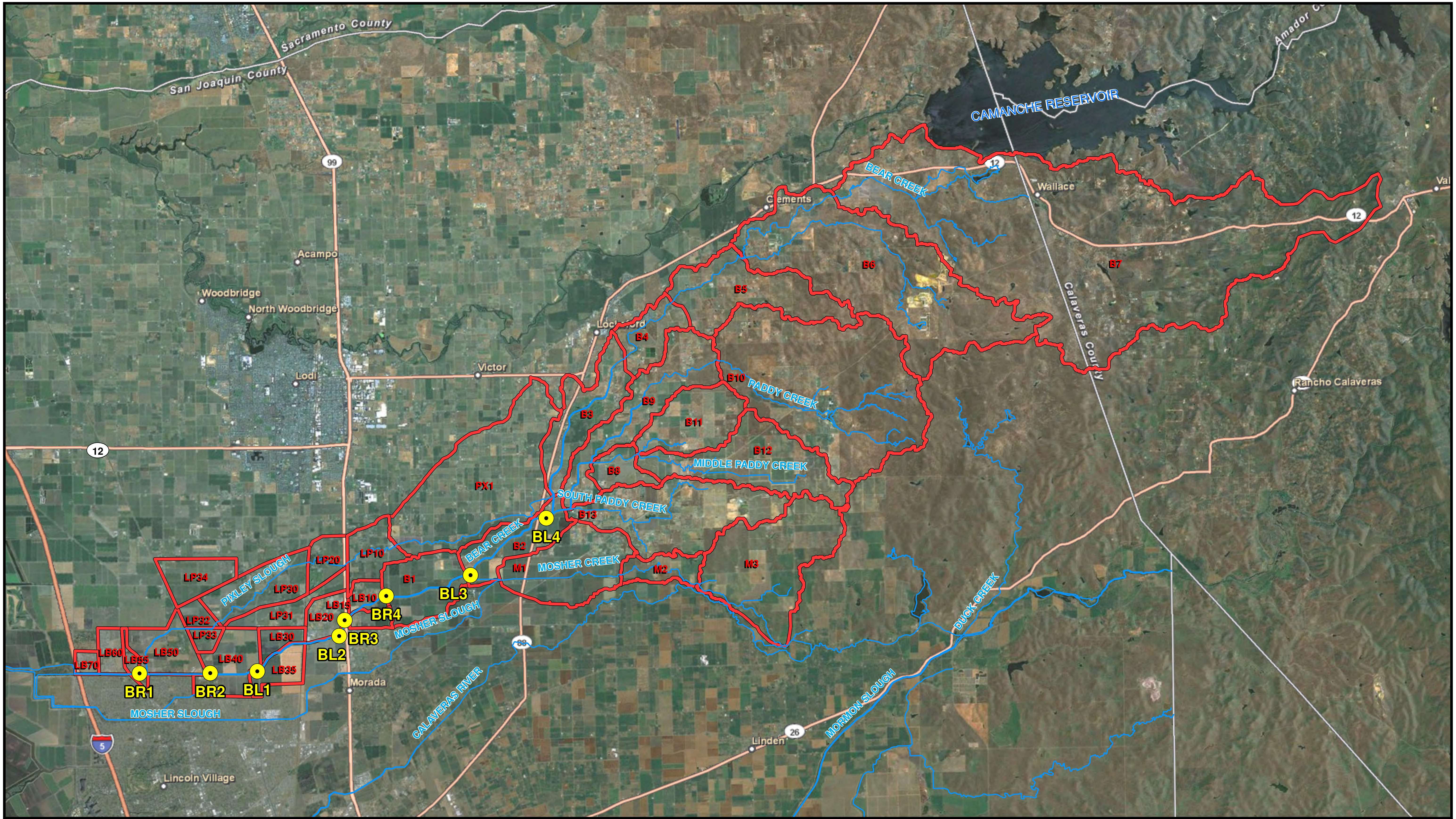
Peak flow results were extracted from HEC-HMS at each LSJRFS index point. Locations of LSJRFS index points within the Bear Creek watershed are shown in Figure 3- 12. Table 3- 6 and Table 3- 7 summarize peak flows for 'Existing Conditions' runs and for 'Future-Without-Project Conditions' runs, respectively.

In all cases, peak flows from the Upper Watershed storm centering scenario are higher than the urban storm centering scenario. The Upper Watershed centering is therefore the controlling scenario for the LSJRFS.

3.7.2. Uncertainty Parameters

For the purposes of the LSJRFS, uncertainty parameters for each flow-frequency dataset can be estimated within HEC-FDA during the project's economic analysis. HEC-FDA defines uncertainty in terms of confidence intervals or standard deviations given inputs of flow-frequency data (provided in Table 3- 6 and Table 3- 7) and an equivalent record length.

The equivalent record length is an estimate of the overall "worth" or "quality" of the flow-frequency function, expressed as the number of years-of-record[19]. For probability functions derived at ungaged locations using model or other data, the equivalent record length is based on a judgment of the quality of that model or data. EM 1110-2-1619[20] provides guidelines for assigning equivalent record lengths and estimates that a rainfall-runoff model calibrated to an observed event at a short-interval runoff gage has an equivalent record length of 20-30 years.



⊙ LSJRFS Index Point

▭ Subshed Boundary

N

0 0.5 1 2 Miles

1 inch = 2 miles

JUNE 20, 2012

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

BEAR CREEK WATERSHED INDEX POINTS

FIGURE

3-12

Table 3-6. Peak Flow Results for Bear Creek - Existing Conditions [cfs]

LSJRFS Index Point ID	Description	Urban Storm Centering								Upper Watershed Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP	1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
BL4	Bear Creek near Hwy 88	1,520	2,290	2,850	3,630	4,250	4,900	5,520	6,510	1,900	2,680	3,300	4,180	4,890	5,560	6,320	7,410
BL3	Bear Creek at Alpine Rd.	1,660	2,510	3,150	4,110	4,940	5,790	6,650	7,850	2,060	2,940	3,630	4,810	5,710	6,620	7,570	8,880
BR4	Bear Creek near CCTRR	1,670	2,540	3,190	4,190	5,030	5,890	6,760	7,990	2,060	2,940	3,670	4,850	5,770	6,680	7,650	8,970
BR3	Bear Creek at Hwy 99	1,670	2,540	3,190	4,230	5,070	5,930	6,810	8,040	2,060	2,940	3,690	4,870	5,790	6,700	7,670	9,000
BL2	Bear Creek d/s of Eight Mile Rd.	1,670	2,550	3,200	4,270	5,110	5,980	6,850	8,090	2,050	2,940	3,700	4,900	5,810	6,730	7,700	9,030
BL1	Bear Creek near West Ln.	1,680	2,570	3,250	4,340	5,200	6,070	6,960	8,200	2,050	2,950	3,740	4,950	5,870	6,800	7,780	9,110
BR2	Bear Creek at UPRR	1,690	2,580	3,310	4,430	5,300	6,190	7,080	8,340	2,050	2,960	3,790	5,020	5,940	6,880	7,870	9,210
BR1	Bear Creek d/s of Pixley Slough confl.	1,720	2,670	3,520	4,810	5,810	6,800	7,800	9,190	2,080	2,990	3,840	5,180	6,200	7,240	8,340	9,820
D2	Bear Creek at I-5	1,760	2,710	3,600	4,900	5,920	6,960	7,990	9,430	2,110	3,020	3,890	5,270	6,340	7,400	8,490	10,000

Table 3-7. Peak Flow Results for Bear Creek - Future Conditions [cfs]

LSJRFS Index Point ID	Description	Urban Storm Centering								Upper Watershed Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP	1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
BL4	Bear Creek near Hwy 88	1,520	2,290	2,850	3,630	4,250	4,900	5,520	6,510	1,900	2,680	3,300	4,180	4,890	5,560	6,320	7,410
BL3	Bear Creek at Alpine Rd.	1,660	2,510	3,150	4,110	4,940	5,790	6,650	7,850	2,060	2,940	3,630	4,810	5,710	6,620	7,570	8,880
BR4	Bear Creek near CCTRR	1,670	2,540	3,190	4,190	5,030	5,890	6,760	7,990	2,060	2,940	3,670	4,850	5,770	6,680	7,650	8,970
BR3	Bear Creek at Hwy 99	1,680	2,550	3,200	4,250	5,090	5,950	6,810	8,070	2,070	2,960	3,710	4,890	5,820	6,730	7,700	9,010
BL2	Bear Creek d/s of Eight Mile Rd.	1,690	2,590	3,260	4,340	5,180	5,980	6,910	8,160	2,070	2,970	3,740	4,920	5,860	6,790	7,790	9,100
BL1	Bear Creek near West Ln.	1,700	2,590	3,300	4,430	5,250	6,080	7,020	8,280	2,080	2,980	3,790	5,000	5,920	6,900	7,870	9,230
BR2	Bear Creek at UPRR	1,740	2,630	3,320	4,540	5,390	6,210	7,230	8,470	2,110	3,020	3,840	5,050	6,070	7,030	7,960	9,380
BR1	Bear Creek d/s of Pixley Slough confl.	1,910	2,790	3,780	5,060	6,320	7,260	8,210	9,460	2,170	3,070	4,050	5,470	6,600	7,750	8,810	10,410
D2	Bear Creek at I-5	2,000	2,830	3,840	5,210	6,440	7,440	8,350	9,710	2,200	3,100	4,140	5,600	6,730	7,910	8,990	10,560

4.0 MOSHER SLOUGH HEC-HMS MODELING

4.1. GENERAL

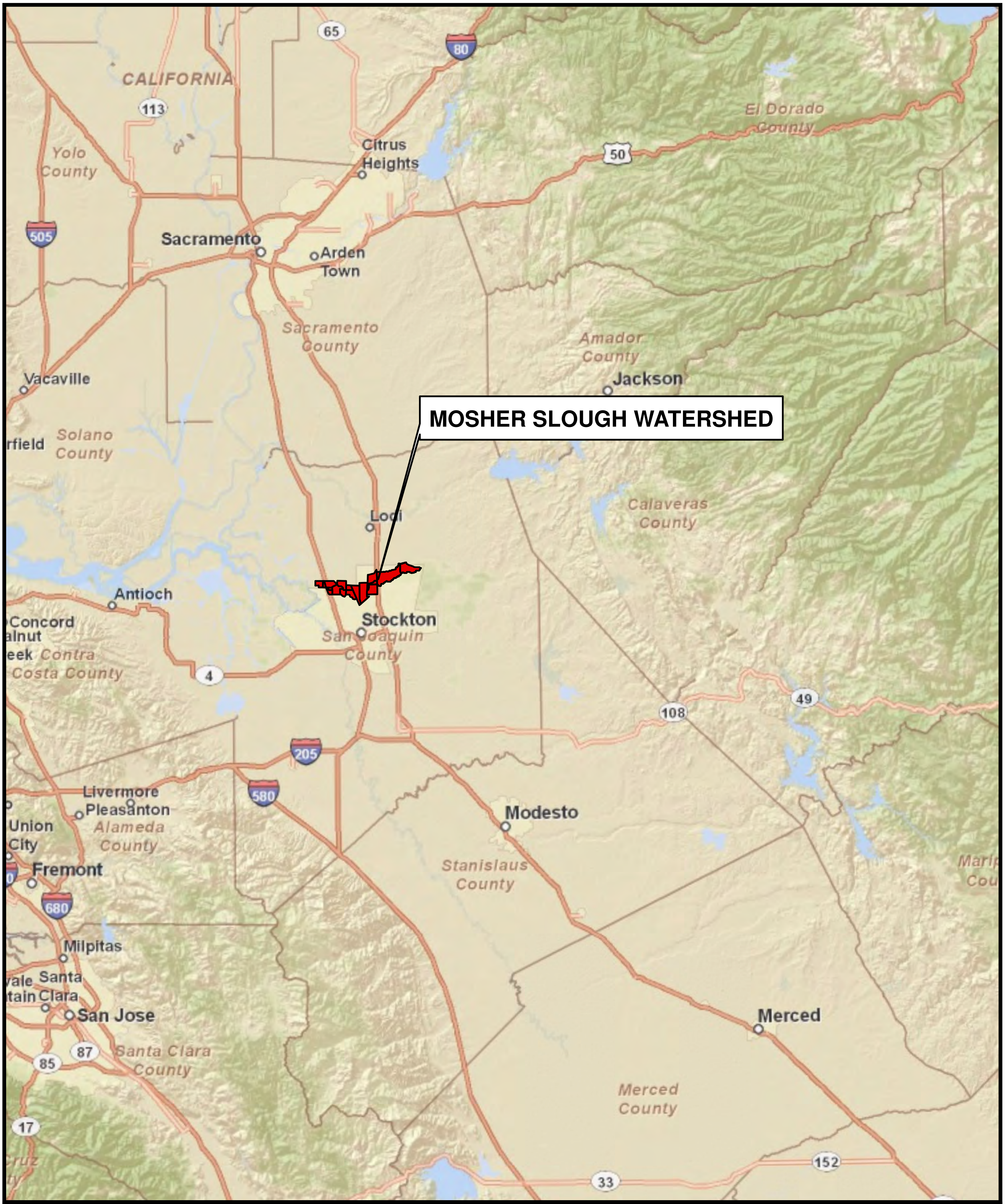
4.1.1. Location

Mosher Slough is located near the city of Stockton in San Joaquin County, California (Figure 4- 1). The majority of the watershed is located in the urbanized area of Stockton between Interstate-5 and Highway 99 with the watershed area totaling approximately 16 square miles. The watershed's terrain has moderate slopes and reaches a maximum elevation of 65 feet above the modeled outlet at the confluence of Mosher Slough and Bear Creek just west of Interstate-5.

The HEC-HMS model described in this report includes only the lower portion of Mosher Slough which begins immediately below the diversion that routes the entirety of Upper Mosher Creek to Bear Creek (see Figure 4- 2). The hydrology for Upper Mosher Creek is included in the Bear Creek HEC-HMS model as described in Section 3.0 of the LSJRFS Hydrology Report.

4.1.2. Topography

The HEC-HMS model utilized for this study is titled the PBI Mosher Slough Model (PBI Model) which is georeferenced to the NAD 1983 State Plane California Coordinate System Zone III (U.S. Survey Feet). Vertical elevations are reported in the NAVD 1988 datum. Topography used for model development included United States Geological Survey (USGS) 30-meter Digital Elevation Models (DEMs)[3]. Department of Water Resources (DWR) LiDAR data[4] was also used to confirm subbasin boundaries (State vertical datum in NAVD 88).



PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

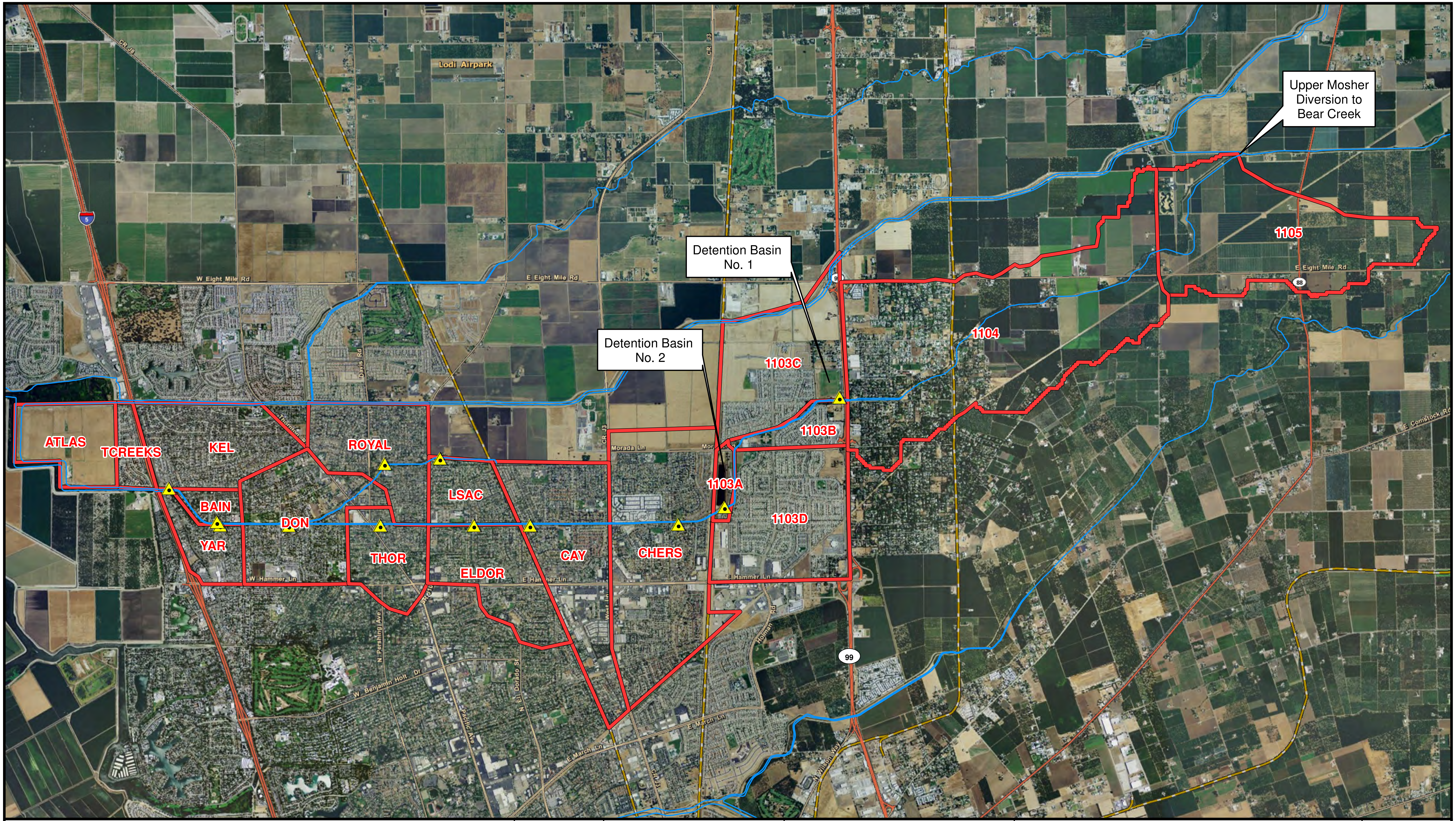
Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

VICINITY MAP OF THE MOSHER SLOUGH STUDY AREA

FIGURE

4-1



🟥 Subbasin Boundary

△ Existing Pump Station

N

0 1,000 2,000 4,000 Feet

1 inch = 4,000 feet

AUGUST 20, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

MOSHER SLOUGH
HEC-HMS SUBBASINS

FIGURE

4-2

4.2. MODEL DEVELOPMENT

The PBI model was developed by converting the 1998 SJAFCA HEC-1 into HEC-HMS format using HEC-HMS version 3.4[5] and HEC-GeoHMS version 4.2[6]. A summary of the tasks performed are listed below:

1. The 1998 SJAFCA HEC-1 model was imported into HEC-HMS (See Section 4.2.2).

2. Subbasin boundaries from SJAFCA HEC-1 model were updated using HEC-GeoHMS and United States Geological Survey (USGS) Digital Elevation Models (DEMs) (See Section 4.3.1).

3. Pump stations were coded into the PBI model based on design pumping rates provided by the City of Stockton[7] (See Section 4.3.2).

4. New diversions and channel routing parameters were coded into the PBI Model (See Sections 4.3.3 and 4.3.5, respectively), replacing those used in the SJAFCA HEC-1 model.

5. New loss rates and impervious percentages were coded into the PBI Model (See Section 4.3.6 and Section 4.3.7) replacing those used in the SJAFCA HEC-1 model.

6. S-graphs and lag times were assigned to each subbasin (See Section 4.3.4).

7. The PBI Model was set up to simulate both Existing (Section 4.5.1) and Future-Without-Project (Section 4.5.2) scenario runs.

4.2.1. SJAFCA HEC-1 Model

The PBI Model is a conversion and update of the HEC-1 model developed for SJAFCA by HDR Engineering, Inc. in 1998[8].

The 1998 SJAFCA HEC-1 model utilized the San Joaquin County LA preprocessor to convert S-graphs to unit hydrographs for each subbasin. Two types of S-graphs were obtained from the San Joaquin County Hydrology Manual[10] and used based on the surface condition classification of the subbasin: Valley Undeveloped and Valley Developed. Lag times were calculated by HDR using basin ‘n’ values, length of subbasin flow, flow length from the centroid, and slope of the basin.

The SJAFCA HEC-1 model used the SCS curve number method to account for subbasin losses. Curve numbers typically ranged from 81 to 86 depending on soil type and cover. Attachment 4-A lists the parameters used in the 1998 SJAFCA HEC-1 model.

4.2.2. Conversion from HEC-1 to HEC-HMS

The 1998 SJAFCA HEC-1 model was successfully imported into HEC-HMS as the fundamental basis for the PBI Model.

Certain features in the HEC-1 software are not supported in HEC-HMS and therefore were not properly transferred during the import process. Pump station data and meteorological data from the SJAFCA HEC-1 model were manually coded into the PBI Model so as to conform to HEC-HMS formatting.

In addition, there are computational differences between the HEC-1 and the HEC-HMS software. One such difference involves the Muskingum-Cunge stream segment routing technique used for the PBI model. In HEC-HMS, channel properties are computed based on the physical characteristics of that channel, whereas in HEC-1 the properties are computed with formulas based on a kinematic wave assumption[5]. This causes minor differences in the flows that are transferred through the routing parameters. HEC-HMS results are preferred because of the refined computational techniques that have been implemented.

For initial PBI Model testing, user-specified hyetographs were assigned to each subbasin based on 1/100 AEP storm data defined in the 1998 SJAFCA HEC-1 model's input files. This storm event was run for debugging purposes and results were made sure to match the SJAFCA HEC-1 model results.

4.3. MODEL FEATURES

The 1998 SJAFCA HEC-1 model was converted and modified for this study to form the PBI Mosher Slough HEC-HMS Model. The PBI Model components are described in the following sections.

4.3.1. Subbasins

Subbasin boundaries used in the 1998 SJAFCA HEC-1 model were cross-checked with USGS 30-meter DEM datasets[3] and modified where appropriate. Any boundary modifications were made using the ArcHydro and HEC-GeoHMS[6] extensions within the ArcGIS software package. These tools utilize geospatial data to interpret drainage patterns and delineate watershed boundaries accordingly. Where available, DWR LiDAR[4] data was used to confirm subbasin boundaries. For the majority of the watershed, west of Highway 99, subbasin boundaries were based on the City of Stockton's *Conceptual Storm Drain Master Plan*[11]. This portion of the watershed is urbanized and the boundaries from the City of Stockton take into account drainage improvements that have been made in the area.

The PBI model contains two additional subbasins when compared to the 1998 SJAFCA HEC-1 Model. The Twin Creeks and Atlas tracts, totaling 0.68 square miles, drain to Mosher Slough and are located just west of Interstate-5. Mosher Slough subbasins included in the PBI Model are shown in Figure 4- 2.

The PBI Model contains a total of 18 subbasins with drainage areas ranging from 0.17 square miles to 3.25 square miles with a total watershed area of approximately 16 square miles.

The GIS horizontal coordinates for each subbasin were used to georeference model elements within the PBI HEC-HMS Model. The subbasin GIS shapefile was inserted into the PBI Model as a background map.

4.3.2. Detention Basins and Pump Stations

The Mosher Slough system includes two detention basins that are intended to help reduce peak flows. 'Detention Basin No. 1' is located just west of Highway 99 on the north side of the main channel. It is connected to the main channel through a lateral weir that induces split flow for channel flows in excess of 230 cfs, with overflows into Detention Basin No. 1. The detained flows are held until the storm peak passes and then pumped back into Mosher Slough. Any inflow that causes Detention Basin No. 1 to exceed its 160 AF capacity is redirected back into the main channel. This is accomplished in HEC-HMS by connecting the main channel to a diversion element which directs any flow in excess of 230 cfs to a reservoir element. This reservoir element represents the detention pond and is coded with a spillway to take any flow exceeding the 160 AF pond capacity and spill it back into the main channel. In subsequent LSJRFS tasks, HEC-RAS runs will better model flow split hydraulics.

Detention Basin No. 2 and pump station are located just upstream from the formerly named Southern Pacific Railroad. This detention basin collects all runoff from subbasins 1103A, 1103B, 1103C, and 1103D. The pump station pumps runoff stored in Detention Basin No. 2 and includes one pump at 10 cfs and an additional three pumps at 25.1 cfs each. During a flow event at or exceeding the 1/100 AEP, however, only the 10 cfs pump is activated while the other three pumps are not utilized until the event has subsided. Any flow that causes the detention basin to exceed its 265 AF capacity will cause a temporary backup of the storm sewer system until the 25.1 cfs pumps activate and drain the pond after the storm peak passes.

Along with the pumps at Detention Basin No. 2, eleven additional pump stations were included in the 'Existing Conditions' PBI Model to represent storm drainage conveyance from developed subbasins to Mosher Slough. Multiple pumps are included at each pump station with capacities assigned based on City of Stockton records. All pumps are set to discharge over the top of the levees and into the receiving channel above the highest stage expected. The exterior and interior areas at the pump stations are independent from one another.

One pump station was added into the 'Future-Without-Project Conditions' model for the Atlas Tract subbasin which is just downstream of I-5. This area is expected to become developed according to the City of Stockton 2035 General Plan[12]. Pump capacity was assigned at a rate of 0.37 cfs per acre of tributary area. This rate is based on the average flow rates of existing pump stations within the City of Stockton's systems.

Table 4- 1 provides a summary of pump stations included in the PBI Model.

Table 4- 1. Summary of Mosher Slough pump stations.

Pump Station Name	Contributing Subbasin Area [Sq. Mi.]	Pump Station Status	Pump Station Capacity [cfs]	Pump Station Notes
Cherbourg	1.78	Existing	199.5	1 @ 9 cfs 3 @ 63.5 cfs
Cayuga	1.17	Existing	269.2	4 @ 67.3 cfs
El Dorado	0.71	Existing	188.5	4 @ 46 cfs 1 @ 4.5 cfs
Thornton	0.47	Existing	26.8	2 @ 13.4 cfs
Lower Sacramento Rd.	0.35	Existing	19.0	1 @ 13.4 cfs 1 @ 5.6 cfs
Royal Oaks	0.73	Existing	204.5	1 @ 6.7 cfs 1 @ 44.6 cfs 2 @ 76.6 cfs
Don Avenue	0.96	Existing	77.7	1 @ 66.8 cfs 1 @ 10.9 cfs
Yarmouth	0.30	Existing	82.1	1 @ 7.8 cfs 1 @ 74.3 cfs
Bainbridge	0.14	Existing	43.5	3 @ 13.4 cfs 1 @ 3.3 cfs
Kelly	0.79	Existing	152.6	1 @ 8.9 cfs 1 @ 47.9 cfs 1 @ 45.7 cfs 1 @ 50.1 cfs
La Morada (Detention Basin No. 2)	8.30	Existing	85.3	1 @ 10 cfs 3 @ 25.1 cfs
Twin Brooks at Twin Creeks	0.17	Existing	34.8	3 @ 11.6 cfs
Atlas	0.51	Future	120.8	Based on 0.37 cfs per acre

4.3.3. Diversions

There is one diversion included in the PBI Mosher Slough model used to represent the lateral weir that diverts excess flows to ‘Detention Pond No.1’ located just west of Highway 99. This weir allows flows exceeding 230 cfs to overflow into the basin thereby regulating flows coming from upstream subbasins 1104 and 1105.

All flows from upper Mosher Creek are diverted to the main stem of Bear Creek at a location just upstream of the Central California Traction Railroad. Because this structure diverts all flow, Upper Mosher Creek was coded as a tributary area to Bear Creek and included in the PBI Bear Creek HEC-HMS model. The diversion was originally constructed by the United

States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS), and improved by SJAFCA in 1998.

4.3.4. S-graphs and Lag Times

As discussed in Section 4.2.1, the 1998 SJAFCA HEC-1 model utilized the San Joaquin County LA preprocessor for converting S-graphs to unit hydrographs. The PBI Model assigns Valley Undeveloped and Valley Developed S-graphs directly into HEC-HMS for each subbasin based on its location. S-graph data points were obtained from the San Joaquin County Hydrology Manual[10]. The S-graphs were developed based on rainfall-runoff data from Southern California catchments considered to be hydrologically similar to the local catchments.

Figure 4- 3and Figure 4- 4 show the time versus discharge relationship for each S-graph.

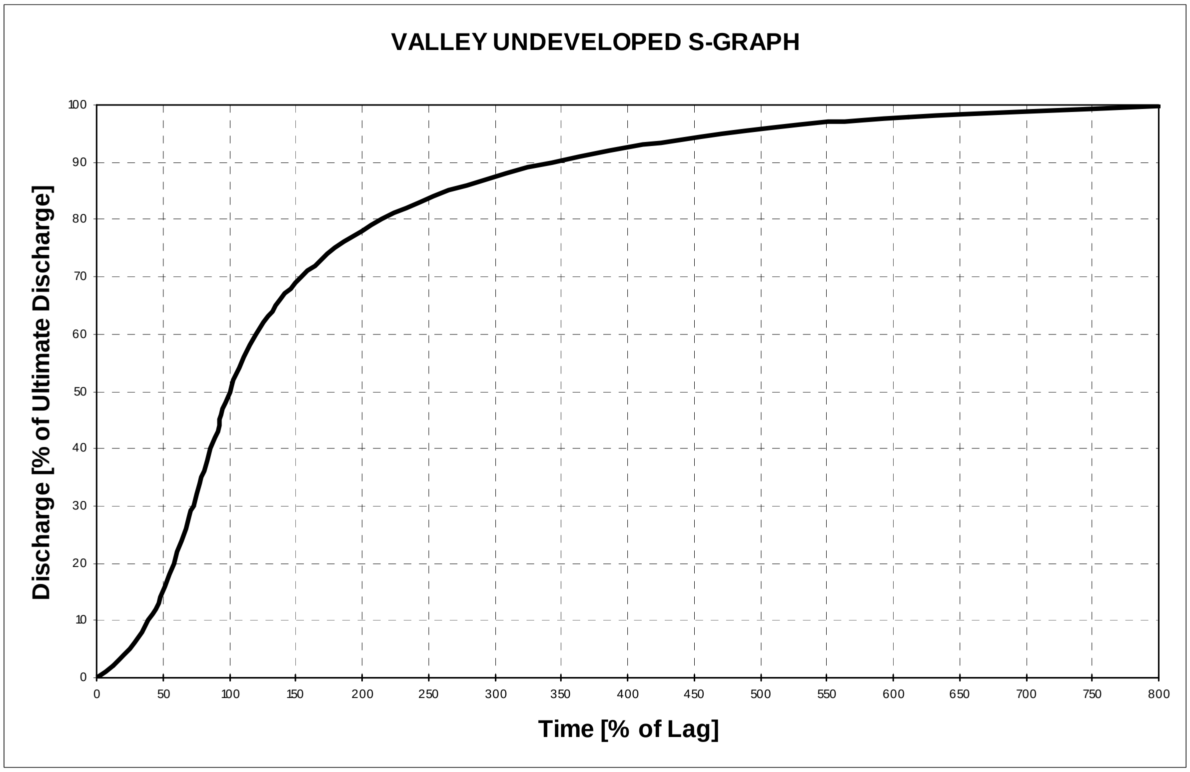


Figure 4- 3. San Joaquin County Valley Undeveloped S-graph

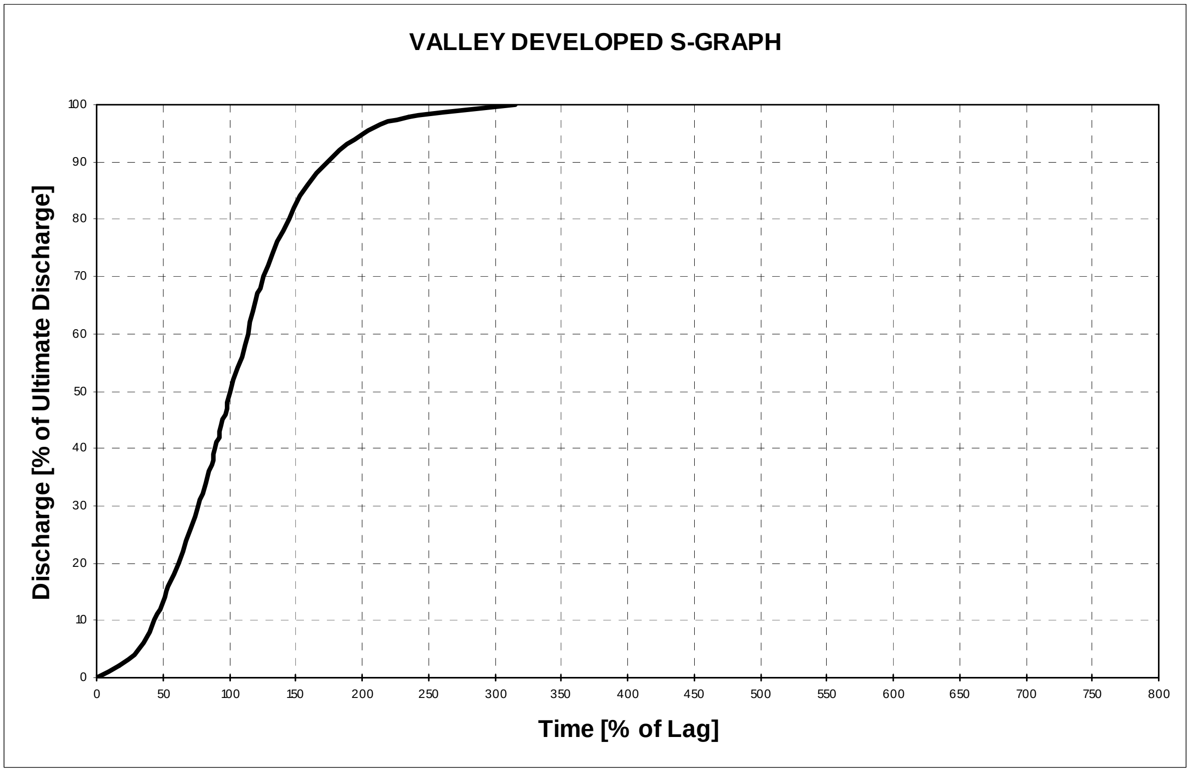


Figure 4- 4. San Joaquin County Valley Developed S-graph

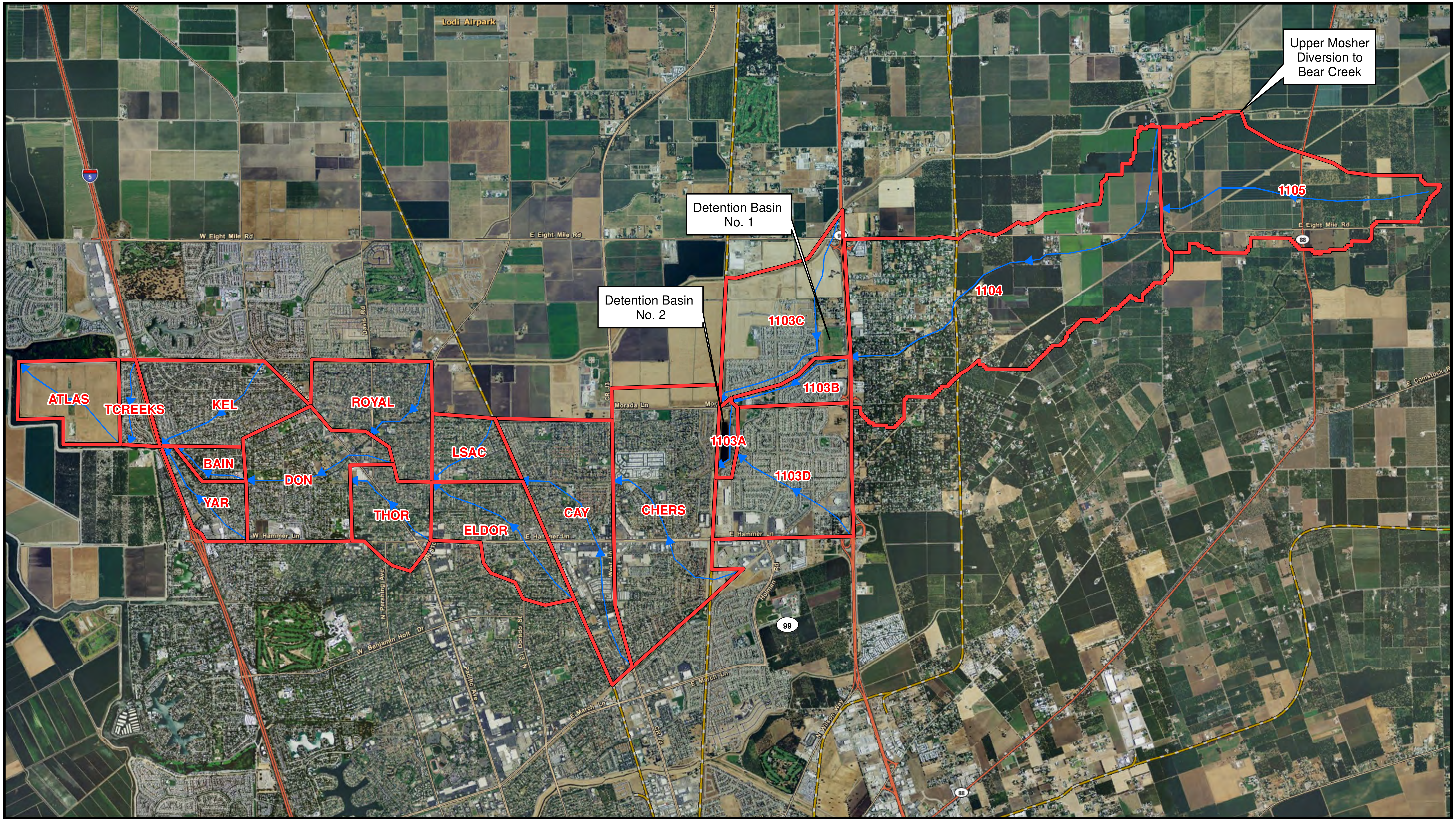
Basin lag times were calculated according to guidelines set forth in the San Joaquin County Hydrology Manual[10]. The following equation was used:

$$Lg = 24n(L\cdot L_C/S^{0.50})^{0.38}$$

where:

Lg = Lag time [hours]
n = Average basin factor estimated using Figure E-2 in the San Joaquin County Hydrology Manual
L = Length of longest watercourse [miles]
L_C = Length of longest watercourse measured to the centroid of the basin [miles]
S = Overall slope of longest watercourse [feet/mile]

L, L_C, and S were calculated using ArcGIS software. Flowpaths identified for these calculations are shown in Figure 4- 5.



Upper Mosher Diversion to Bear Creek

Detention Basin No. 1

Detention Basin No. 2

🟥 Subbasin Boundary

✳ Subbasin Centroid

→ Subbasin Flowpath

N

0 1,000 2,000 4,000 Feet

1 inch = 4,000 feet

AUGUST 20, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

MOSHER SLOUGH SUBBASIN FLOWPATHS

FIGURE

4-5

4.3.5. Channel Routing

The PBI Model utilizes the Muskingum-Cunge routing method to represent attenuation of flood waves within Mosher Slough channels. Routing reach lengths and slopes were measured using ArcGIS software. Manning's n values and channel cross-sections were imported from the 1998 SJAFCA HEC-1 model.

Table 4- 2 provides a summary of routing elements included in the PBI Model.

Table 4- 2. Summary of Mosher Slough model routing elements.

			Manning's n		Description	
Routing Element	Length [ft]	Slope [ft/ft]	Main Channel	Overbank	From	To
R1104	16,260	0.0009	0.035	0.06	1105	1104
R0403	5,760	0.0009	0.035	0.06	1104	1103B/1103C
R3B3A	3,230	0.0015	0.035	0.06	1103B/1103C	1103A
RNC	4,620	0.0011	0.035	0.06	1103A	CHER
RCC	4,130	0.0004	0.035	0.06	CHER	CAY
RCE	3,880	0.0013	0.035	0.06	CAY	ELD
RET	3,700	0.0011	0.035	0.06	ELD	THOR
RTD	4,540	0.0020	0.035	0.06	THOR	DON/RYAL
RLSAC	3,030	0.0010	0.035	0.06	LSAC	RYAL
RRYAL	5,930	0.0022	0.035	0.06	RYAL	DON
RYB	1,740	0.0006	0.035	0.06	DON/RYAL	BAIN
RBK	720	0.0014	0.035	0.06	BAIN	KELLY/YAR
RKT	1,790	0.0006	0.035	0.06	KELLY/YAR	TCREEKS
RTA	8,210	0.0004	0.035	0.06	TCREEKS	ATLAS

Fourteen reaches covering a total of approximately 13 miles of the Mosher Slough stream system are included in the PBI Model.

4.3.6. Loss Rates

As discussed in Section 4.2.1, the 1998 SJAFCA HEC-1 model used the SCS Curve Number method to calculate loss rates. The PBI Model differs from the SJAFCA HEC-1 model in that it uses the initial and constant loss rate method to model subbasin losses.

The Natural Resources Conservation Service (NRCS) has classified all soils into four hydrologic soil groups (A, B, C, and D) according to their infiltration rates[15]:

Table 4- 3. NRCS hydrologic soil groups.

Hydrologic Soil Group	Loss Rate Range [in/hr]	PBI's Assumed Loss Rate[a] [in/hr]
A	> 0.30	0.35
B	0.15 - 0.30	0.2
C	0.05 - 0.15	0.1
D	0.00 - 0.05	0.025

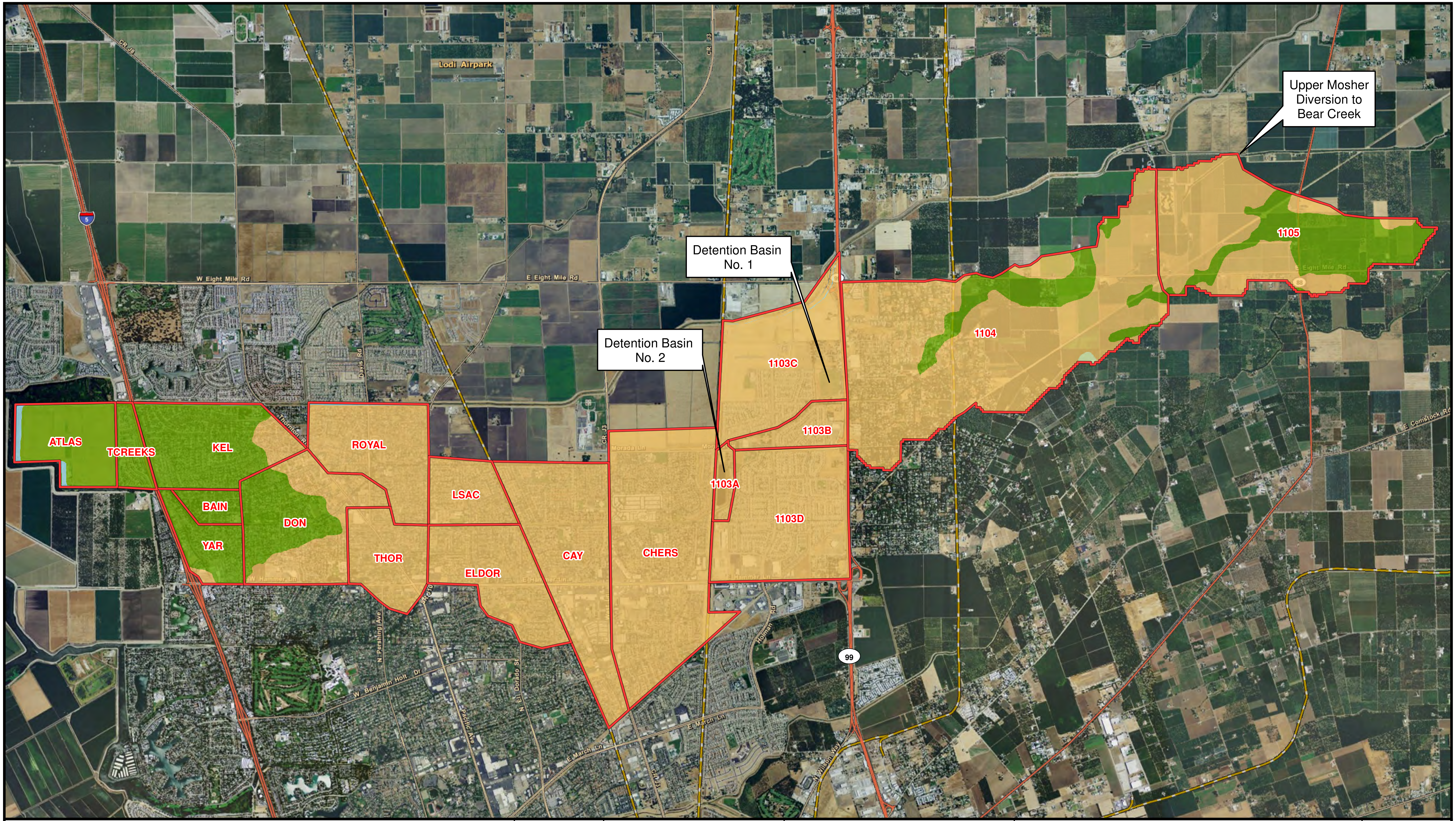
[a]This loss rate value was assigned to each soil group for initial calculations of composite loss rates. The calculated composite loss rates were then adjusted during the calibration process.

A GIS soils layer was obtained from the NRCS[13] and used to determine the proportional coverage of soil groups within Mosher Slough subbasins (Figure 4- 6). A weighted average of loss rates was calculated for each subbasin and adjusted during the calibration process (See Section 4.4). After the calibration adjustment, subbasin loss rates range from 0.02 inches per hour to 0.08 inches per hour as shown in Attachment 4-B.

Initial losses were set at 1.5 inches for pervious areas within all subbasins to account for precipitation that is infiltrated or stored in the watershed before surface runoff begins. This value was selected based on a the Army Corps of Engineers' Comprehensive HEC-HMS study[9] which suggested a range of 1.5 to 2.5 inches for initial losses in the Mosher Slough study area.

4.3.7. Impervious Percentages

Impervious percentages were assigned based on the extent of urbanization within each subbasin. Aerial photos including those contained within 2010 LiDAR datasets[4] covering the Mosher Slough watershed were used to assess existing urbanization. Subbasins were classified into three categories with assumed impervious percentages as shown in Table 4- 4.



🟧 Group A
🟩 Group B
🟩 Group C
🟨 Group D
🟦 Water/Other

0 1,000 2,000 4,000 Feet
1 inch = 4,000 feet

AUGUST 20, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

MOSHER SLOUGH SOILS MAP

FIGURE

4-6

The impervious percentages corresponding to each land use type were selected with the guidance of San Joaquin County *Hydrology Manual*[10].

Table 4- 4. Land use types and their corresponding impervious percentages.

Land Use Type	Impervious Percentage
Agricultural	2%
Agricultural with Rural Residential Development	5%
Fully Developed Residential	60%

4.4. MODEL CALIBRATION

The 1998 SJAFCA HEC-1 model documentation does not mention how/if the Mosher Slough model was calibrated. Lower Mosher Slough is largely regulated through pump stations and detention ponds. This flow regulation reduces the importance of model calibration.

Calibration to an observed rainfall/runoff event was considered for the PBI Model, however there was very little concurrent rainfall/runoff data in the Mosher Slough watershed. The available runoff data included stage recordings and did not include a rating curve. Calibration to an observed event would have contained a large amount of uncertainty and therefore was not included in the Mosher Slough analysis.

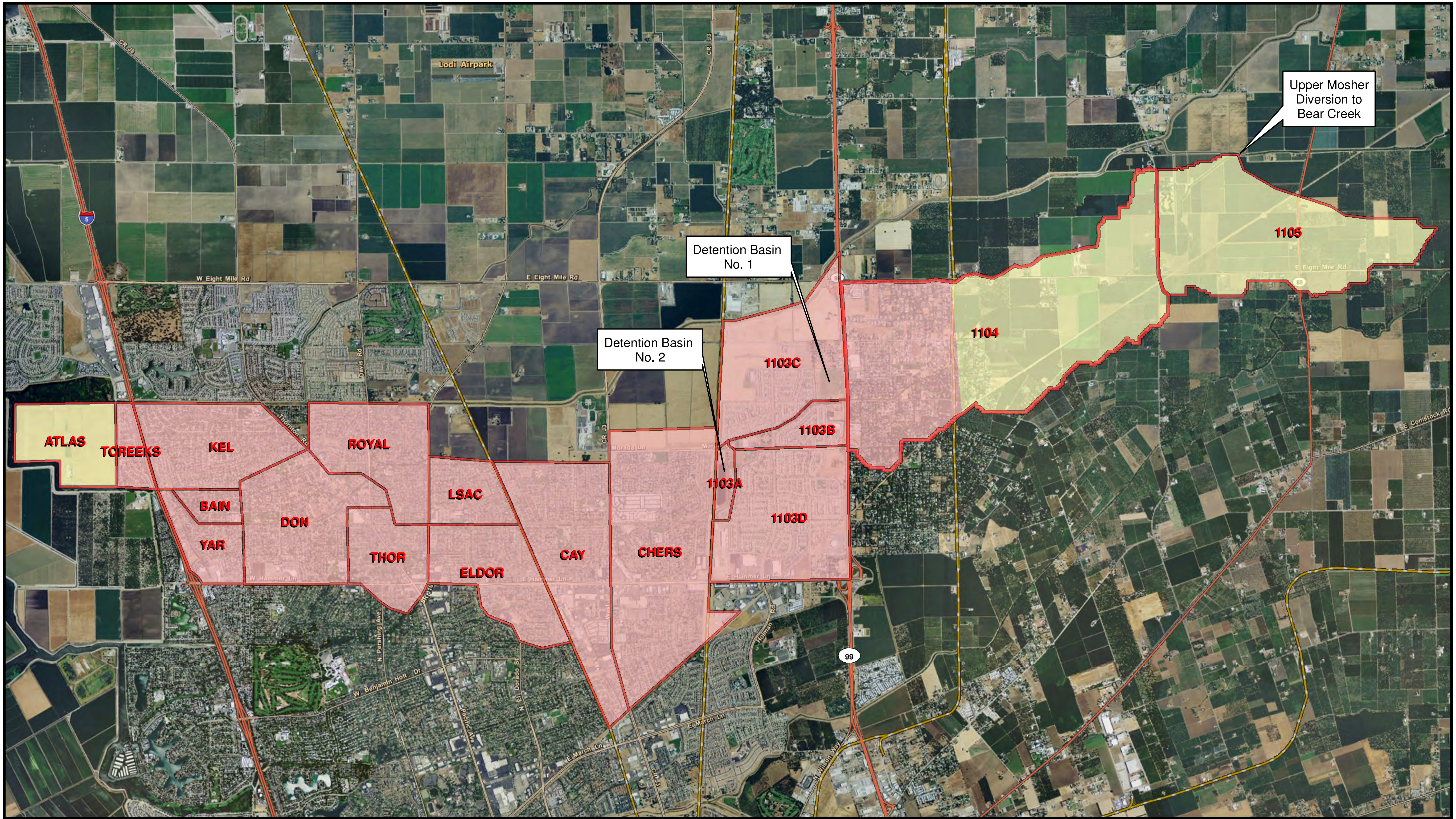
Constant loss rates were adjusted for each subbasin by a factor of 0.80 (Attachment 4- B). The adjustment factor was determined through a HEC-HMS calibration for the neighboring Bear Creek watershed. This watershed has similar characteristics to the Mosher Slough watershed and has more reliable stream flow data. Further details of the Bear Creek model calibration can be found in Section 3.4.

4.5. DEVELOPMENT CONDITIONS

4.5.1. Existing Conditions

An 'Existing Conditions' model run was performed to evaluate peak flows given current (2010) land use and hydrologic conditions within the Mosher Slough watershed. Subbasin S-graphs, 'n' values, and impervious percentages were set according to current land cover conditions using field knowledge supplemented by aerial photos.

As shown in Figure 4- 7, the downstream watershed (west of Highway 99) is considered fully developed and generally consists of residential neighborhoods. Runoff from each of these



🟥 Developed Area

🟨 Undeveloped Area

N

0 1,000 2,000 4,000 Feet
1 inch = 4,000 feet

AUGUST 20, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING
1180 Iron Point Rd., Suite 260
Folsom, CA 95630
Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

EXISTING DEVELOPMENT CONDITIONS FOR MOSHER SLOUGH WATERSHED

FIGURE
4-7

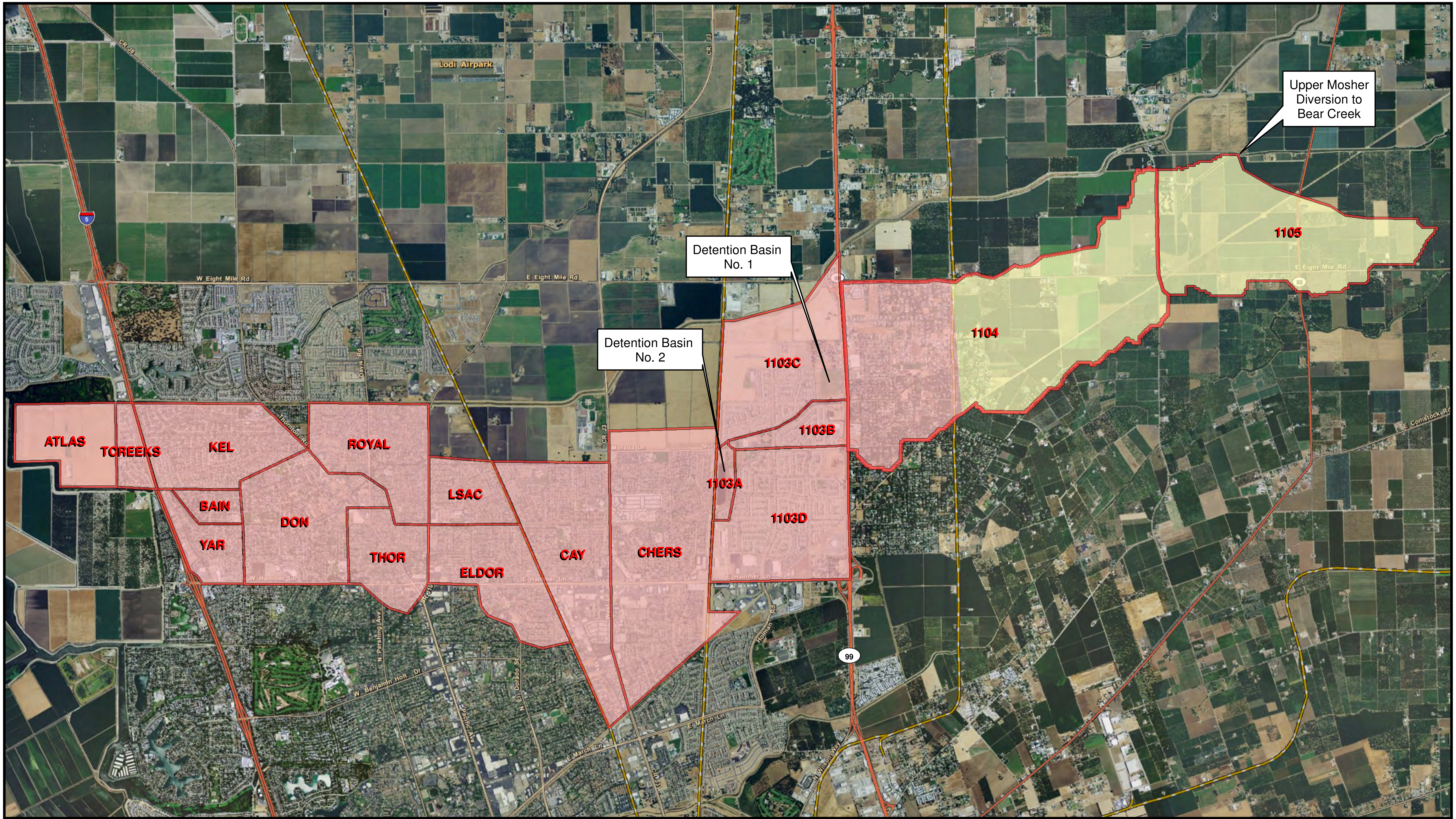
subbasins is routed through pump stations that discharge flows up to their design capacities (see Section 4.3.2) into Mosher Slough. Any subbasin flows exceeding pump station capacities would result in temporary ponding within the subbasin. This ponding would be entirely due to inadequate pump capacities and would be independent of exterior stage conditions in the receiving stream.

The subbasins east of Highway 99 are primarily agricultural lands. Flows from these subbasins are regulated by ‘Detention Basin No. 1’ as discussed in Sections 4.3.2. A summary table of the subbasin characteristics used for ‘Existing Conditions’ model runs is provided in Attachment 4-C.

4.5.2. Future-Without-Project Conditions

A ‘Future-Without-Project Conditions’ model run was performed to evaluate peak flows for future (2070) land use and hydrologic conditions within the Mosher Slough watershed. Land use conditions are based on the City of Stockton 2035 General Plan[12] and the San Joaquin County General Plan[17].

As shown in Figure 4- 8, land use remains largely unchanged from the ‘Existing Conditions’ model given that most of the watershed was already developed. The only change in land use conditions occurs in the Atlas tract subbasin. This 0.51 square mile area is expected to become developed and is routed through a stormwater pump station into Mosher Slough at a maximum capacity of 120.8 cfs (see Section 3.3.2). A summary table of the subbasin characteristics used for ‘Future-Without-Project Conditions’ model runs is provided in Attachment 4-D.



🟥 Developed Area

🟨 Undeveloped Area

0 1,000 2,000 4,000 Feet

1 inch = 4,000 feet

AUGUST 20, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

FUTURE DEVELOPMENT CONDITIONS FOR MOSHER SLOUGH WATERSHED

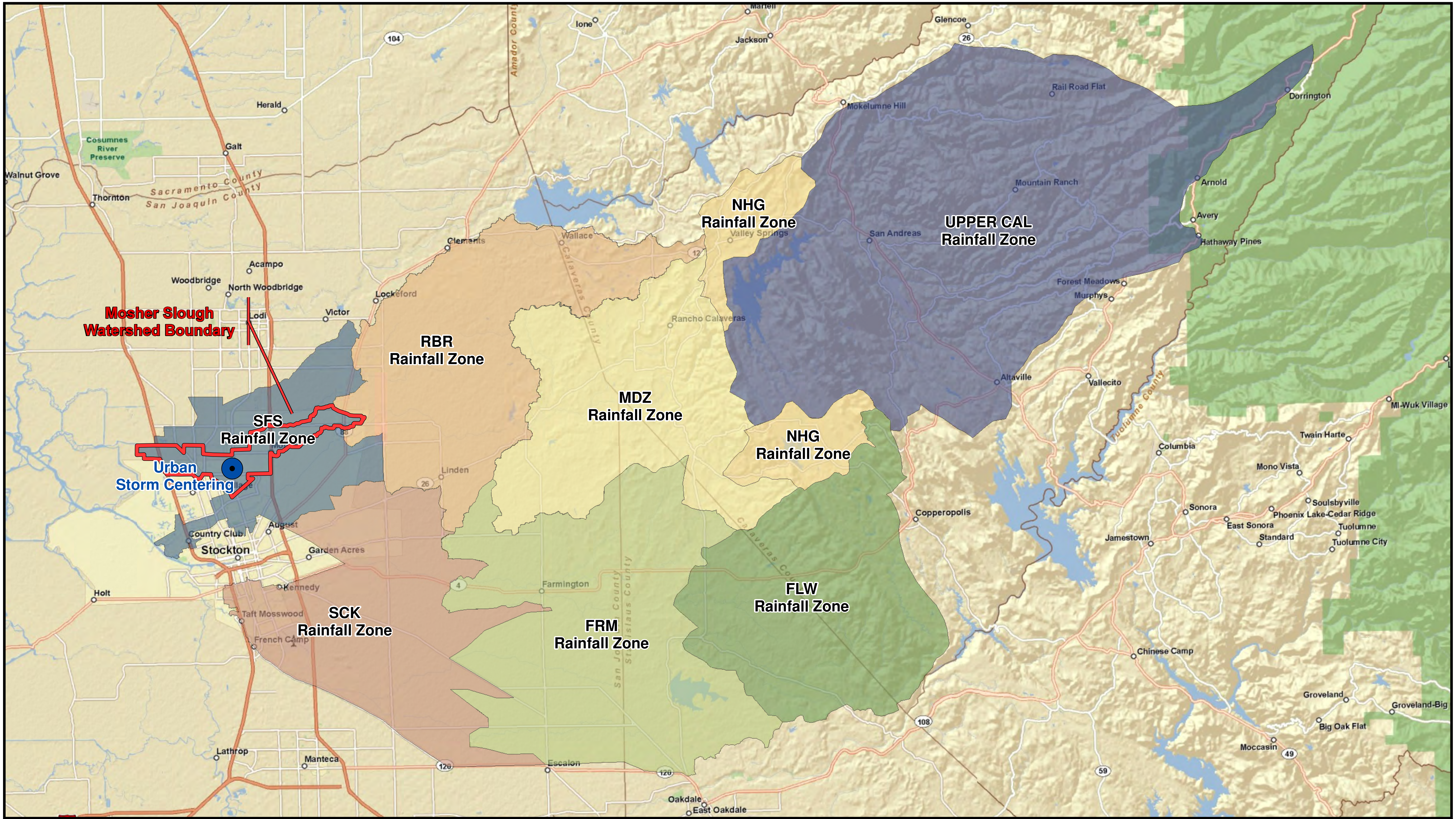
FIGURE

4-8

4.6. STORM CENTERINGS

Because of the smaller size of the watershed, only one storm centering was analyzed for Mosher Slough (Figure 4- 9). This urban centering was placed directly over the center of the watershed and the 8 AEP storm frequencies were analyzed.

Calculated area reduction factors and resulting area-reduced rainfall depths for each rainfall zone are provided in Attachment 4-E for all frequency-duration combinations.



N

0 5 Miles

1 : 300,000

JUNE 24, 2011

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

MOSHER SLOUGH WATERSHED STORM CENTERINGS

FIGURE

4-9

4.7. MODEL SIMULATIONS

Mosher Slough production runs include 16 scenarios with unique combinations of development conditions and storm frequencies.

Table 4- 5. Mosher Slough production run scenarios.

Development Conditions	Storm Centerings	AEP Events
Existing Conditions	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
Future-Without-Project Conditions (2070)	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500

4.7.1. Summary of Results

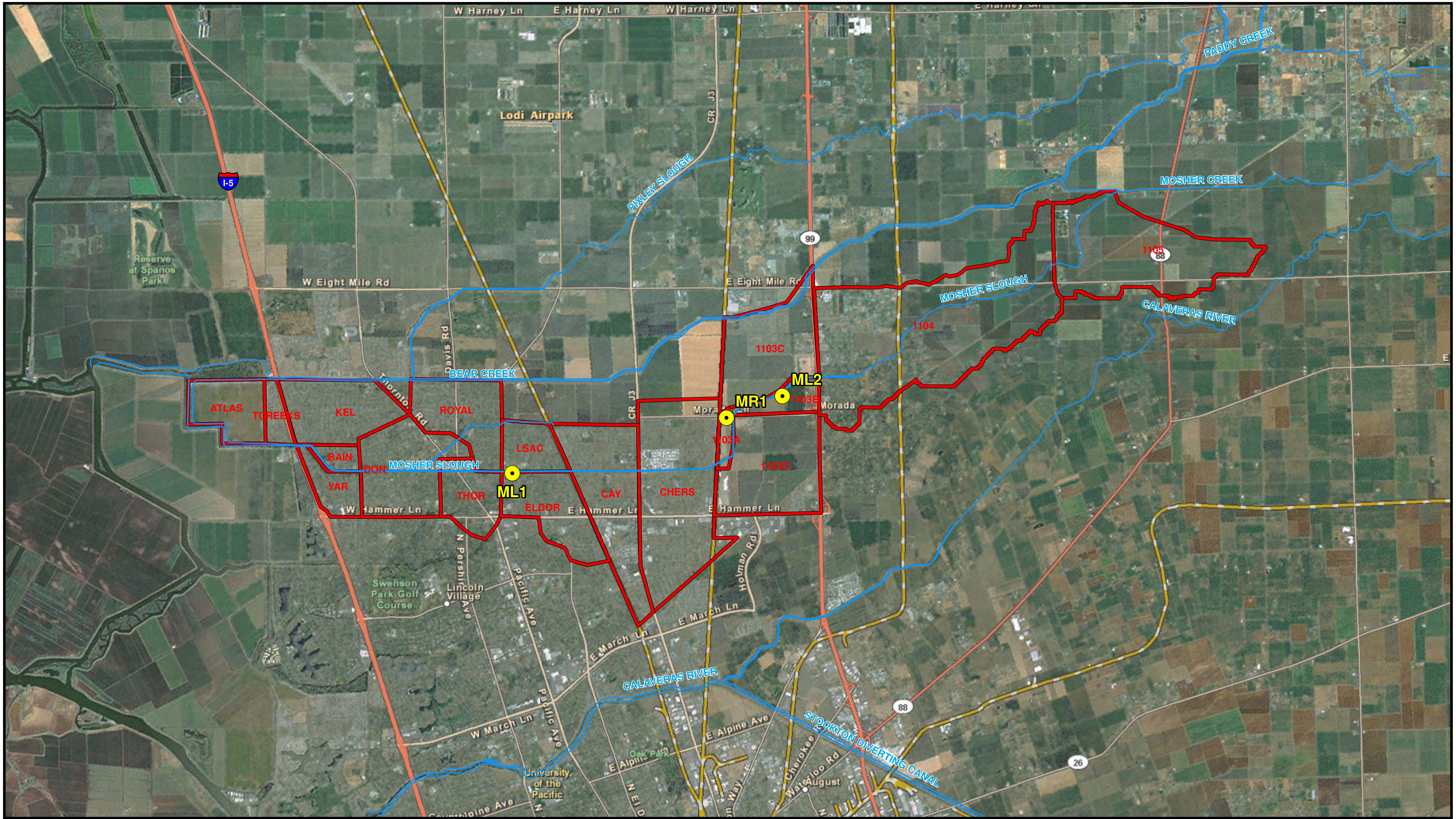
Peak flow results were extracted from HEC-HMS at each LSJRFS index point. Locations of LSJRFS index points within the Mosher Slough watershed are shown in Figure 4- 10. Table 4- 6 and Table 4- 7 summarize peak flows for 'Existing Conditions' runs and for 'Future-Without-Project Conditions' runs, respectively.

4.7.2. Uncertainty Parameters

For the purposes of the LSJRFS, uncertainty parameters for each flow-frequency dataset can be estimated within HEC-FDA during the project's economic analysis. HEC-FDA defines uncertainty in terms of confidence intervals or standard deviations given inputs of flow-frequency data (provided in Table 4- 6 and Table 4- 7) and an equivalent record length.

The equivalent record length is an estimate of the overall "worth" or "quality" of the flow-frequency function, expressed as the number of years-of-record[19]. For probability functions derived at ungaged locations using model or other data, the equivalent record length is based on a judgment of the quality of that model or data. EM 1110-2-1619[20] provides guidelines for assigning equivalent record lengths and estimates that a rainfall-runoff model calibrated to an observed event at a short-interval runoff gage has an equivalent record length of 20-30 years.

The Mosher Slough model wasn't calibrated to an observed event, however, because stream flows are largely dependent on pumped flows, the degree of uncertainty is judged to be equivalent to a calibrated model.



● LSJRFS Index Point

▭ Subshed Boundary

N

0 0.25 0.5 1 Miles

1 inch = 1 mile

JUNE 20, 2012

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

MOSHER SLOUGH WATERSHED INDEX POINTS

FIGURE

4-10

Table 4-6. Peak Flow Results for Mosher Slough - Existing Conditions [cfs]

LSJRFS Index Point ID	Description	Urban Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
--	Mosher Slough at Hwy 99	170	240	290	360	420	470	530	600
ML2	Mosher Slough d/s of Detention Basin #1	170	230	230	230	230	230	230	510
--	Mosher Slough at SPRR (d/s of La Morada)	180	240	270	320	320	320	320	590
--	Mosher Slough at UPRR	390	530	570	640	730	770	780	840
ML1	Mosher Slough at El Dorado St.	440	620	690	800	890	940	960	970
--	Mosher Slough at Thornton Ave.	450	590	690	790	890	950	980	1,000
--	Mosher Slough at Don Ave	630	810	930	1,030	1,160	1,230	1,270	1,290
--	Mosher Slough at I-5	690	860	1,040	1,250	1,360	1,420	1,500	1,540
--	Mosher Slough u/s of Bear Creek Confluence	570	750	890	1,050	1,160	1,260	1,390	1,450

Table 4-7. Peak Flow Results for Mosher Slough - Future Conditions [cfs]

LSJRFS Index Point ID	Description	Urban Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
--	Mosher Slough at Hwy 99	170	240	290	360	420	470	530	600
ML2	Mosher Slough d/s of Detention Basin #1	170	230	230	230	230	230	230	510
--	Mosher Slough at SPRR (d/s of La Morada)	180	240	270	320	320	320	320	590
--	Mosher Slough at UPRR	390	530	570	640	730	770	780	840
ML1	Mosher Slough at El Dorado St.	440	620	690	800	890	940	960	970
--	Mosher Slough at Thornton Ave.	450	590	690	790	890	950	980	1,000
--	Mosher Slough at Don Ave	630	810	930	1,030	1,160	1,230	1,270	1,290
--	Mosher Slough at I-5	690	860	1,040	1,250	1,360	1,420	1,500	1,540
--	Mosher Slough u/s of Bear Creek Confluence	590	760	910	1,070	1,190	1,290	1,400	1,480

5.0 CALAVERAS RIVER HEC-HMS MODELING

5.1. GENERAL

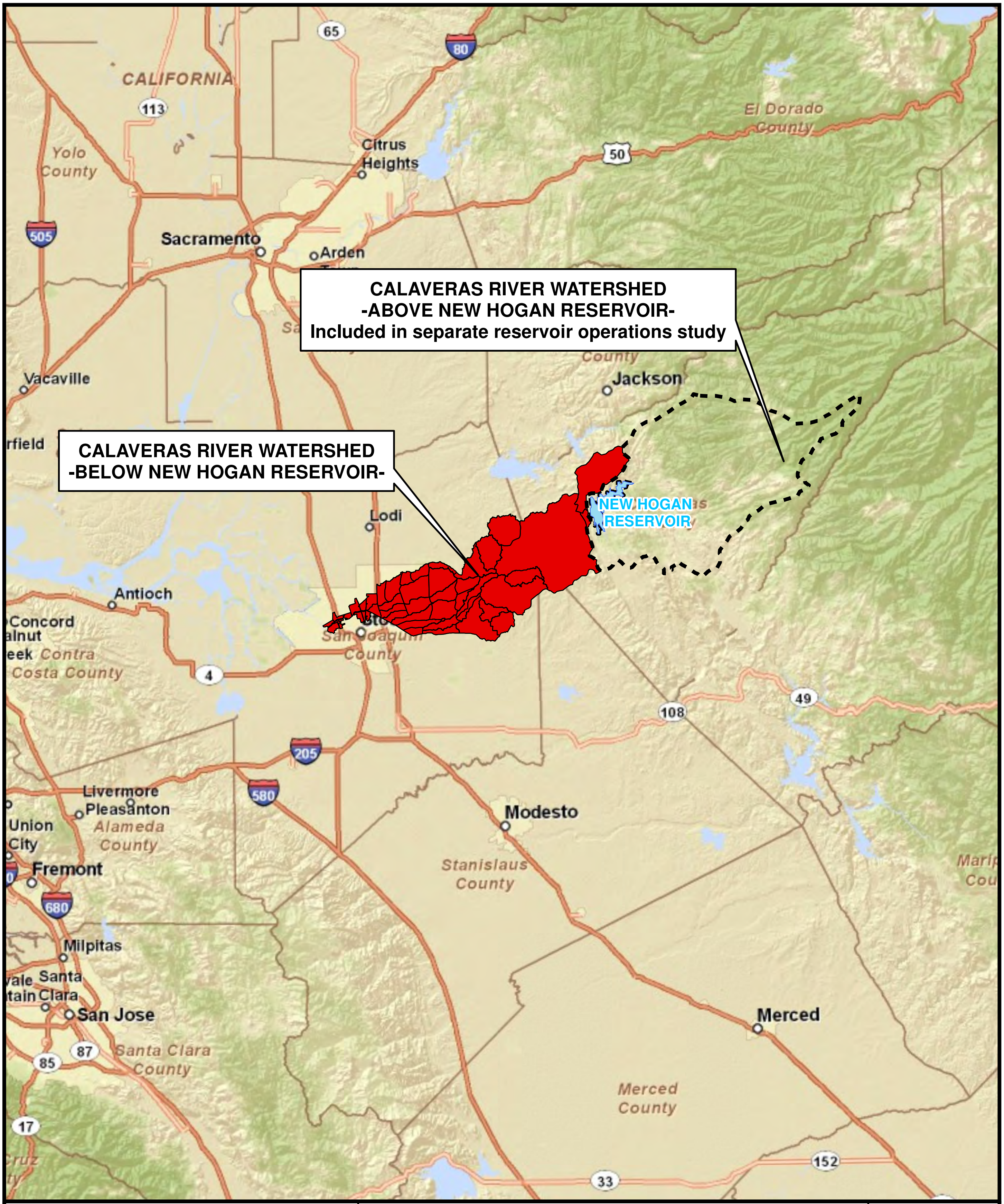
5.1.1. Location

The Calaveras River watershed is located near the city of Stockton in San Joaquin County, California (Figure 5- 1). The watershed runs east from the city of Stockton into the Sierra Nevada foothills in Calaveras County. The Calaveras River watershed can be split into two sections: above New Hogan Dam and below New Hogan Dam. This document focuses on the section of the Calaveras River below the dam whereas the section above the dam is part of a separate reservoir operations study[21].

The watershed includes a total area of 597 square miles with 352 square miles of this tributary area flowing into New Hogan Reservoir. The watershed discussed in this TM (below New Hogan Reservoir) includes the remaining 245 square miles and achieves maximum elevations of 1,500 feet. It then descends through moderate slopes to the lower portion of the watershed which lies at sea-level. Flow in the stream system is largely affected by releases from New Hogan Reservoir. The entire watershed is low enough in elevation to be rainfall dominant. The HEC-HMS model described in this memorandum includes the Calaveras River, Cosgrove Creek, Mormon Slough, Potter Creek, and the Stockton Diverting Canal systems and discharges to the San Joaquin River to the west of Interstate-5.

5.1.2. Topography

The HEC-HMS model utilized for this study is titled the PBI Calaveras River Model (PBI Model) which is georeferenced to the NAD 1983 State Plane California Coordinate System Zone III (U.S. Survey Feet). Vertical elevations are reported in the NAVD 1988 datum. Topography used for model development included United States Geological Survey (USGS) 30-meter Digital Elevation Models (DEMs)[3]. Where available, Department of Water Resources (DWR) LiDAR data[4] was also used to confirm subbasin boundaries.



CALAVERAS RIVER WATERSHED
-ABOVE NEW HOGAN RESERVOIR-
Included in separate reservoir operations study

CALAVERAS RIVER WATERSHED
-BELOW NEW HOGAN RESERVOIR-

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

VICINITY MAP OF THE
CALAVERAS RIVER STUDY AREA

FIGURE
5-1

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

5.2. MODEL DEVELOPMENT

The PBI model was developed using HEC-HMS version 3.4[5] and HEC-GeoHMS version 4.2[6]. A summary of the tasks performed are listed below:

1. The 1998 SJAFCA HEC-1 model was imported into HEC-HMS (See Section 5.2.2).
2. Subbasin boundaries were updated using HEC-GeoHMS and United States Geological Survey (USGS) Digital Elevation Models (DEMs)[3] (See Section 5.3.1).
3. Lower Calaveras River subbasins (below the confluence with the Diverting Canal) were added to the PBI Model (See Section 5.3.1).
4. Pump stations were coded into the PBI Model based on design pumping rates provided by the City of Stockton[7] (See Section 5.3.2).
5. New Hogan Reservoir outflows were determined through a separate reservoir operations study (See Section 5.3.3)
6. Diversions and channel routing parameters were coded into the PBI Model (See Sections 5.3.4 and 5.3.6, respectively).
7. S-graphs and lag times were assigned to each subbasin (See Section 5.3.5).
8. Loss rates and impervious percentages were coded into the PBI Model (See Section 5.3.7 and Section 5.3.8).
9. The PBI Model was calibrated using historical rainfall and runoff data (See Section 5.4).
10. The PBI Model was set up to simulate both ‘Existing’ (see Section 5.5.1) and ‘Future-Without-Project’ (see Section 5.1.1) scenario runs.

5.2.1. SJAFCA HEC-1 Model

The PBI Model is a conversion and update of the HEC-1 model developed for SJAFCA by HDR Engineering, Inc. in 1998[8].

The 1998 SJAFCA HEC-1 model utilized the San Joaquin County LA preprocessor to convert S-graphs to unit hydrographs for each subbasin. Three types of S-graphs were obtained from the San Joaquin County Hydrology Manual[10] and used based on the surface condition classification of the subbasin: Foothill, Valley Undeveloped, and Valley

Developed. Lag times were calculated by HDR using basin ‘n’, length of subbasin flow, flow length from the centroid, and slope of the basin.

The SJAFCA HEC-1 model used the SCS curve number method to account for subbasin losses. Curve numbers typically ranged from 70 to 84 depending on soil type and cover.

Calibration of the SJAFCA HEC-1 model included calculating the flow per square mile for the 1/100 AEP event at the Duck Creek near Farmington gage and comparing it to the modeled flow per square mile coming from the foothill portions of Potter Creek. HDR made adjustments to basin ‘n’ values such that the 1/100 AEP rainfall event would produce the 1/100 AEP streamflow event.

5.2.2. Conversion from HEC-1 to HEC-HMS

The 1998 SJAFCA HEC-1 model was imported into HEC-HMS as the fundamental basis for the PBI Model. Parameters from the HEC-1 model are listed in Attachment 5-A.

Certain features in the HEC-1 software are not supported in HEC-HMS and therefore were not properly transferred during the import process. Pump station data and meteorological data from the SJAFCA HEC-1 model were manually coded into the PBI Model so as to conform to HEC-HMS formatting.

In addition, there are computational differences between the HEC-1 and the HEC-HMS software. One such difference involves the Muskingum-Cunge stream segment routing technique used for the PBI model. In HEC-HMS, channel properties are computed based on the physical characteristics of that channel, whereas in HEC-1 the properties are computed with formulas based on a kinematic wave assumption[5]. This causes minor differences in the flows that are transferred through the routing parameters. HEC-HMS results are preferred because of the refined computational techniques that have been implemented.

Once the conversion from HEC-1 to HEC-HMS was completed successfully, the HEC-HMS model was modified to run with updated features.

5.3. MODEL FEATURES

The 1998 SJAFCA HEC-1 model was converted and modified for this study to form the PBI HEC-HMS Model. The PBI Model components are described in the following sections.

5.3.1. Subbasins

Subbasin boundaries used in the 1998 SJAFCA HEC-1 model were cross-checked with USGS 30-meter DEM datasets[3] and modified where appropriate. Subbasin boundaries were delineated using the ArcHydro and HEC-GeoHMS[6] extensions within the ArcGIS software package. These tools utilize geospatial data to interpret drainage patterns and delineate watershed boundaries accordingly. Subbasin outlet points were set similar to the locations utilized in the SJAFCA HEC-1 model. Where available, DWR LiDAR[4] data was used to confirm subbasin boundaries.

The 1998 SJAFCA HEC-1 model boundary was extended along the Lower Calaveras River (below the confluence with the Diverting Canal) by adding 12 subbasins. These subbasin boundaries were based on the existing storm drain system and the City of Stockton's *Conceptual Storm Drain Master Plan*[11]. The Calaveras River subbasins included in the PBI Model are shown in Figure 5- 2.

Subbasin 'C80' from the SJAFCA HEC-1 Model was renamed to 'HOLM' as it corresponds to the Holman stormwater pump station's drainage area.

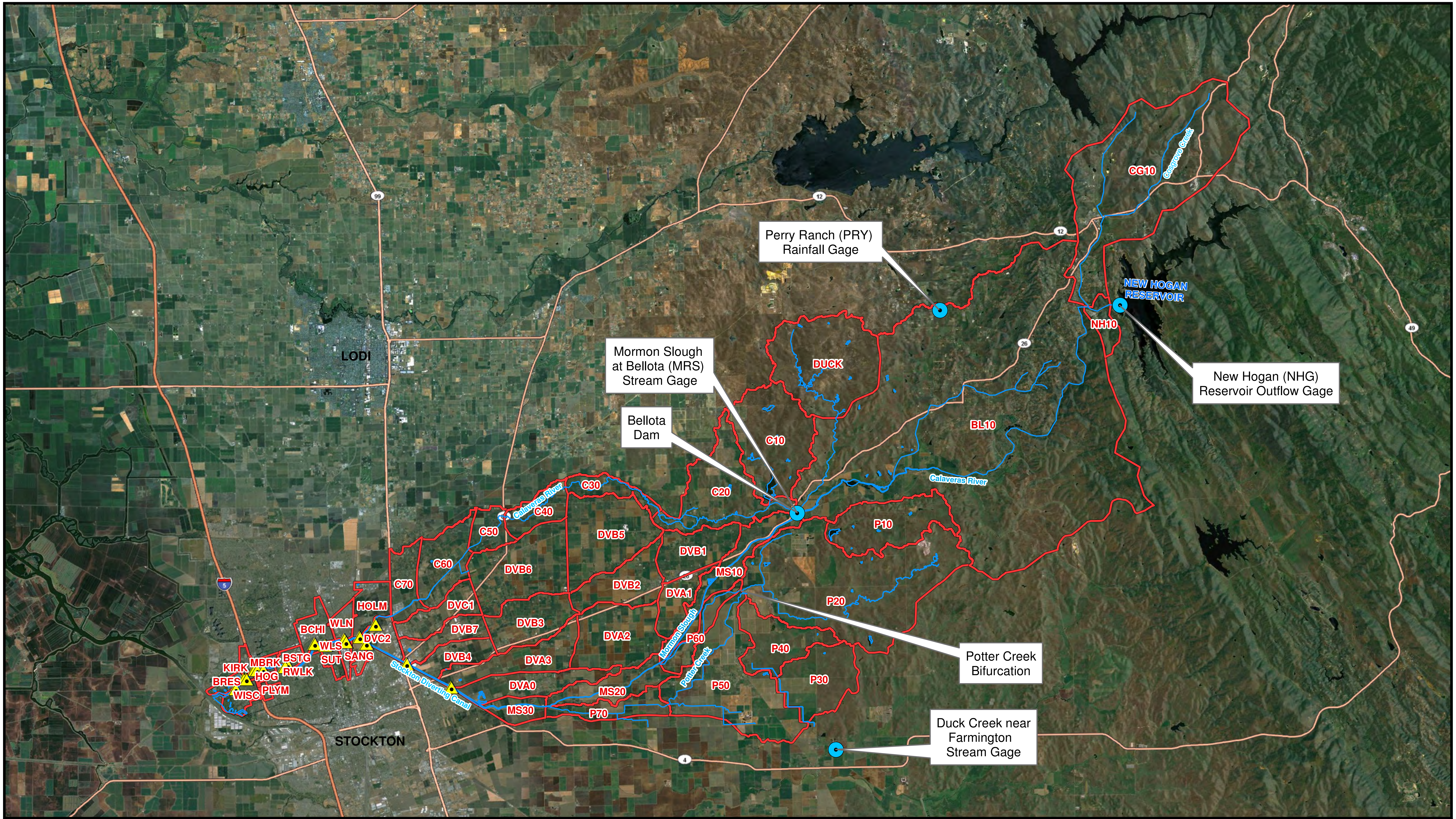
The PBI Model contains a total of 48 subbasins with drainage areas ranging from 0.02 square miles to 72.63 square miles and a total watershed area of approximately 245 square miles.

For subbasins that are on the outside of a levee which do not have pump stations, runoff is coded to enter the main channel at road crossings where there are through-levee culverts. The assumption is made that the culvert headgates will remain open and allow outside flow to enter the main channel. This assumption was made to remain conservative and to account for the potential replacement of culverts by pump stations in the future.

The GIS horizontal coordinates for each subbasin were used to georeference model elements within the PBI HEC-HMS Model. The subbasin GIS shapefile was inserted into the PBI Model as a background map.

5.3.2. Pump Stations

Pump stations were included in the PBI Model to represent storm drain conveyance from developed subbasins to the main channels. There are sixteen (16) pump stations included in the PBI model. Pumps along the Diverting Canal were imported directly from the 1998 SJAFCA HEC-1 Model. Pump stations along the Lower Calaveras River (below the confluence with the Diverting Canal) include multiple pumps with capacities assigned based on City of Stockton records[7]. All pumps are set to discharge over the top of the levees and into the receiving channel above the highest stage expected. The exterior and interior areas at the pump stations are independent from one another.



🟥 Subbasin Boundary

⚠ Existing Pump Station

0 1.5 3 Miles

1 inch = 3 miles

SEPTEMBER 21, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

CALAVERAS RIVER
HEC-HMS SUBBASINS

FIGURE

5-2

Table 5- 1 provides a summary of pump stations included in the PBI Model.

Table 5- 1. Summary of Calaveras River pump stations.

Pump Station Name	Contributing Subbasin(s)	Pump Station Capacity [cfs]	Pump Station Notes
Brookside Estates South	BRES	39.0	3 @ 12.0 cfs 1 @ 2.9 cfs
Wisconsin	WISC	21.7	1 @ 10.2 cfs 1 @ 11.5 cfs
March-Brookside	MBRK	121.3	1 @ 7.7 cfs 3 @ 37.9 cfs
Kirk	KIRK	14.5	1 @ 14.5 cfs
Plymouth	PLYM	6.0	1 @ 6.0 cfs
Hogue-Tyler	HGTY	6.2	1 @ 6.2 cfs
Riverwalk	RWLK	10.5	1 @ 10.5 cfs
Brookside-Stagg	BSTG	132.7	1 @ 7.9 cfs 2 @ 62.4 cfs
Bianchi	BCHI	176.9	2 @ 1.6 cfs 1 @ 22.3 cfs 2 @ 64.6 cfs 1 @ 13.4 cfs 1 @ 8.9 cfs
Sutter	SUT	54.1	1 @ 11.1 cfs 1 @ 20.7 cfs 1 @ 22.3 cfs
West Lane - North	WLN	254.0	1 @ 8.9 cfs 5 @ 49.0 cfs
West Lane - South	WLS	47.5	1 @ 36.3 cfs 1@ 11.1 cfs
Sanguinetti	SANG	92.2	1 @ 9.8 cfs 2 @ 41.2 cfs
Holman	HOLM	140.1	2 @ 34.5 cfs 1 @ 2.0 cfs 2 @ 34.5 cfs
Diverting Canal & Route 26 (P1)	DIVA0, DIVA1, DIVA2, DIVA3	16.0	1 @ 16.0 cfs
Diverting Canal & HWY 99 (P2)	DIVB1, DIVB2, DIVB3, DIVB4, DIVB5, DIVB6, DIVB7 + Excess from P1	100.0	1 @ 100 cfs

There is an additional pump station located along the Diverting Canal at its confluence with the Calaveras River. The coding of this model element (P-OUT) includes the combined flow from a small pump station and two 6’x 6’ reinforced concrete box culverts that relieve ponding behind the Diverting Canal.

5.3.3. New Hogan Reservoir

David Ford Consulting Engineers (DFCE) completed a separate reservoir operations analysis for New Hogan Reservoir as part of the LSJRFS[21]. This analysis was later amended by USACE as documented in their *Draft Memorandum for Record: Lower San Joaquin River Feasibility Study, Bellota and Farmington Regulated Flow Hydrographs* (07 FEB 2012)[22]. One of the final deliverables from this study was regulated hydrographs at the Bellota control point for each of the 8 LSJRFS AEP storm events. These hydrographs include all flows coming out of New Hogan Dam along with all local flows upstream of Bellota. These regulated flow hydrographs were coded into the PBI HEC-HMS model as time-series discharge gages and supersede all HEC-HMS inflow that comes from above Bellota.

The Ford report and the USACE amendment should be referenced for any details regarding the reservoir operations study.

The following table is based on the information in the USACE amendment and shows the flow-frequency relationship for modeled flows at the Bellota control point.

Table 5- 2. Flow-frequency at Bellota Control Point

Regulated Peak Flow values and associated volumes: Mormon Slough at Bellota					
Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes1 (as average flow for given duration)			
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)
0.5	3,515	2,491	2,400	2,144	1,527
0.2	9,515	7,702	7,164	6,053	4,562
0.1	9,388	8,527	7,560	6,102	5,345
0.04	10,319	9,307	9,206	7,943	5,485
0.02	12,500	10,300	9,900	9,400	7,800
0.01	12,500	11,400	11,300	10,900	10,100
0.005	12,500	12,400	12,200	12,000	11,300
0.002	16,000	13,500	13,100	13,000	12,500
Revised to reflect graphical fit of observed data from Jan1988 to Sep2010 for the 0.5 to the 0.04 AEP; the graphically fit data was further refined to fit the local flow frequency data by PBI. The 0.02 to 0.002 AEP events are from the revised flow transform and regulated flow-freq curve. The volumes were computed from the regulated peak to , volume transforms in the Ford report and were warped to mesh with the graphically derived peak and volume flow for the 0.5 to 0.04 AEP events.					

5.3.4. Diversions

Diversions in HEC-HMS are coded to simulate either manmade diversions or topographic flow splits. Several diversions were imported from the 1998 SJAFCA HEC-1 Model and included in the PBI Model.

Calaveras River flows are completely diverted to Mormon Slough at Bellota (see Figure 5-2). This makes subbasin C10 the initial tributary basin of the Upper Calaveras River downstream of the Bellota diversion.

The Upper Calaveras River includes two topographic diversions located downstream of Jack Tone Road. The berms along this segment of the river prevent most subbasin runoff from entering the main channel. The diversions are used to route a portion of subbasin flow through small culvert inlets at Jack Tone Road and at Highway 88. The remainder of subbasin flow is routed overland to subsequent subbasins and ultimately enters the main channel at Highway 99.

Potter Creek splits into two main branches at a location downstream of Gilmore Reservoir. A diversion element is included in the PBI Model to represent this bifurcation. Cross-sections of the two stream branches were used to determine the proper split of flow at this location.

Two diversion elements are used to represent pump stations located along the Diverting Canal. These relatively small pump stations help to relieve ponded flooding against the east levee. Any runoff coming from upstream subbasins that exceeds pump capacities is diverted overland to downslope subbasins and pump stations.

5.3.5. S-graphs and Lag Times

As discussed in Section 5.2.1, the 1998 SJAFCA HEC-1 model utilized the San Joaquin County LA preprocessor for converting S-graphs and to unit hydrographs. The PBI Model assigns Foothill, Valley Undeveloped, and Valley Developed S-graphs directly into HEC-HMS for each subbasin based on its location. S-graph data points were obtained from the San Joaquin County Hydrology Manual[10]. The S-graphs were developed based on rainfall-runoff data from Southern California catchments considered to be hydrologically similar to the local catchments. The following figures show the time versus discharge relationship for each S-graph.

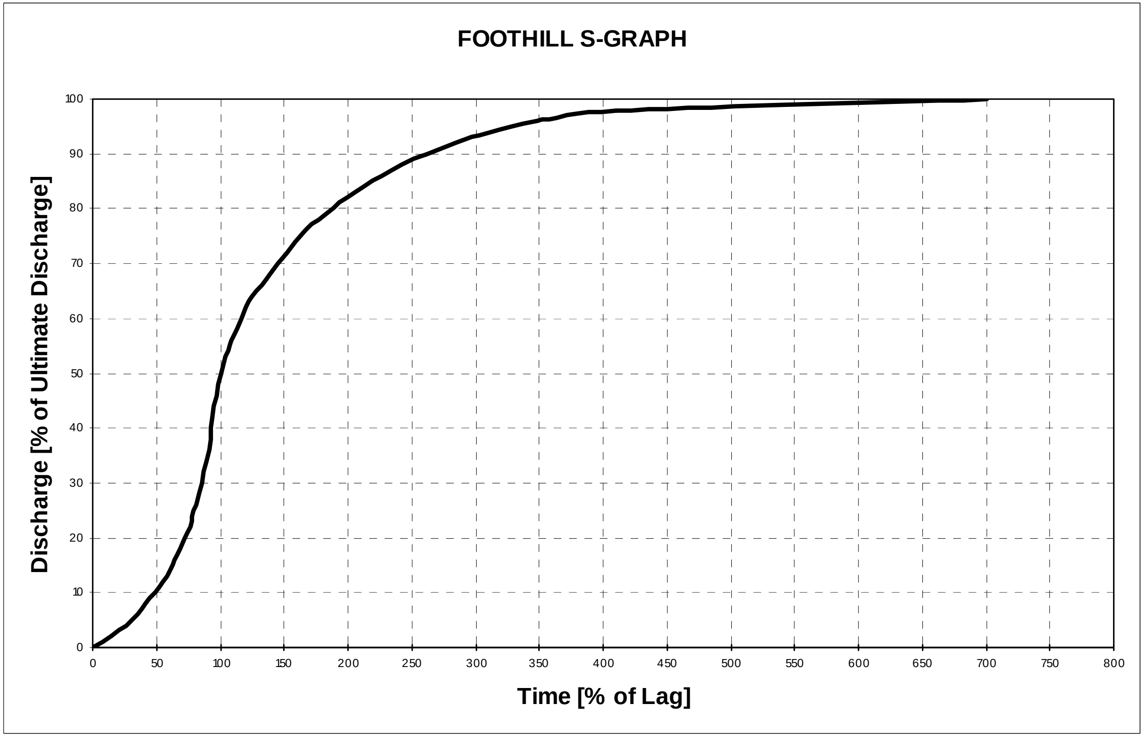


Figure 5- 3. San Joaquin County Foothill S-graph

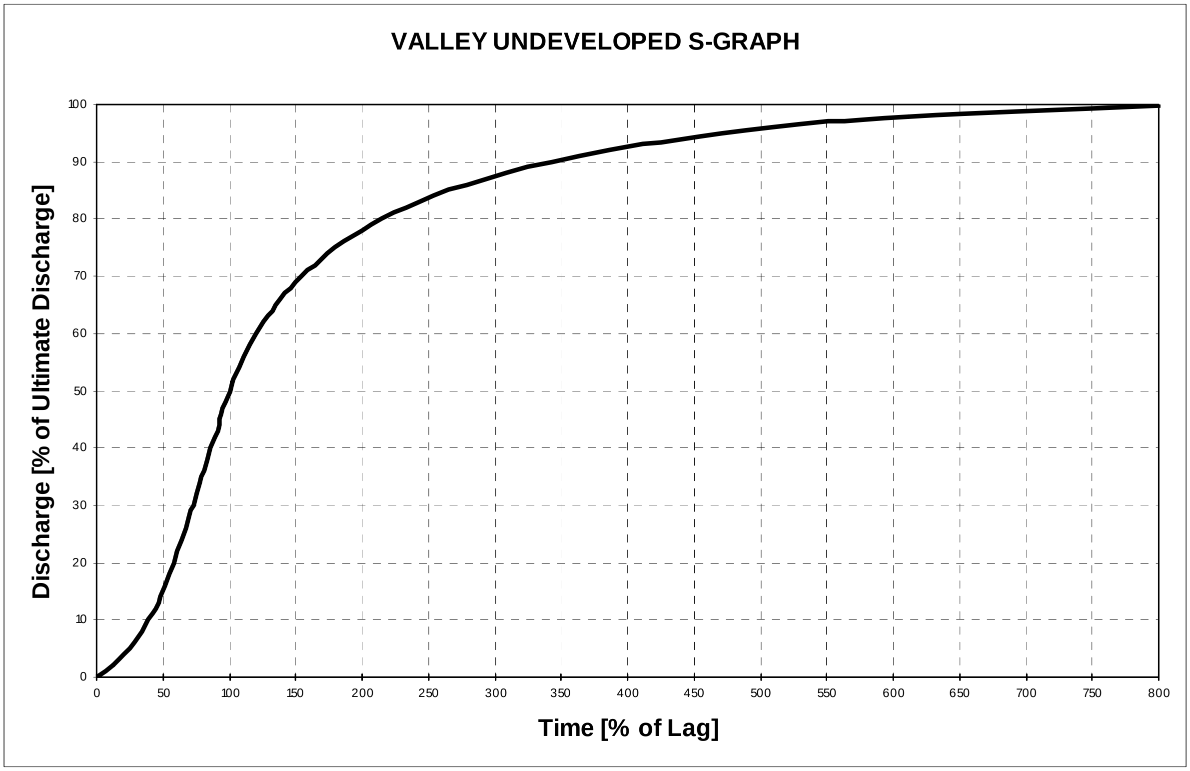


Figure 5- 4. San Joaquin County Valley Undeveloped S-graph

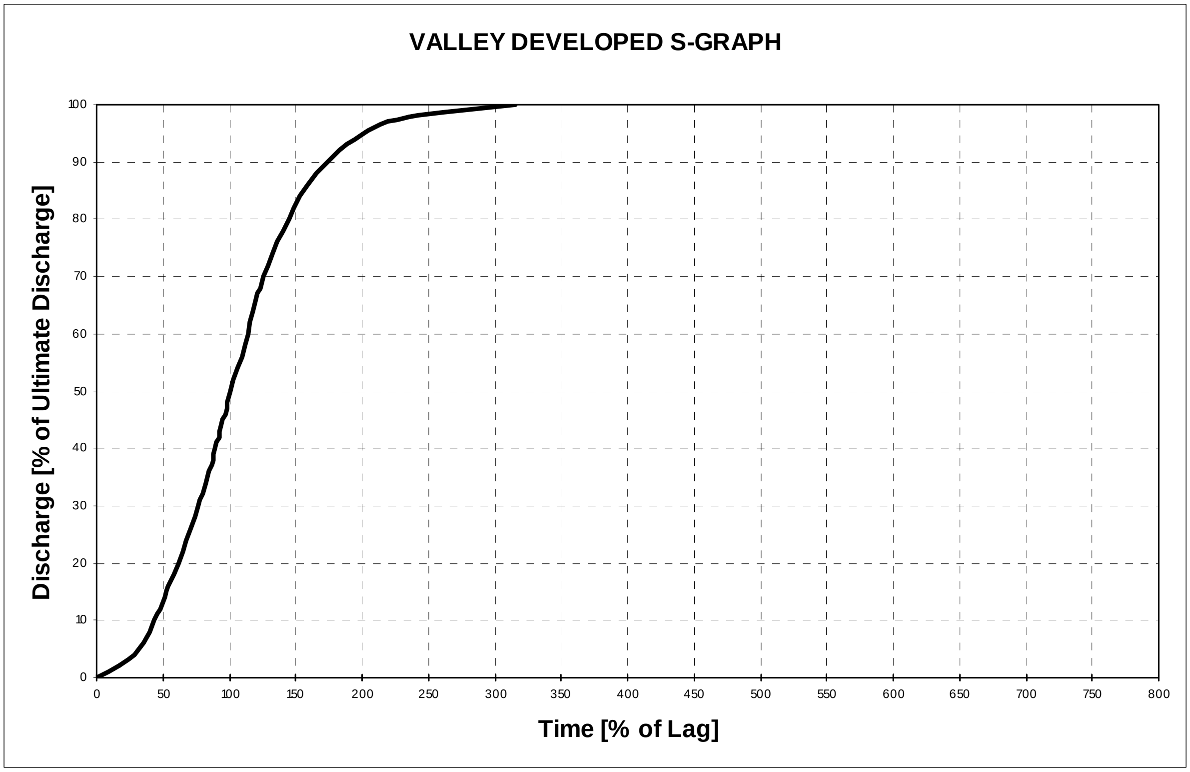


Figure 5- 5. San Joaquin County Valley Developed S-graph

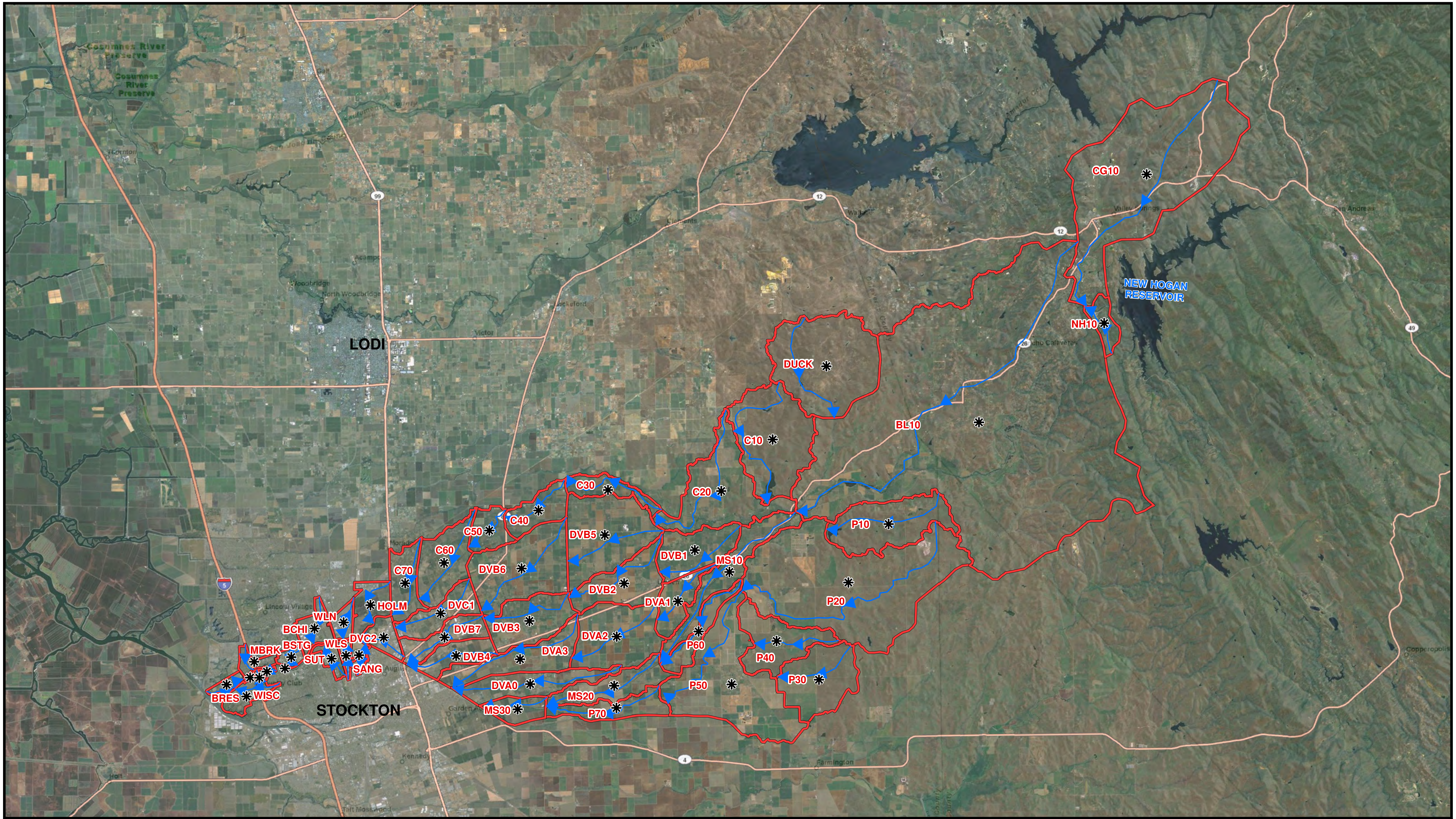
Basin lag times were calculated according to guidelines set forth in the San Joaquin County Hydrology Manual[10]. The following equation was used:

$$Lg = 24n(L \cdot L_C/S^{0.50})^{0.38}$$

where:

Lg	=	Lag time [hours]
n	=	Average basin factor estimated using Figure E-2 in the San Joaquin County Hydrology Manual
L	=	Length of longest watercourse [miles]
L_C	=	Length of longest watercourse measured to the centroid of the basin [miles]
S	=	Overall slope of longest watercourse [feet/mile]

L, L_C, and S were calculated using ArcGIS software. Flowpaths identified for these calculations are shown in Figure 5- 6. S-graph assignments and lag time calculations for each subbasin are provided in Attachment 5-B.



▭ Subbasin Boundary

✳ Subbasin Centroid

→ Subbasin Flowpath

0 1.5 3 Miles

1 inch = 3 miles

SEPTEMBER 21, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

CALAVERAS RIVER SUBBASIN FLOWPATHS

FIGURE

5-6

5.3.6. Channel Routing

The PBI Model utilizes the Muskingum-Cunge routing method to represent attenuation of flood waves within Calaveras channels. Routing reach lengths and slopes were measured using ArcGIS software. Manning's n values and channel cross-sections were imported from the 1998 SJAFCA HEC-1 model[8].

Table 5- 3 provides a summary of routing elements included in the PBI Model.

Table 5- 3. Summary of Calaveras River model routing elements.

			Manning's n			Description	
Routing Element	Length [ft]	Slope [ft/ft]	Main Channel	Left Overbank	Right Overbank	From	To
RBL	77,630	0.0049	0.035	0.05	0.05	CG10-NH10	BL10
RDUCK	19,500	0.0022	0.035	0.05	0.05	DUCK	BL10
R1010	36,830	0.0010	0.035	0.08	0.05	BL10-P10	MS10-P60
R2060	21,680	0.0009	0.04	0.08	0.08	P20	P60
R1020M	22,100	0.0014	0.035	0.06	0.06	MS10-P60	MS20
R1020P	22,950	0.0017	0.04	0.08	0.08	P10	P20
R2050	28,380	0.0009	0.04	0.08	0.08	P20	P30-P40-P50
R5070	22,350	0.0011	0.045	0.05	0.05	P50	P70
R2030M	11,280	0.0011	0.04	0.08	0.08	MS20-P70	MS30
R7080	7,040	0.0017	0.04	0.08	0.08	MS30	DIVA0-DIVA3
R8090	8,410	0.0004	0.04	0.08	0.08	DIVA0-DIVA3	DIVB4-DIVB7
R9092	8,370	0.0005	0.04	0.08	0.08	DIVB4-DIVB7	SANG
R92	1,590	0.0006	0.04	0.08	0.08	SANG	HOLM-DIVC2
R1020	22,460	0.0016	0.055	0.08	0.06	C10	C20
R2030	19,630	0.0013	0.055	0.08	0.06	C20	C30
R3040	13,110	0.0015	0.045	0.07	0.075	C30	C40
RJDIV	11,000	0.0011	0.050	0.08	0.080	C30	C40
RDV40	9,000	0.0009	0.050	0.08	0.080	C40	C50
RSRES	18,000	0.0006	0.050	0.08	0.080	C50	C60
R60	5,000	0.0014	0.050	0.08	0.080	C60	C70
R4070	28,260	0.0008	0.05	0.08	0.075	C40	C70
R70	4,610	0.0011	0.05	0.08	0.08	C70	HOLM
R80	3,810	0.0013	0.05	0.08	0.08	HOLM	DIVC2
R100	2,720	0.0004	0.04	0.08	0.08	HOLM-DVC2	WLN-WLS
R110	4,470	0.0009	0.04	0.08	0.08	WLN-WLS	BCHI-SUT
R120	8,170	0.0009	0.04	0.08	0.08	BCHI-SUT	BSTG-RWLK
R130	4,810	0.0010	0.04	0.08	0.08	BSTG-RWLK	MBRK-HGTY-PLYM
R140	3,280	0.0006	0.04	0.08	0.08	MBRK-HGTY-PLYM	BRES-KIRK
R150	2,280	0.0004	0.04	0.08	0.08	BRES-KIRK	WISC
R160	4,560	0.0004	0.04	0.08	0.08	WISC	OUTLET

Thirty reaches covering approximately 85 miles of the Calaveras River and Mormon Slough stream systems are included in the PBI Model.

5.3.7. Loss Rates

As discussed in Section 5.2.1, the SJAFCA HEC-1 model used the SCS Curve Number method to calculate loss rates. The PBI Model differs from the 1998 SJAFCA HEC-1 model in that it uses the initial and constant loss rate method to model subbasin losses.

The Natural Resources Conservation Service (NRCS) has classified all soils into four hydrologic soil groups (A, B, C, and D) according to their infiltration rates[15]:

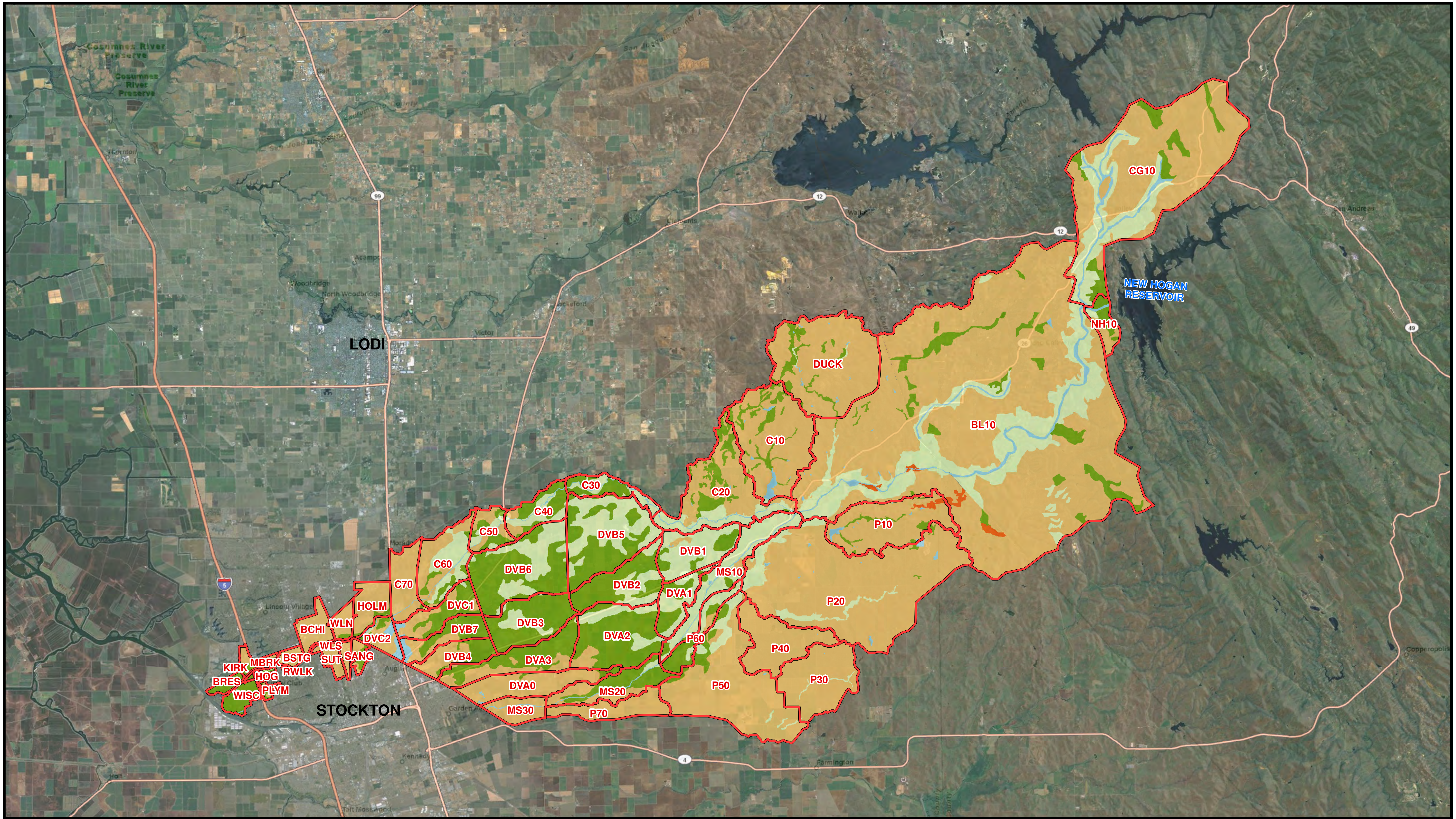
Table 5- 4. NRCS hydrologic soil groups.

Hydrologic Soil Group	Loss Rate Range [in/hr]	PBI's Assumed Loss Rate[a] [in/hr]
A	> 0.30	0.35
B	0.15 - 0.30	0.2
C	0.05 - 0.15	0.1
D	0.00 - 0.05	0.025

[a]This loss rate value was assigned to each soil group for initial calculations of composite loss rates. The calculated composite loss rates were then adjusted during the calibration process.

GIS soils data was obtained from the NRCS[13] and used to determine the proportional coverage of soil groups within Calaveras River subbasins (Figure 5- 7). NRCS GIS soils data was not available for Calaveras County. Soils data for this part of the study area was obtained from the Calaveras County Soil-Vegetation Survey[14]. A weighted average of loss rates was then calculated for each subbasin and adjusted during the calibration process (See Section 5.4). After the calibration adjustment, subbasin loss rates range from 0.021 inches per hour to 0.158 inches per hour as shown in Attachment 5-C.

EM 1110-2-1417[18] recommends that initial losses are set between 0.5-1.5 inches for agricultural areas. Initial losses were set to 0.5 inches for all agricultural/rural subbasins in the foothills and to 1.5 inches for agricultural/rural subbasins in the valley. For urban subbasins, initial losses were set to 0.2 inches also based on guidelines listed in *EM 1110-2-1417*.



▭ Subbasin Boundary

🟧 Group A

▭ Group B

🟩 Group C

🟨 Group D

🟦 Water/Other

N

0 1.5 3 Miles

1 inch = 3 miles

SEPTEMBER 21, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

CALAVERAS RIVER WATERSHED SOILS MAP

FIGURE

5-7

5.3.8. Impervious Percentages

Impervious percentages were assigned based on the extent of urbanization within each subbasin. Aerial photos including those contained within 2010 LiDAR datasets[4] were used to assess existing urbanization in the Calaveras River watershed. Subbasins were classified into several categories with assigned impervious percentages as shown in Table 5- 5. The impervious percentages corresponding to each land use type were selected with the guidance of the San Joaquin County *Hydrology Manual*[10].

Table 5- 5. Land use types and their corresponding impervious percentages.

Land Use Type	Impervious Percentage
Agricultural /Open Space	2%
Agricultural with Rural Residential Development	5%
Fully Developed Residential	60%

5.4. MODEL CALIBRATION

HDR's Calibration of the 1998 SJAFCA HEC-1 model included calculating the flow per square mile for the 1/100 AEP event at the Duck Creek near Farmington gage and comparing it to the modeled flow per square mile coming from the foothill subbasins that feed into Potter Creek. Adjustments were made to basin 'n' values such that the 1/100 AEP rainfall event would produce the 1/100 AEP streamflow event.

The PBI Model was calibrated to an observed rainfall-runoff event using gaged data retrieved from the California Data Exchange Center (CDEC)[24]. The three gages shown in Figure 5- 2 were used for the calibration: the Perry Ranch (PRY) gage was used for its rainfall data, the New Hogan Lake (NHG) gage was used for its reservoir outflow data, and the Mormon Slough at Bellota (MRS) gage was used for its flow records.

The storm selected to calibrate the PBI Model was one of the largest events recorded by the MRS gage. The rainfall event took place between March 25, 2006 and April 16, 2006 (23-day duration) and totaled 7.9 inches.

The MRS gage location corresponds to Model Element BL15. During the calibration process, constant loss rates were adjusted to match the PBI Model's hydrograph at Model Element BL15 with observed streamflow records from the MRS gage. Constant loss rates were initially calculated based on the makeup of soils in each subbasin (see Section 5.3.7). The loss rates were then adjusted by a factor of 0.85 during the calibration process.

Figure 5- 8 shows the observed New Hogan outflow during the calibration storm event. The results of the calibration are shown in Figure 5- 9.

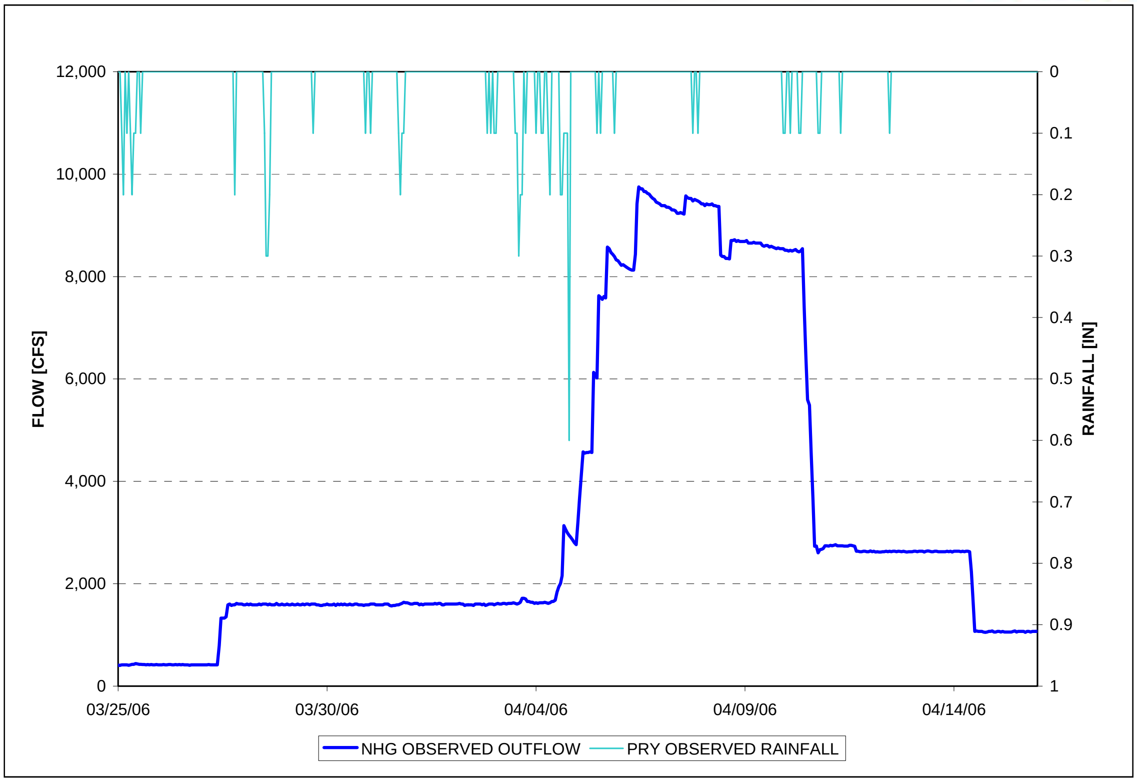


Figure 5- 8. Observed New Hogan Outflow During the Calibration Storm Event.

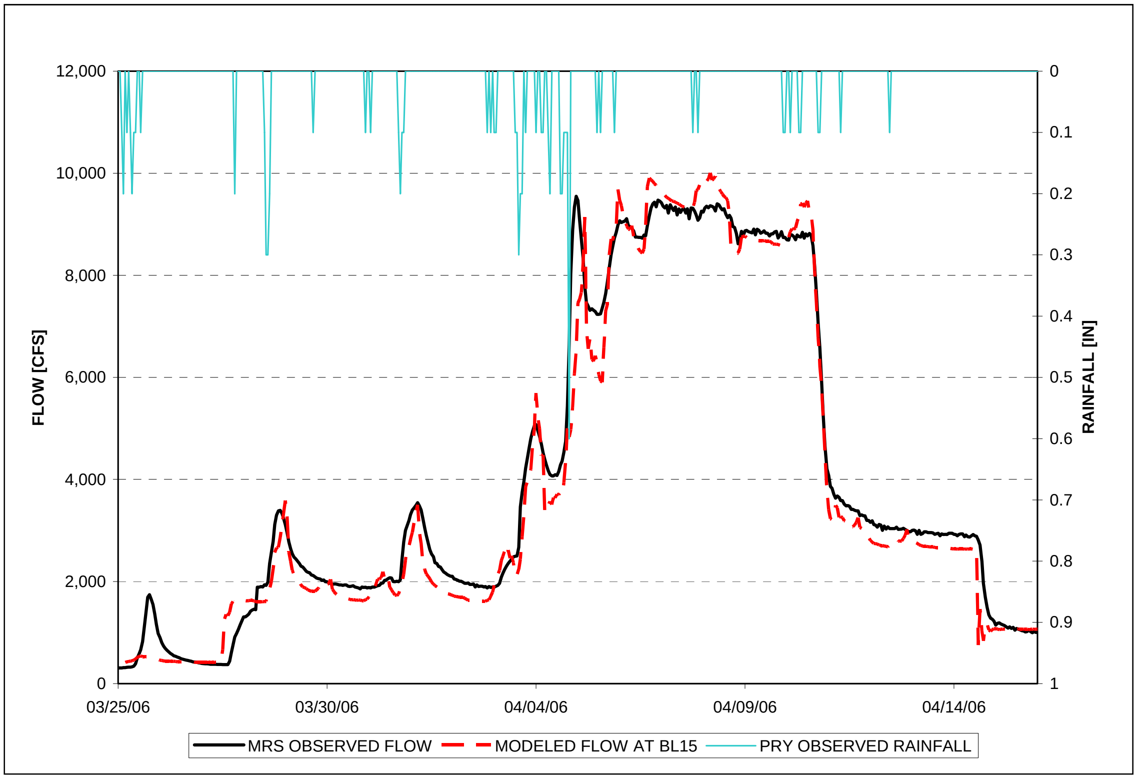


Figure 5- 9. Observed versus Modeled Flow at Bellota for the Calibration Storm Event.

At the onset of the storm, the initial runoff response is not picked up by the HEC-HMS model. This is due to the initial losses (see Section 5.3.7) assigned to the subbasins upstream of the MRS gage. These subbasins are largely undeveloped with little impervious area and therefore the soils capture the initial rainfall. Although this runoff response could be captured by decreasing the initial losses, initial loss was held at their assigned values based on the ranges suggested in *EM-1417*[18]. Furthermore, the emphasis of the hydrologic analysis is on peak event estimation which is relatively insensitive to initial loss assumptions.

5.5. DEVELOPMENT CONDITIONS

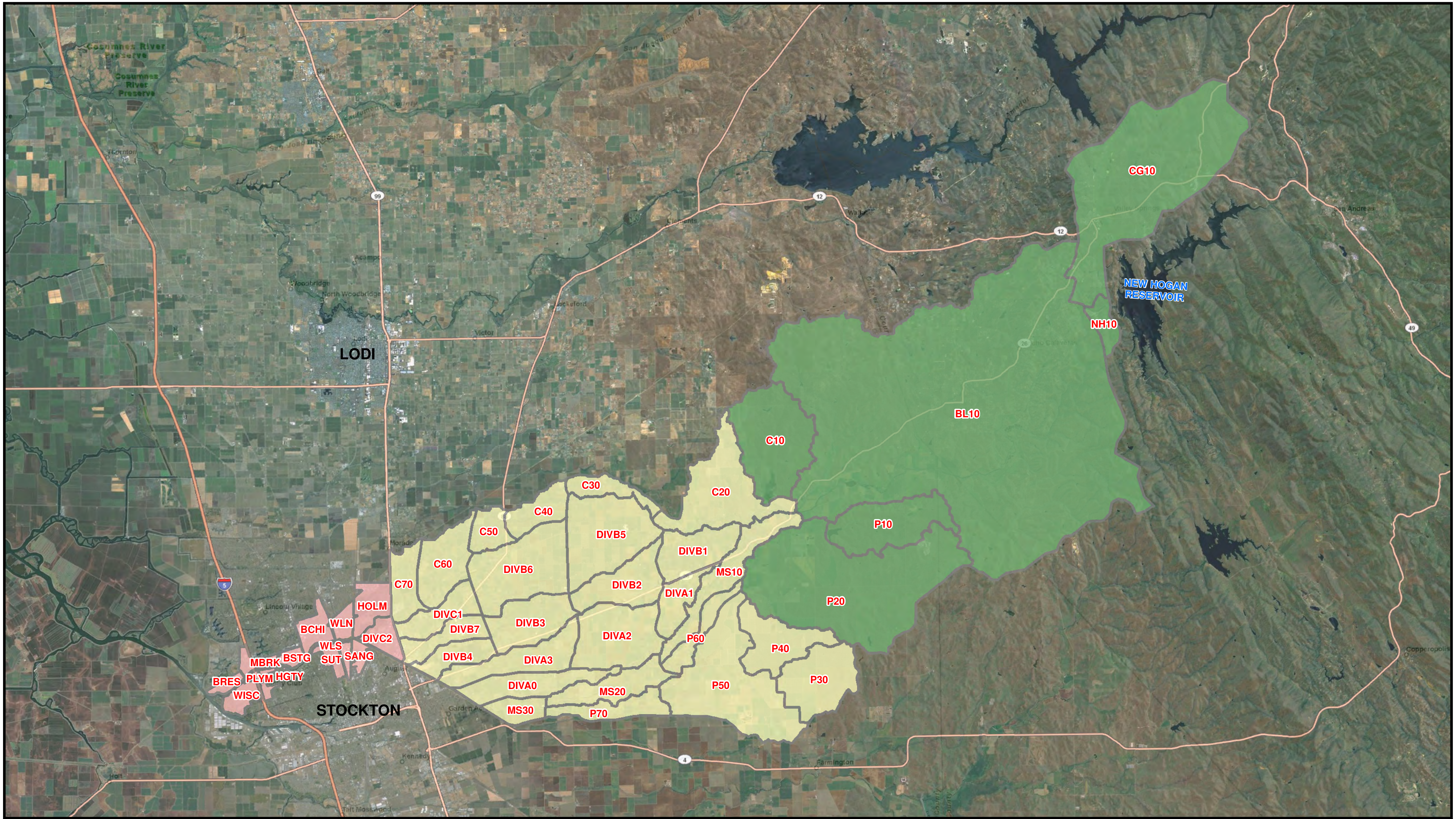
5.5.1. Existing Conditions

An ‘Existing Conditions’ model run was performed to evaluate peak flows given current (2010) land use and hydrologic conditions within the Calaveras watershed. Subbasin S-graphs, ‘n’ values, and impervious percentages were set according to current land cover conditions using field knowledge supplemented by aerial photos.

In general, the upstream watershed consists of natural or agricultural land whereas the lower portions of Calaveras watershed (below the confluence with the Diverting Canal) are developed areas in and around the city of Stockton. Figure 5- 10 displays the development conditions and S-graphs assigned to each subbasin. A summary table of the subbasin characteristics used for ‘Existing Conditions’ model runs is provided in Attachment 5-B.

5.1.1. Future-Without-Project Conditions

A ‘Future-Without-Project Conditions’ model run was considered to evaluate peak flows for future (2070) land use and hydrologic conditions within the Calaveras River watershed. However, the City of Stockton 2035 General Plan[12] and the San Joaquin County General Plan[17] show that land use remains unchanged from the ‘Existing Conditions’ model. Because of this, the ‘Future-Without-Project’ model simulations will be identical to ‘Existing Conditions’ simulations.



🟩 Foothill S-Graph

🟨 Valley Undeveloped S-Graph

🟥 Valley Developed S-Graph

N

0 1.5 3 Miles

1 inch = 3 miles

SEPTEMBER 21, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

EXISTING DEVELOPMENT CONDITIONS FOR CALAVERAS RIVER WATERSHED

FIGURE

5-10

5.2. STORM CENTERINGS

Four storm centerings were analyzed for the Calaveras River watershed:

- The *Above New Hogan* centering was analyzed to stress both New Hogan Dam and the watershed below.

- The *Bellota* centering was analyzed to stress the unregulated portion of the watershed directly below the dam.

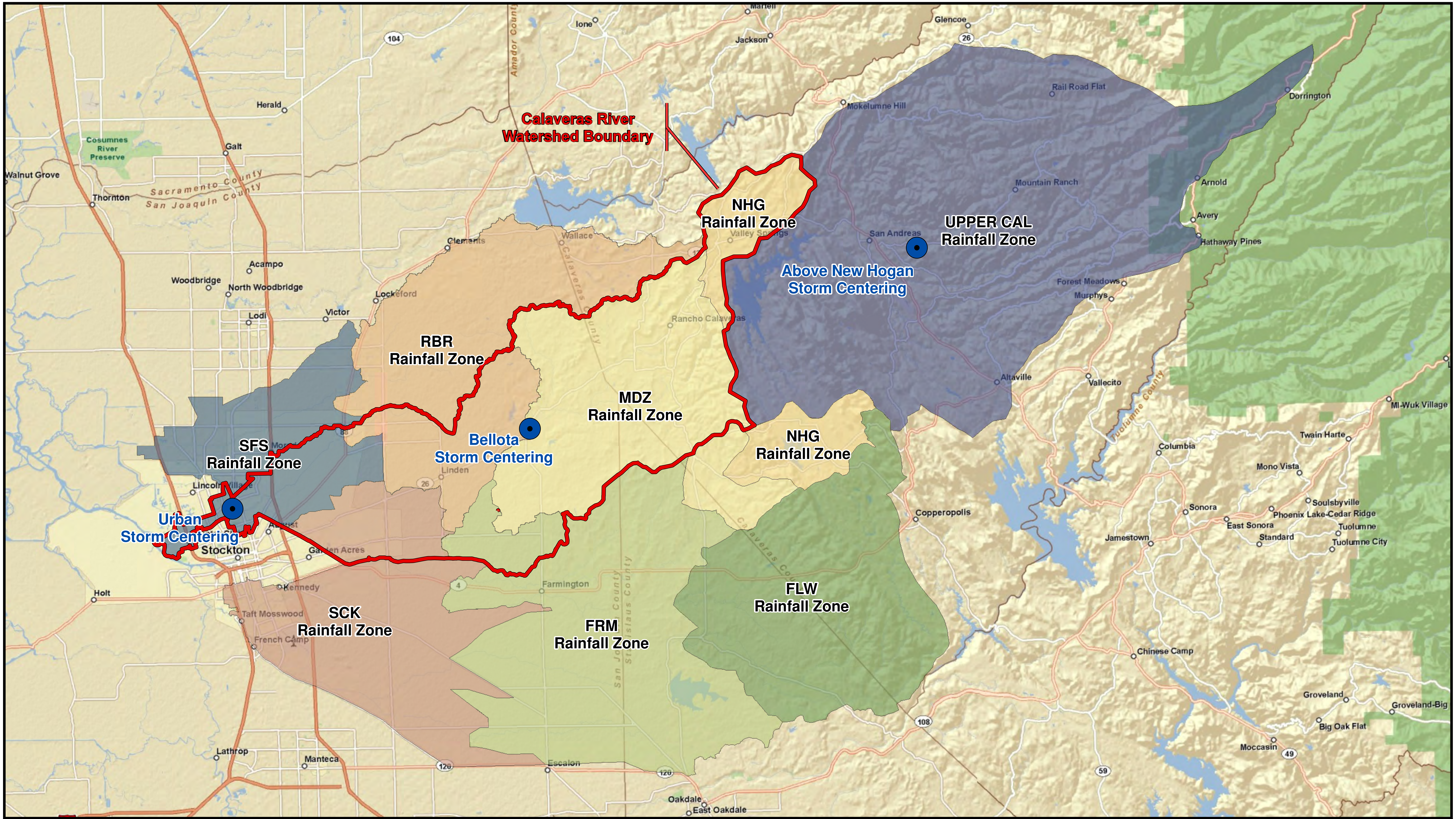
- The *"Average"* centering took the average of the *Above New Hogan* and *Bellota* area reduction factors to come up with rainfall depths.

  Reservoir flows from the Ford analysis were reported for an "Average" storm centering and are most applicable for this scenario. The "Average" storm centering scenario is considered the LSJRFS design storm scenario for the Calaveras River production runs and therefore are the only flows reported in Table 5- 7.

- The *Urban* centering was analyzed for interior drainage purposes and is directly centered over the urban areas in the lower watershed.

Eight design storms with the 1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, and 1/500 AEP events will be produced for each centering. This selection of design storms provides a wide range of scenarios that can be used for planning purposes.

Calculated area reduction factors and resulting area-reduced rainfall depths for each rainfall zone are provided in Attachment 5-D for all frequency-duration-storm centering combinations.



N

0 5 Miles

1 : 300,000

JULY 7, 2011

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

CALAVERAS RIVER WATERSHED STORM CENTERINGS

FIGURE

5-11

5.3. MODEL SIMULATIONS

Calaveras River production runs include 32 scenarios with unique combinations of development conditions, storm frequencies, and storm centerings.

Table 5- 6. Calaveras River production run scenarios.

Development Conditions	Storm Centerings	AEP Events
Existing Conditions	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Bellota	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	"Average"	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Above New Hogan	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500

5.3.1. Summary of Results

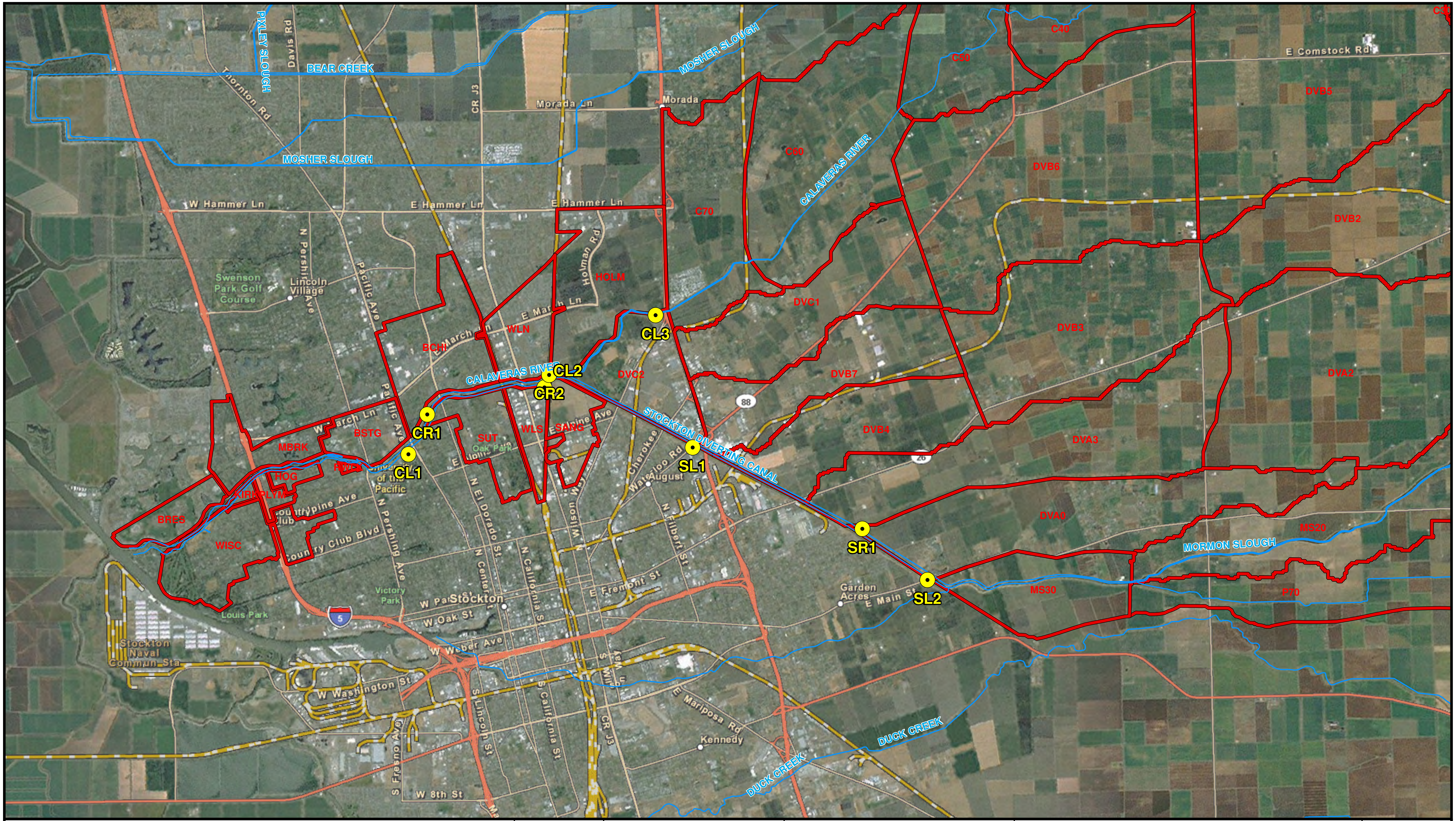
Peak flow results were extracted from HEC-HMS at each LSJRFS index point. Locations of LSJRFS index points within the Calaveras River watershed are shown in Figure 5- 12. Table 5- 7 summarizes peak flows from the “Average” storm centering production runs.

Reservoir flows from the Ford analysis were reported for an “Average” storm centering and are most applicable for this scenario. The “Average” storm centering scenario is considered the LSJRFS design storm scenario for the Calaveras River production runs and therefore are the only flows reported in Table 5- 7. Flows produced from this storm centering are to be used moving forward with the LSJRFS.

5.3.2. Uncertainty Parameters

For the purposes of the LSJRFS, uncertainty parameters for each flow-frequency dataset can be estimated within HEC-FDA during the project’s economic analysis. HEC-FDA defines uncertainty in terms of confidence intervals or standard deviations given inputs of flow-frequency data (provided in Table 5- 7) and an equivalent record length.

The equivalent record length is an estimate of the overall “worth” or “quality” of the flow-frequency function, expressed as the number of years-of-record[19]. For probability functions derived at ungaged locations using model or other data, the equivalent record length is based on a judgment of the quality of that model or data. EM 1110-2-1619[20] provides guidelines for assigning equivalent record lengths and estimates that a rainfall-runoff model calibrated to an observed event at a short-interval runoff gage has an equivalent record length of 20-30 years.



🟥 Subshed Boundary

● LSJRFS Index Point

0 0.25 0.5 1 Miles

1 inch = 1 mile

JUNE 20, 2012

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

CALAVERAS RIVER WATERSHED INDEX POINTS

FIGURE

5-12

Table 5-7. Peak Flow Results for Calaveras River - Existing and Future Conditions [cfs] [1]

LSJRFS Index Point ID	Description	Average Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
--	Calaveras R. at Duncan Rd.	110	230	300	410	500	580	670	780
CL3	Calaveras River near Hwy 99	110	230	300	440	530	620	720	810
--	Mormon Slough at Bellota[2]	3,520	9,520	9,390	10,320	12,500	12,500	12,500	16,000
--	Mormon Slough at Potter A Confl.	4,150	10,150	10,630	12,130	14,200	14,940	15,280	19,460
SL2	Mormon Slough at Diverting Canal	4,150	10,150	10,620	12,140	14,210	14,960	15,320	19,510
SR1	Diverting Canal u/s of Hwy 26	4,150	10,150	10,630	12,150	14,220	14,970	15,340	19,530
SL1	Diverting Canal near Hwy 99	4,150	10,150	10,670	12,230	14,320	15,070	15,440	19,620
CR2 & CL2	Calaveras River at Diverting Canal Confl.	3,810	9,620	10,050	12,530	13,670	15,650	16,110	20,230
CR1	Calaveras R. d/s of El Dorado St.	3,700	9,660	9,780	12,520	13,320	15,610	16,100	20,190
CL1	Calaveras R. d/s of Pacific Ave.	3,700	9,660	9,780	12,520	13,320	15,610	16,100	20,190
D4 & D5	Calaveras R. u/s of San Joaquin R. Confl.	3,560	9,520	9,760	12,390	12,780	15,600	16,100	20,160

[1] There were no changes from Existing to Future conditions, therefore only one results table is shown.

[2] Input hydrographs at Bellota provided to PBI by USACE on 2/1/12.

6.0 FRENCH CAMP SLOUGH HEC-HMS MODELING

6.1. GENERAL

6.1.1. Location

The French Camp Slough watershed is located near the city of Stockton in San Joaquin County, California (Figure 6- 1). The watershed runs east from the city of Stockton into the Sierra Nevada foothills in Calaveras County. It achieves maximum elevations of 2,100 feet and includes a total area of 430 square miles. It then descends through moderate slopes to the lower portion of the watershed which lies at sea-level. None of the watershed experiences snowfall; all floods are rainfall-induced.

The HEC-HMS model described in this memorandum includes the Duck Creek, Lone Tree Creek, Temple Creek, Rock Creek, Webb Creek, Littlejohns Creek, and the French Camp Slough systems and discharges to the San Joaquin River to the west of Interstate-5.

6.1.2. Topography

The HEC-HMS model utilized for this study is titled the PBI French Camp Slough Model (PBI Model) which is georeferenced to the NAD 1983 State Plane California Coordinate System Zone III (U.S. Survey Feet). Vertical elevations are reported in the NAVD 1988 datum. Topography used for model development included United States Geological Survey (USGS) 30-meter Digital Elevation Models (DEMs)[3]. Where available, Department of Water Resources (DWR) LiDAR data[4] was also used to confirm subbasin boundaries.

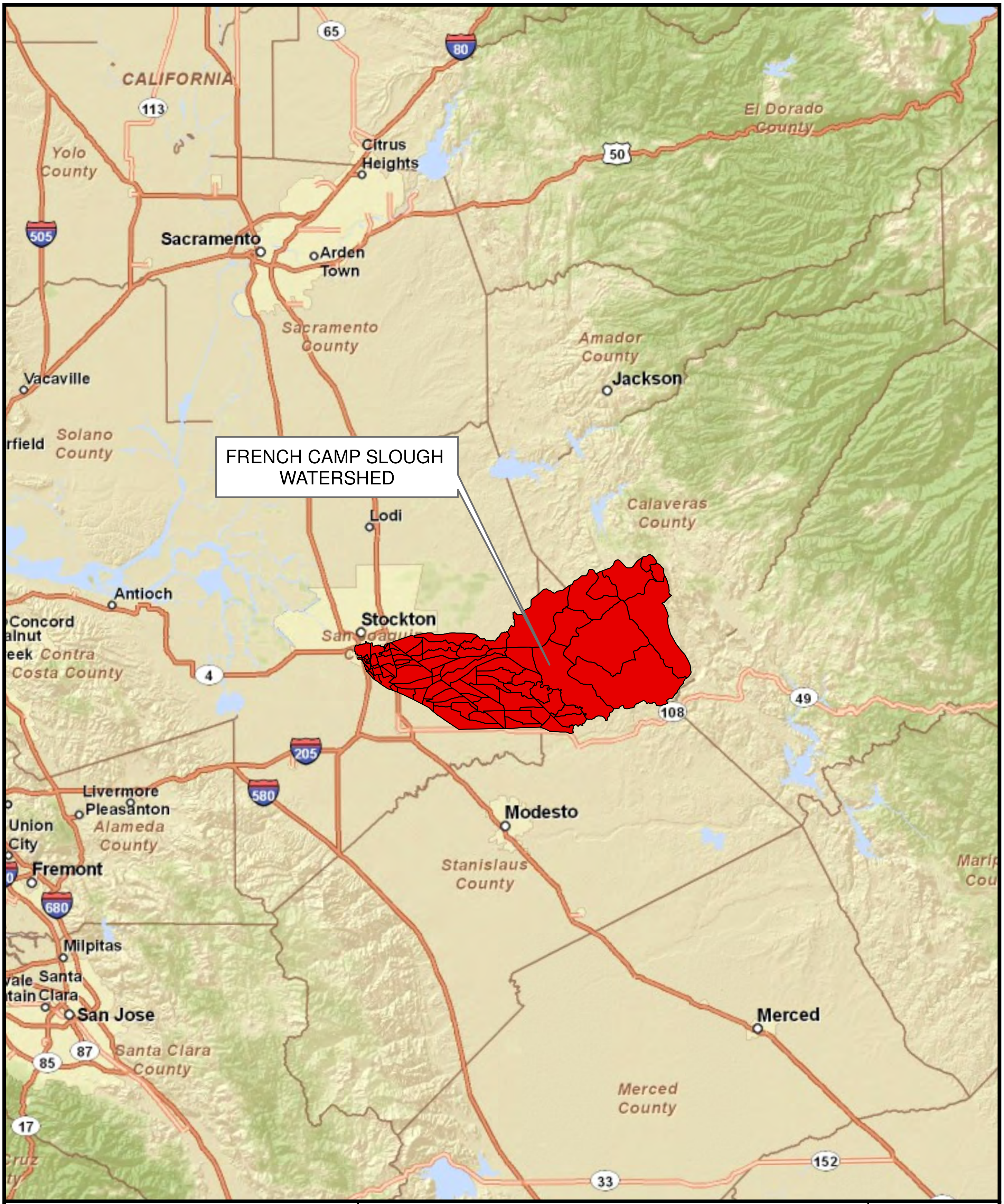
6.2. MODEL DEVELOPMENT

The PBI model was developed using HEC-HMS version 3.4[5] and HEC-GeoHMS version 4.2[6]. A summary of the tasks performed are listed below:

1. A previous HEC-HMS model used in the *Conditional Letter of Map Revision (CLOMR) for the Tidewater Crossing Flood Control Project*[25] (Tidewater Model) provided the basis for the PBI Model (See Section 6.2.1).

2. Additional subbasins were added to the Tidewater model. Eleven (11) subbasins on Duck Creek and North Littlejohns Creek were imported from a previous HEC-1 model constructed as part of the *Mariposa Lakes Off-Site Regional Hydrologic Investigation*[26] (Mariposa Lakes Model) (See Section 6.2.2). Eighteen (18) additional subbasins extended the PBI Model to French Camp Slough's outlet on the San Joaquin River.

3. Pump stations were coded into the PBI Model based on design pumping rates



PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630
Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

VICINITY MAP OF THE FRENCH CAMP SLOUGH STUDY AREA

FIGURE

6-1

provided by City of Stockton records[7] (See Section 6.3.2).

4. Diversions and channel routing parameters were coded for the added subbasins of the PBI Model (See Sections 6.3.4 and 6.3.6, respectively).

5. S-graphs and lag times were coded into the PBI Model (See Section 6.3.5).

6. Loss rates and impervious percentages were coded into the PBI Model (See Section 6.3.7 and Section 6.3.8).

7. The PBI Model was calibrated using historical rainfall and runoff data (See Section 6.4).

8. The PBI Model was set up to simulate both ‘Existing’ (see Section 6.5.1) and ‘Future-Without-Project’ (see Section 6.5.2) scenario runs.

6.2.1. Tidewater HEC-HMS Model

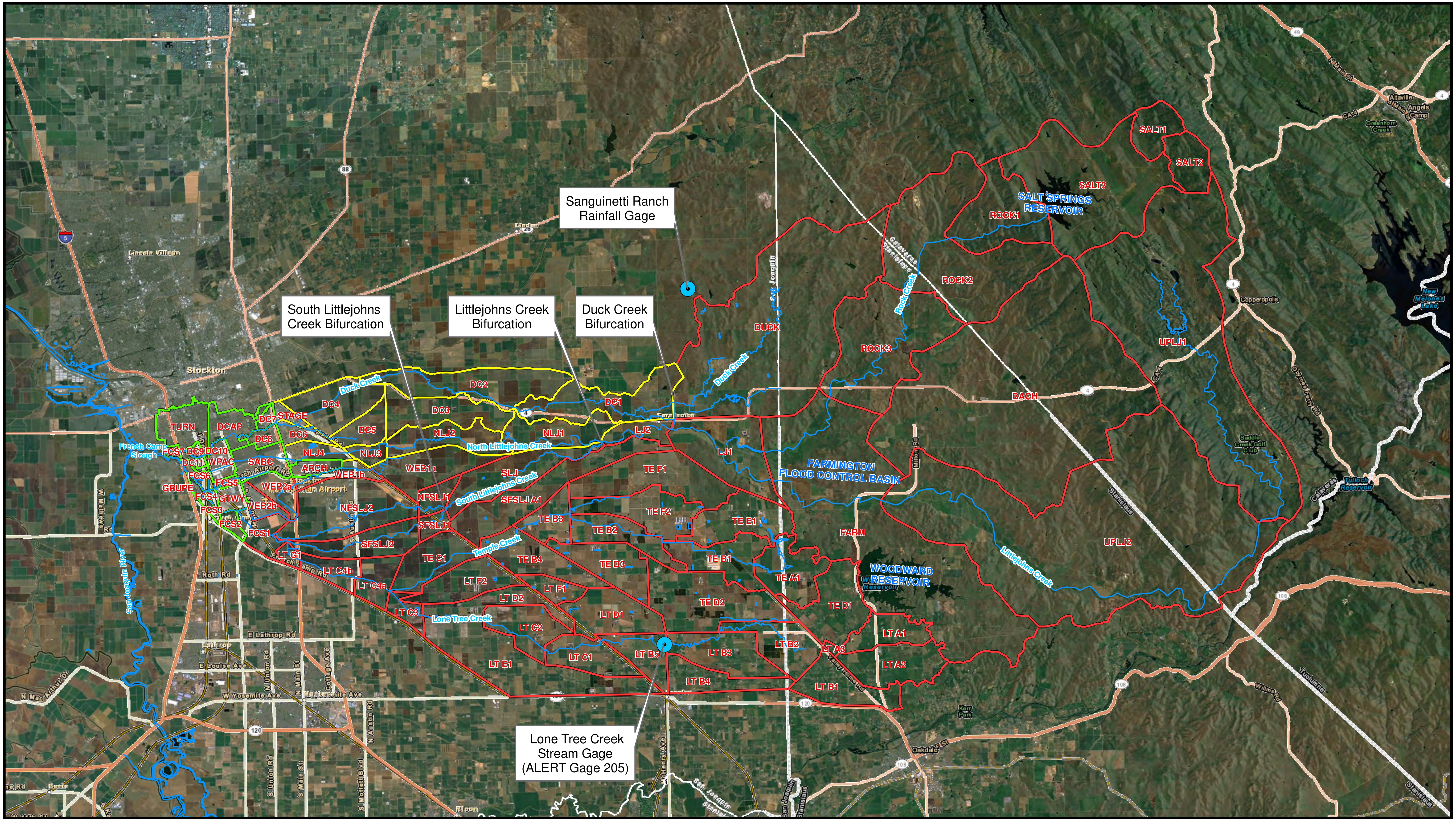
The PBI Model is an update and expansion of the HEC-HMS model developed in 2007 for the *Conditional Letter of Map Revision (CLOMR) for the Tidewater Crossing Flood Control Project*[25].

The 2007 Tidewater Model includes 55 subbasins which are coded with SCS Curve Numbers, San Joaquin County S-Graphs, and calculated lag times. Reach routing, diversions, and reservoir routing were also coded into the Tidewater Model based on field visits conducted by Domenichelli & Associates.

Calibration of the Tidewater Model used a January 1, 2006 storm which was estimated to be a 1/10 AEP event. This enabled observed high water marks on Lone Tree Creek to be calibrated to recorded rainfall data.

6.2.2. Mariposa Lakes HEC-1 Model

A HEC-1 model was previously developed in 2006 as part of the *Mariposa Lakes Off-Site Regional Hydrologic Investigation*[26]. PBI extracted 11 subbasins from the Mariposa Lakes HEC-1 Model and imported them into the Tidewater HEC-HMS model. The imported Mariposa Lakes model elements are located on Duck Creek and North Littlejohns Creek systems.



Sanguinetti Ranch Rainfall Gage

South Littlejohns Creek Bifurcation

Littlejohns Creek Bifurcation

Duck Creek Bifurcation

Lone Tree Creek Stream Gage (ALERT Gage 205)

🟥 Subbasins from the Tidewater Model

🟨 Subbasins from the Mariposa Lakes Model

🟩 Subbasins Added by PBI

N

0 1.5 3 Miles

1 inch = 3 miles

OCTOBER 19, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

FRENCH CAMP SLOUGH HEC-HMS SUBBASINS

FIGURE

6-2

6.3. MODEL FEATURES

The 2007 Tidewater HEC-HMS model was updated and expanded to form the PBI French Camp Slough Model. The PBI Model components are described in the following sections.

6.3.1. Subbasins

The PBI Model contains a total of 85 subbasins with drainage areas ranging from 0.04 square miles to 51.62 square miles and a total watershed area of approximately 430 square miles. Figure 6- 2 displays the subbasin boundaries used for the PBI Model.

As previously discussed, 55 of the PBI Model's subbasins come from the 2007 Tidewater Model. These subbasin boundaries include 385 square miles of tributary area and cover much of the Duck Creek, Lone Tree Creek, Temple Creek, Webb Creek, Rock Creek, Littlejohns Creek, and the French Camp Slough systems.

The calculated areas of several subbasins on Lone Tree Creek and Temple Creek were adjusted to account for parts of the subbasin that were considered to be isolated and not contributing to runoff. These adjustments were based on field investigations conducted for the Tidewater Model which determined that ponding in fields would occur and this ponded area would not contribute to the subbasins' modeled runoff. A summary of the adjusted subbasin areas is provided in Attachment 6-A.

Eleven (11) subbasins totaling 31 square miles were extracted from the Mariposa Lakes HEC-1 Model. These subbasins cover portions of the Duck Creek and North Littlejohns Creek systems.

Eighteen (18) subbasins totaling 12 square miles were added by PBI which extend the model boundaries to near French Camp Slough's outlet on the San Joaquin River. Subbasins to the west drain to the San Joaquin River. Many of these subbasin boundaries were based on the existing storm drain system and the City of Stockton's *Conceptual Storm Drain Master Plan*[11]. Many of these subbasins discharge to the main channels through various stormwater pump stations described in Section 3.3.2. Areas that are not drained through the storm sewer system are gravity-driven. USGS 30-meter DEM datasets[3] were used to identify local topography for delineating gravity-driven subbasins. Where available, DWR LiDAR[4] data was used to confirm subbasin boundaries.

The GIS horizontal coordinates for each subbasin were used to georeference model elements within the PBI HEC-HMS Model. The subbasin GIS shapefile was inserted into the PBI Model as a background map.

6.3.2. Pump Stations

Stormwater pump stations were included in the PBI Model to represent storm drain conveyance from developed subbasins to the main channels. There are nine (9) pump stations included in the PBI model with capacities assigned based on City of Stockton records[7].

Table 6-1 provides a summary of the City of Stockton pump stations included in the PBI Model.

Table 6- 1. Summary of French Camp Slough pump stations.

Pump Station Name	Contributing Subbasin(s)	Subbasin Area [sq. mi.]	Pump Station Status	Pump Station Capacity [cfs]	Pump Station Notes
Stockton Airport Business Center	SABC	1.80	Existing	66.8	4 @ 15.4 cfs 1 @ 5.2 cfs
Duck Creek & Airport Way	DCAP	1.79	Existing	114.8	1 @ 5.6 cfs 1 @ 49.0 cfs 1 @ 60.2
Arch Road	ARCH	0.74	Existing	10.8	2 @ 5.4 cfs
Clayton & Harvey	CLAY	0.10	Existing	9.8	2 @ 4.9 cfs
Grupe Business Park	GRUPE	0.20	Existing	120.7	1 @ 11.1 cfs 2 @ 54.8 cfs
Duck Creek & Stagecoach	STAGE	0.50	Existing	155.4	1 @ 5.6 cfs 2 @ 74.9 cfs
Walker Slough & Turnpike	TURN	2.33	Existing	116.9	1 @ 8.0 cfs 2 @ 37.4 cfs 1 @ 34.1 cfs
Western Pacific Industrial Park	WPIP	0.93	Existing	60.7	1 @ 2.7 cfs 2 @ 29.0 cfs
Airport Gateway	GTWY	0.77	Existing	22.3	1 @ 1.9 cfs 2 @ 10.2 cfs
PS-DC10	DC10	0.04	Future	9.5	Based on 0.37 cfs/acre
PS-DC11	DC11	0.23	Future	54.5	Based on 0.37 cfs/acre
PS-DC4	DC4	3.80	Future	899.8	Based on 0.37 cfs/acre
PS-DC5	DC5	1.68	Future	397.8	Based on 0.37 cfs/acre
PS-DC6	DC6	0.92	Future	217.9	Based on 0.37 cfs/acre
PS-DC7	DC7	0.32	Future	75.8	Based on 0.37 cfs/acre
PS-DC8	DC8	0.62	Future	146.8	Based on 0.37 cfs/acre
PS-DC9	DC9	0.17	Future	40.3	Based on 0.37 cfs/acre
PS-FCS1	FCS1	1.70	Future	402.6	Based on 0.37 cfs/acre
PS-FCS2	FCS2	0.46	Future	108.9	Based on 0.37 cfs/acre
PS-FCS3	FCS3	0.26	Future	61.6	Based on 0.37 cfs/acre
PS-FCS4	FCS4	0.20	Future	47.4	Based on 0.37 cfs/acre

con't Table 6- 1…

Pump Station Name	Contributing Subbasin(s)	Subbasin Area [sq. mi.]	Pump Station Status	Pump Station Capacity [cfs]	Pump Station Notes
PS-FCS5	FCS5	0.30	Future	71.0	Based on 0.37 cfs/acre
PS-FCS6	FCS6	0.38	Future	90.0	Based on 0.37 cfs/acre
PS-FCS7	FCS7	0.12	Future	28.4	Based on 0.37 cfs/acre
PS-LT C4b	LT C4b	1.19	Future	281.8	Based on 0.37 cfs/acre
PS-LT G1	LT G1	0.45	Future	106.6	Based on 0.37 cfs/acre
PS-NFSLJ2	NFSLJ2	6.78	Future	1605.5	Based on 0.37 cfs/acre
PS-NLJ3	NLJ3	0.75	Future	177.6	Based on 0.37 cfs/acre
PS-NLJ4	NLJ4	1.19	Future	281.8	Based on 0.37 cfs/acre
PS-SFSLJ2	SFSLJ2	3.30	Future	781.4	Based on 0.37 cfs/acre
PS-Web1b	Web1b	1.11	Future	262.8	Based on 0.37 cfs/acre
PS-Web2a	Web2a	1.42	Future	336.3	Based on 0.37 cfs/acre

Twenty-three pump stations were then added into the ‘Future-Without-Project Conditions’ model to represent subbasins that are expected to become developed according to the City of Stockton 2035 General Plan[12]. Pump capacities were assigned at a rate of 0.37 cfs per acre of tributary area. This rate is based on the average flow rates of existing pump stations within the City of Stockton’s systems and correlates to approximately 10-year peak flows.

6.3.3. Reservoirs

There are three main reservoirs in the PBI Model study area: Salt Springs Reservoir, Woodward Reservoir, and Farmington Flood Control Basin.

Salt Springs Reservoir

The Salt Springs Reservoir is a small reservoir that impounds flow on Rock Creek and is primarily used for recreation. A field study conducted for the Mariposa Lakes model confirmed that water simply spills over this small concrete dam structure when the reservoir is full[26]. Inflow roughly equals outflow and the hydraulic effects of this reservoir become negligible.

Woodward Reservoir

Woodward Reservoir is operated by South San Joaquin Irrigation District (SSJID) and releases water directly into a SSJID irrigation canal. The Tidewater model assumed that during the flood season (November-March) Woodward Reservoir's tributary area drains towards the Farmington Flood Control Basin and through Farmington Dam[25]. A telephone conversation with a SSJID engineer confirmed that Woodward Reservoir is an off-stream reservoir and any major releases are limited to the irrigation season (April-October). There is no spillway associated with this reservoir and any overtopping during the flood season would follow the natural topography of the land traveling through Simmons Slough and over towards Farmington Flood Control Basin.

Farmington Flood Control Basin

Farmington Reservoir is a large flood control basin located about 20 miles east of Stockton and impounds flow from both Rock Creek and Littlejohns Creek. The dam itself is approximately 7,800 feet long and 58 feet high with two outlets controlled by slide gates.

David Ford Consulting Engineers (DFCE) completed a separate reservoir operations analysis for Farmington Reservoir as part of the LSJRFS[23]. This analysis was later amended by USACE as documented in their *Draft Memorandum for Record: Lower San Joaquin River Feasibility Study, Bellota and Farmington Regulated Flow Hydrographs* (07 FEB 2012)[22]. One of the final deliverables from this study was regulated hydrographs at the Farmington control point for each of the 8 LSJRFS AEP storm events. These hydrographs include all flows coming out of Farmington Dam along with all local flows between the Town of Farmington and Farmington Dam. These regulated flow hydrographs were coded into the PBI HEC-HMS model as time-series discharge gages and supersede all HEC-HMS inflow that comes from above the Town of Farmington.

The Ford report and the USACE amendment should be referenced for any details regarding the reservoir operations study.

The table on the following page was taken from the USACE amendment and shows the flow-frequency relationship for Littlejohn Creek at the Farmington control point.

Table 6- 2. Flow-frequency at Farmington Reservoir (from Ford Report[23])

Regulated Peak Flow values and associated volumes:
Littlejohn Creek at Farmington

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration)			
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)
0.5	1,400	1,206	1,041	797	550
0.2	2,170	1,870	1,796	1,614	1,138
0.1	2,368	2,018	1,921	1,756	1,426
0.04	2,615	2,089	2,002	1,839	1,736
0.02	3,744	3,486	2,070	1,900	1,843
0.01	9,900	8,600	7,400	5,400	3,800
0.005	12,900	12,000	10,000	7,400	4,400
0.002	16,600	15,200	12,000	8,600	5,200

1) Revised to reflect graphical fit of observed data from Oct1949 to Dec2011 for the 0.5 to the 0.02 AEP. The 0.01 to 0.002 AEP events are from the revised flow transform and regulated flow-freq curve. The volumes were computed from the regulated peak to volume transforms in the Ford report.

Miscellaneous Reservoir Elements

Along with the three reservoirs discussed, several additional reservoir elements were included in the PBI Model to represent flow restrictions as channels encounter road and railway crossings. For development of the Tidewater Model, measurements were taken of culvert and bridge geometry. Rating curves estimating the hydraulic performance of many crossings in the mid- and upper watershed were determined by entering measured geometries into HEC-RAS and simulating a range of flows through the structures. Hydraulic calculations associated with assigning reservoir storage/discharge relationships were performed for the 2007 Tidewater Model and are included in Attachment 6-B. The computed flow relationships were incorporated into reservoir elements to represent flow impedance at the selected road and railway crossings.

Some reservoirs include elevation-storage functions in which the elevations were reported in the NGVD29 coordinate system. To stay consistent with current conventions, the elevations were converted to the NAVD88 coordinate system using CORPSCON v6.0.1[27] software. A summary of the conversion is provided in Attachment 6-C.

6.3.4. Diversions

Diversions in HEC-HMS are coded to simulate either manmade diversions or topographic flow splits. Twenty-five (25) diversions are included in the PBI Model were imported from the Tidewater and Mariposa Lakes hydrologic models.

There are three (3) diversions used to represent channel bifurcations in the PBI Model. Channel bifurcations occur on Duck Creek, Littlejohns Creek, and South Littlejohns Creek as seen in Figure 6- 2. Coding for the Duck Creek and South Littlejohns Creek diversions was taken from the Tidewater HEC-HMS model whereas the Littlejohns Creek diversion coding was taken from the Mariposa Lakes HEC-1 model. The Duck Creek bifurcation has a structure to control the flow diverted to Littlejohns Creek whereas diversion flows for the Littlejohns Creek and South Littlejohns Creek bifurcations were proportionally based on channel geometries.

The remaining twenty-two (22) diversions included in the PBI Model are used to represent topographic flow splits at road and railway crossings and were imported from the Tidewater HEC-HMS model. As mentioned in Section 6.3.3, road and railway crossings were modeled using reservoir elements. For the cases where floodwaters in the overbank areas are unable to return to the main channel due to berms and other impedances, a diversion element was utilized to take this excess water and route it through an additional reservoir element. This reservoir element then routes the excess flow appropriately through small pipes or overland surfaces as it eventually returns back to the main channel.

6.3.5. S-graphs and Lag Times

The PBI Model assigns a Foothill, Valley Undeveloped, or Valley Developed S-graph to each subbasin based on its location. S-graph data points were obtained from the San Joaquin County Hydrology Manual[10]. The S-graphs were developed based on rainfall-runoff data from Southern California catchments considered to be hydrologically similar to the local catchments. The following figures show the time versus discharge relationship for each S-graph.

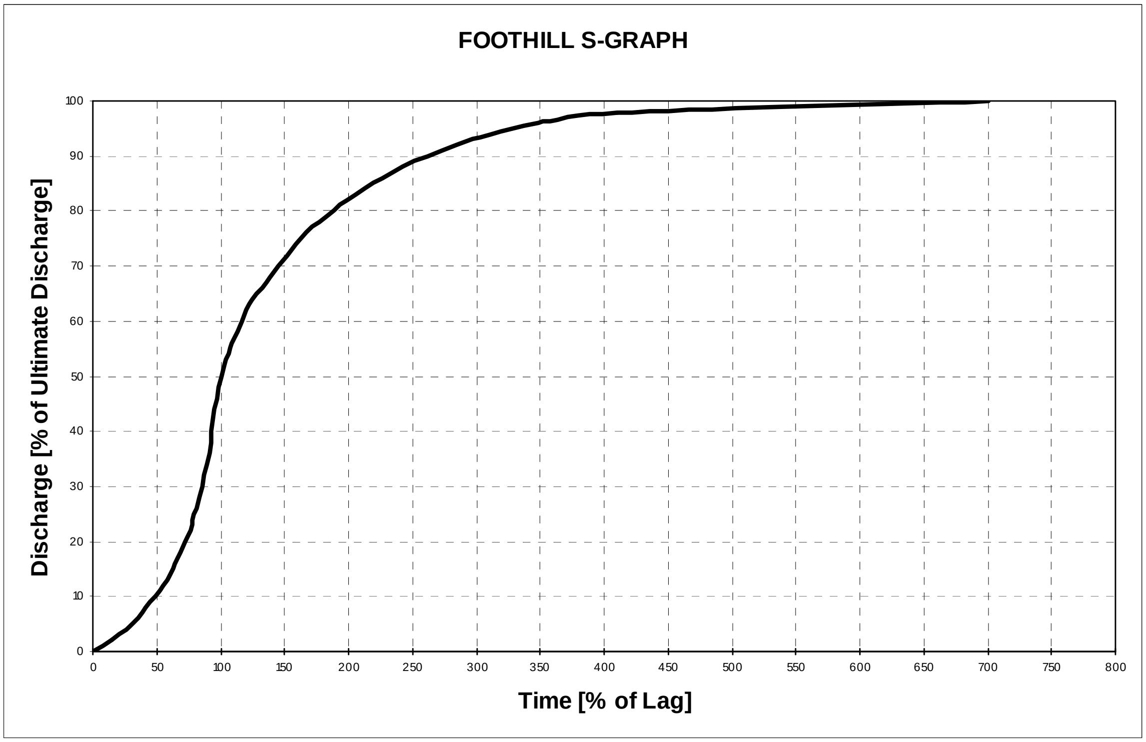


Figure 6- 3. San Joaquin County Foothill S-graph

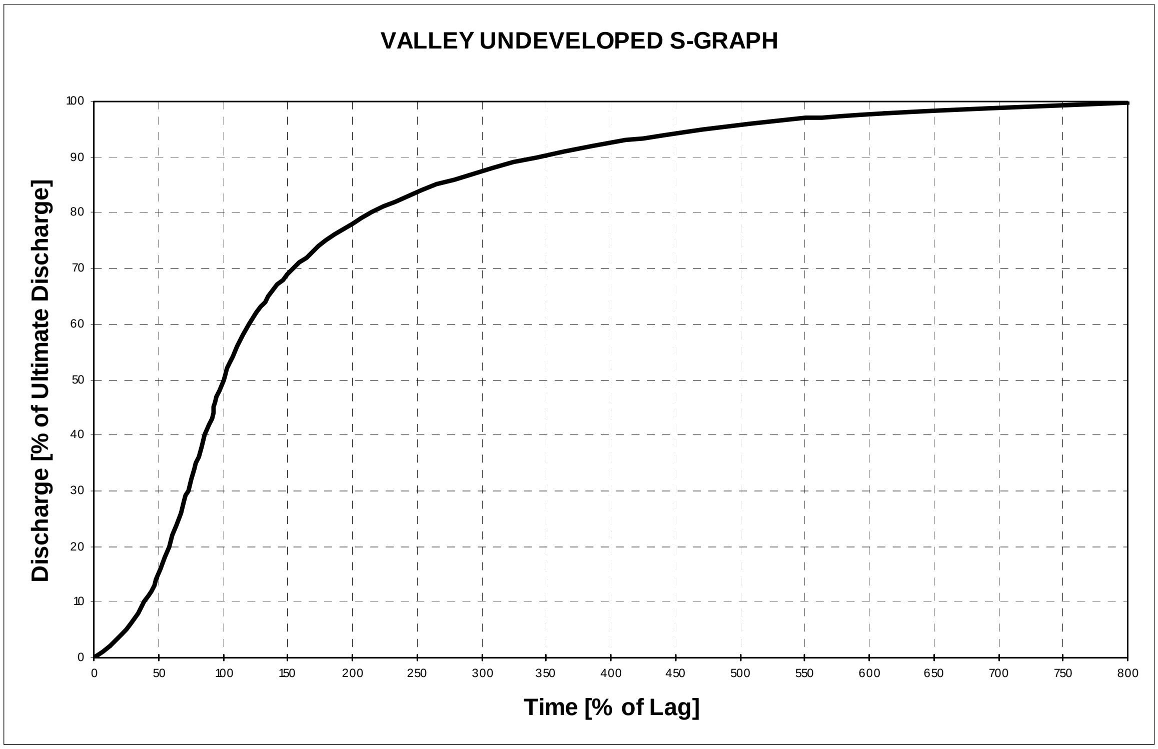


Figure 6- 4. San Joaquin County Valley Undeveloped S-graph

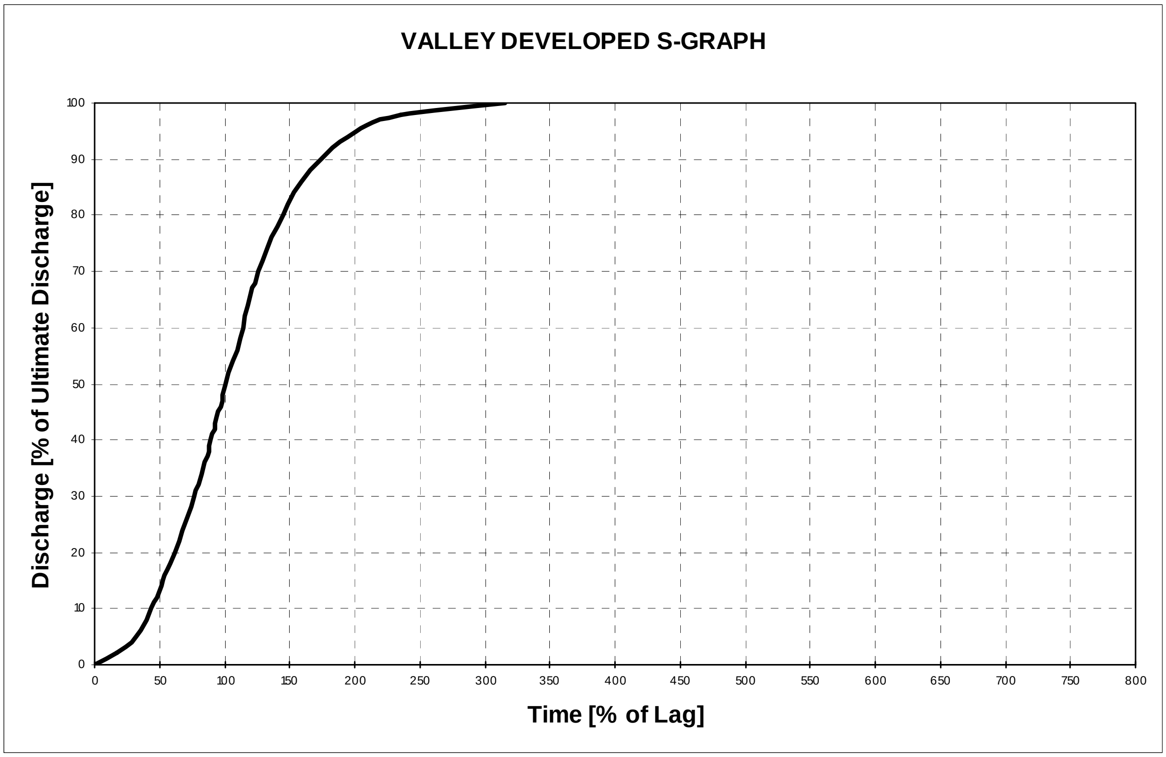


Figure 6- 5. San Joaquin County Valley Developed S-graph

Basin lag times were calculated according to guidelines set forth in the San Joaquin County Hydrology Manual[10]. The following equation was used:

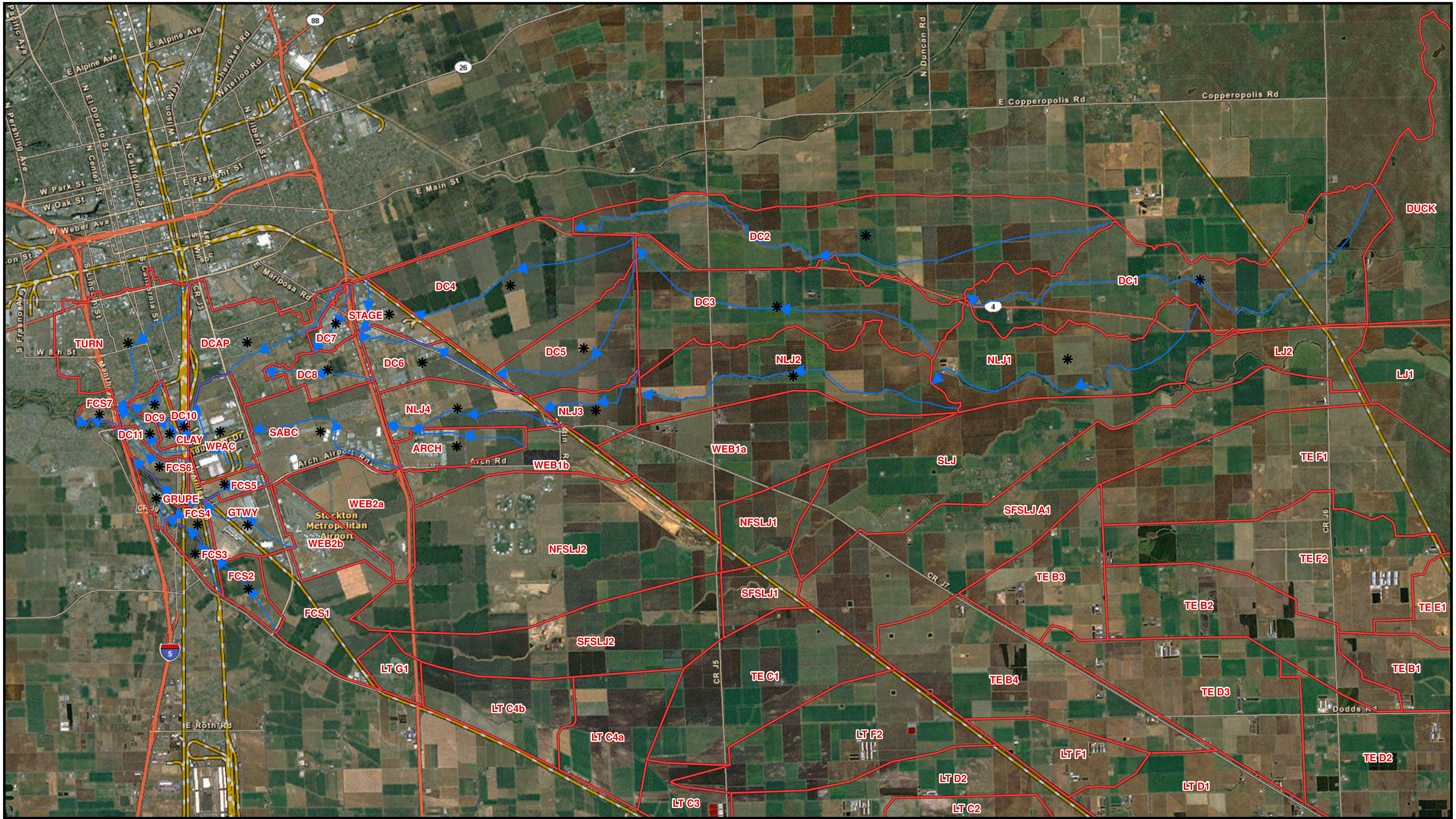
$$Lg = 24n(L \cdot L_C/S^{0.50})^{0.38}$$

where:

Lg = Lag time [hours]
n = Average basin factor estimated using Figure E-2 in the San Joaquin County Hydrology Manual
L = Length of longest watercourse [miles]
L_C = Length of longest watercourse measured to the centroid of the basin [miles]
S = Overall slope of longest watercourse [feet/mile]

For the 55 subbasins that originated from the Tidewater Model (see Figure 6- 2), L, L_C, S and n values were determined by Domenichelli & Associates[25].

For the remaining 30 subbasins, lag time parameters were calculated by PBI using ArcGIS software. Flowpaths identified for these calculations are shown in Figure 6- 6. S-graph assignments and lag time calculations for each subbasin are provided in Attachment 6-D and Attachment 6-E for ‘Existing’ and ‘Future-Without-Project’ Conditions, respectively.



□ Subbasin Boundary

* Subbasin Centroid

→ Subbasin Flowpath

N

0 0.5 1 Miles

1 : 75,000

OCTOBER 19, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

FRENCH CAMP SLOUGH
SUBBASIN FLOWPATHS CALCULATED BY PBI

FIGURE

6-6

6.3.6. Channel Routing

The PBI Model includes 72 routing reaches to represent attenuation of flood waves within channels. Forty-six (46) routing reaches were imported from the Tidewater HEC-HMS model. Muskingum, Muskingum-Cunge, Kinematic Wave, Lag, and Modified Puls routing methods were all implemented for these reaches depending on conditions observed in the field during development of the Tidewater model.

Ten (10) routing reaches were imported from the Mariposa Lakes HEC-1 model. These reaches use the Muskingum-Cunge routing method with channel parameters measured during development of the Mariposa Lakes model.

The remaining 16 reaches were added by PBI and used the Muskingum-Cunge routing method. Reach lengths and slopes were measured using ArcGIS software. Manning's n values were assigned based on recommendations made in the San Joaquin County Flood Insurance Study conducted by the Federal Emergency Management Agency (FEMA)[28]. Channel cross-sections were cut using detailed topographic data from DWR LiDAR dataset[4].

6.3.7. Loss Rates

Subbasins for the PBI Model utilize the initial and constant loss rate method in HEC-HMS to model subbasin losses.

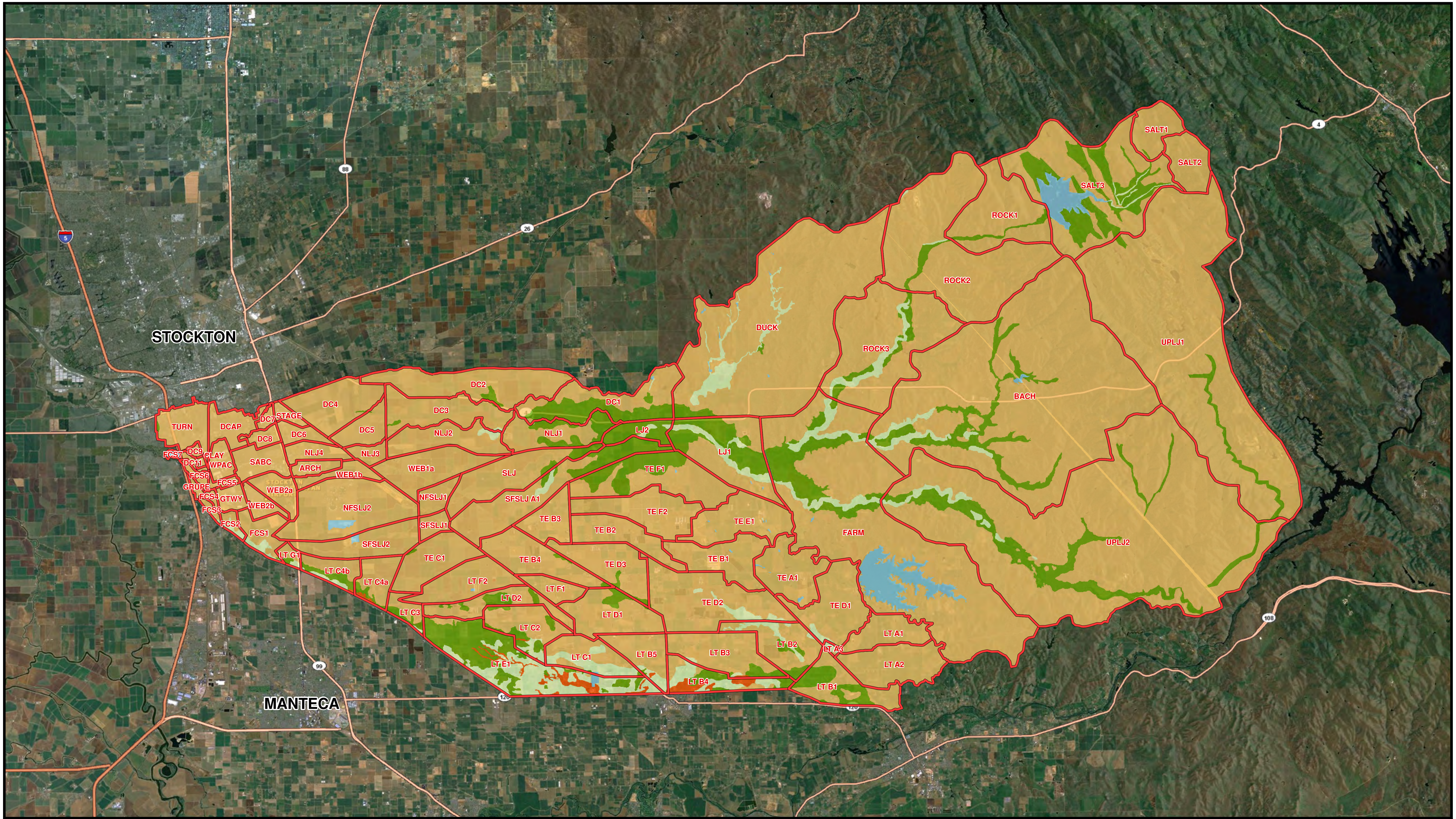
The Natural Resources Conservation Service (NRCS) has classified all soils into four hydrologic soil groups (A, B, C, and D) according to their infiltration rates[15]:

Table 6- 3. NRCS hydrologic soil groups.

Hydrologic Soil Group	Loss Rate Range [in/hr]	PBI's Assumed Loss Rate[a] [in/hr]
A	> 0.30	0.35
B	0.15 - 0.30	0.2
C	0.05 - 0.15	0.1
D	0.00 - 0.05	0.025

[a]This loss rate value was assigned to each soil group for initial calculations of composite loss rates. The calculated composite loss rates were then adjusted during the calibration process.

GIS soils data was obtained from the NRCS[13] and used to determine the proportional coverage of soil groups within French Camp Slough subbasins (Figure 6- 7). NRCS GIS soils data was not available for Calaveras County. Soils data for this part of the study area was obtained from the Calaveras County Soil-Vegetation Survey[14].



🟧 Group A
🟩 Group B
🟩 Group C
🟨 Group D
🟦 Water/Other

N

0 1.5 3 Miles

1 inch = 3 miles

OCTOBER 19, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

FRENCH CAMP SLOUGH
SOILS MAP

FIGURE

6-7

A weighted average of loss rates was then calculated for each subbasin and adjusted during the calibration process (See Section 6.4). After the calibration adjustment, subbasin loss rates range from 0.021 inches per hour to 0.144 inches per hour as shown in Attachment 6-F.

EM 1110-2-1417[18] recommends that initial losses are set between 0.5-1.5 inches for agricultural areas. Initial losses were set to 0.5 inches for all agricultural/rural subbasins in the foothills and to 1.5 inches for agricultural/rural subbasins in the valley. For urban subbasins, initial losses were set to 0.2 inches also based on guidelines listed in *EM 1110-2-1417*.

6.3.8. Impervious Percentages

Impervious percentages were assigned based on the extent of urbanization within each subbasin. Aerial photos including those contained within 2010 LiDAR datasets[4] were used to assess existing urbanization in the French Camp Slough watershed. The impervious percentages corresponding to each land use type were selected with the guidance of the San Joaquin County *Hydrology Manual*[10].

Table 6- 4. Land use types and their corresponding impervious percentages.

Land Use Type	Impervious Percentage
Agricultural/Open Space	2%
Agricultural with Rural Residential Development	5%
Fully Developed Residential	60%
Industrial	90%

6.4. MODEL CALIBRATION

Calibration to an observed rainfall/runoff event was considered for the PBI Model, however there was very little concurrent rainfall/runoff data in the French Camp Slough watershed. The available runoff data included stage recordings and did not include a rating curve. Calibration to an observed event would have contained a large amount of uncertainty and therefore was not included in the French Camp Slough analysis.

Constant loss rates were adjusted for each subbasin by a factor of 0.85 (Attachment 6-F). The adjustment factor was determined through a HEC-HMS calibration for the neighboring Calaveras River watershed. This watershed has similar characteristics to the French Camp

Slough watershed and has more reliable stream flow data. Further details of the Calaveras River model calibration can be found in Section 5.4.

6.5. DEVELOPMENT CONDITIONS

6.5.1. Existing Conditions

An ‘Existing Conditions’ model run was performed to evaluate peak flows given current (2010) land use and hydrologic conditions within the French Camp Slough watershed. Subbasin S-graphs, ‘n’ values, and impervious percentages were set according to current land cover conditions using field knowledge supplemented by aerial photos.

In general, the upstream watershed consists of natural or agricultural land whereas the lower portions of French Camp Slough watershed (west of Highway 99) are developed areas in and around the city of Stockton. Figure 6- 8 displays the existing development conditions and S-graphs assigned to each subbasin. A summary table of the subbasin characteristics used for ‘Existing Conditions’ model runs is provided in Attachment 6-D.

6.5.2. Future-Without-Project Conditions

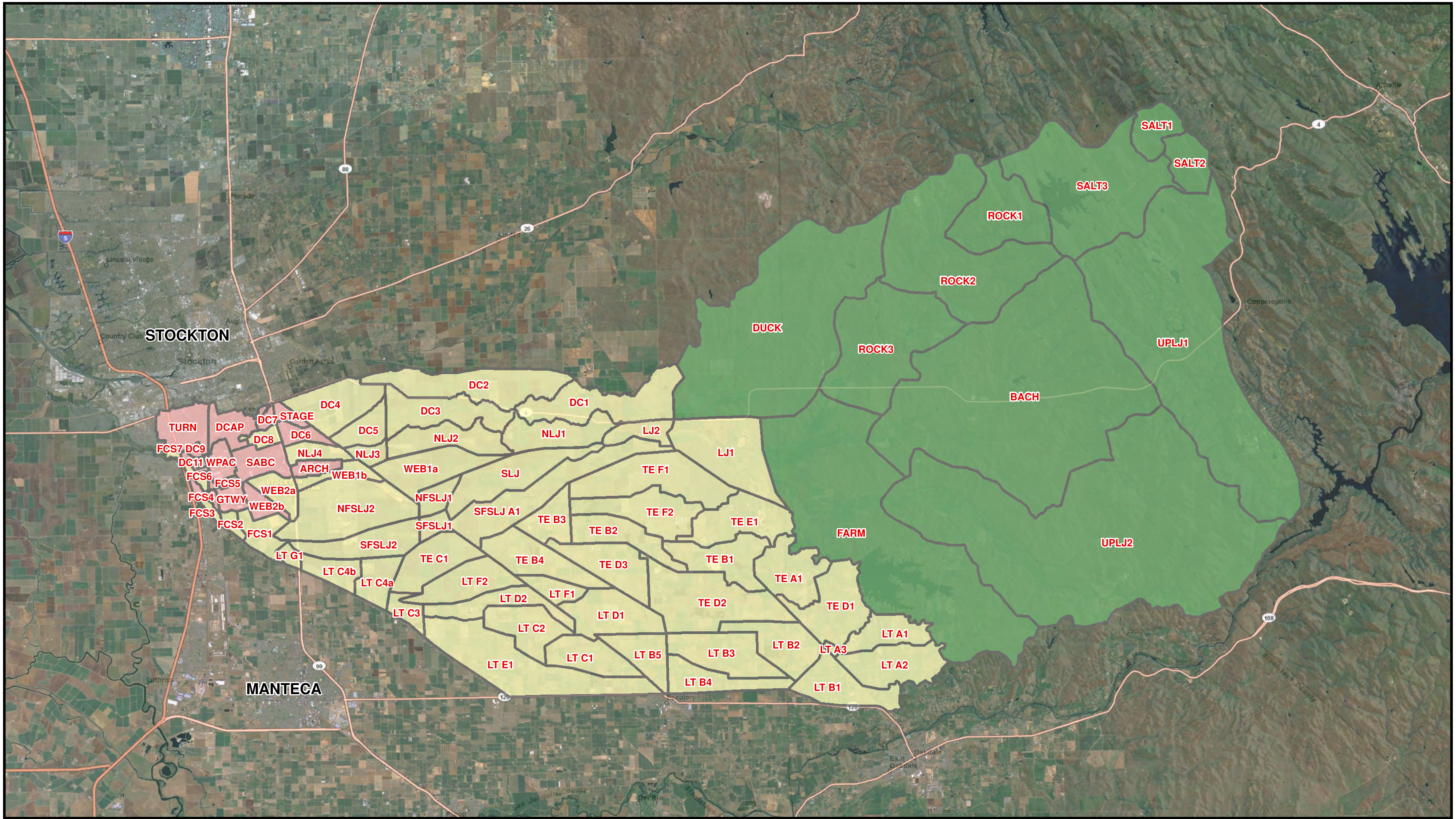
A ‘Future-Without-Project Conditions’ model run was performed to evaluate peak flows for future (2070) land use and hydrologic conditions within the French Camp Slough watershed.

Future land use conditions are based on the City of Stockton 2035 General Plan[12] and the San Joaquin County General Plan[17].

As shown in Figure 6- 9, the upstream watershed remains unchanged and consists of natural or agricultural land whereas the lower portions of French Camp Slough watershed experience an increase in development. There are sixteen (16) subbasins that were previously undeveloped in the ‘Existing Conditions’ model and are assumed to be fully developed for the ‘Future-Without-Project Conditions’ model.

In addition to updating subbasin S-graphs, ‘n’ values, and impervious percentages for the newly developed areas, storm water pump stations were also added to these subbasins. As previously mentioned, flows exceeding pump station capacities would cause temporary ponding, which was assumed to be mitigated within the subbasin through on-site detention.

A summary table of subbasin characteristics used for ‘Future-Without-Project Conditions’ model runs is provided in Attachment 6-E.



🟥 Valley Developed S-Graph

🟨 Valley Undeveloped S-Graph

🟩 Foothill S-Graph

N

0 1.5 3
Miles

1 inch = 3 miles

OCTOBER 19, 2010

Peterson . Brustad . Inc
Engineering . Consulting

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

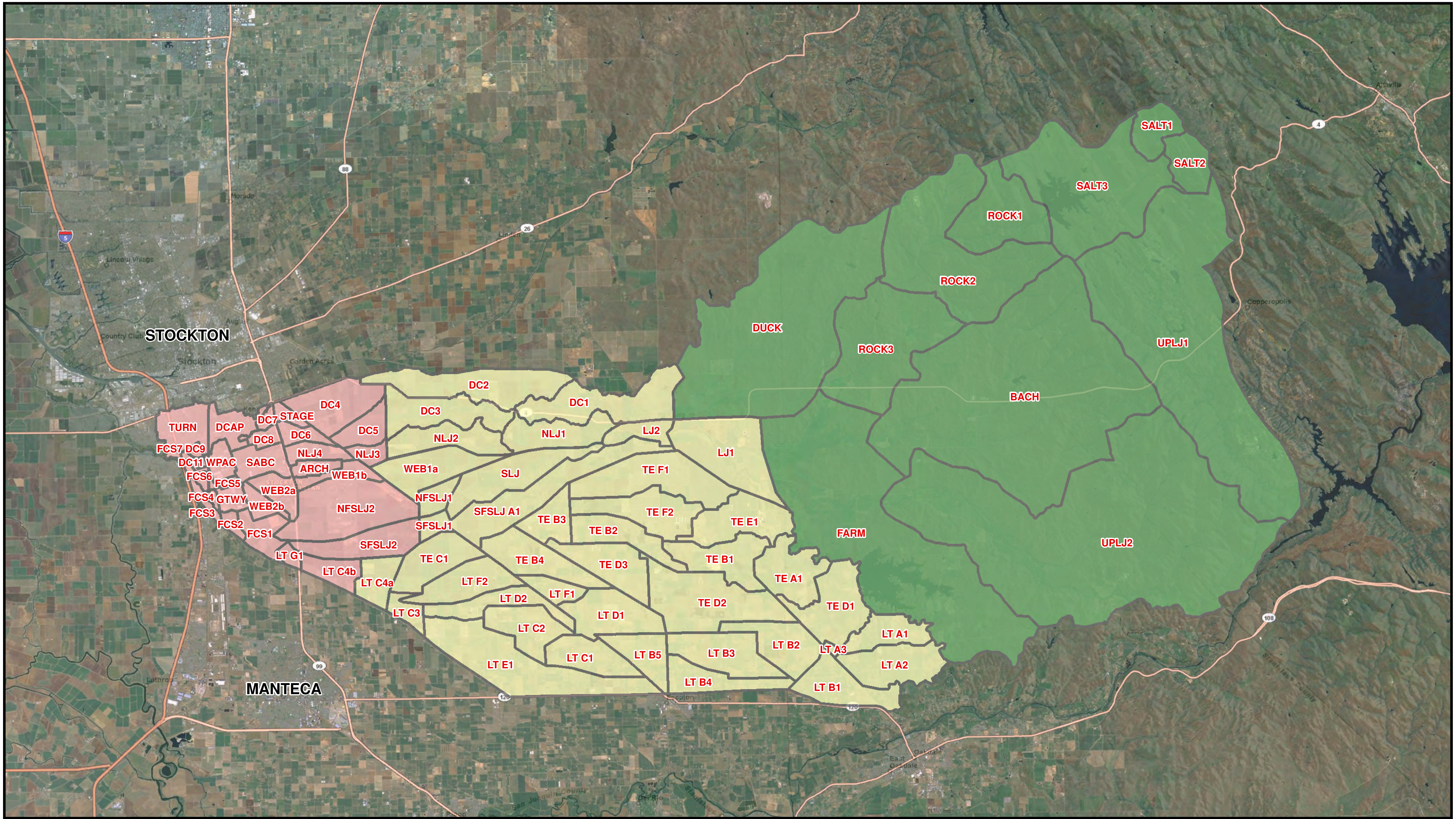
Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

EXISTING DEVELOPMENT CONDITIONS FOR FRENCH CAMP SLOUGH WATERSHED

FIGURE

6-8



🟥 Valley Developed S-Graph
🟨 Valley Undeveloped S-Graph
🟩 Foothill S-Graph

N

0 1.5 3
Miles
1 inch = 3 miles

OCTOBER 19, 2010

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING
1180 Iron Point Rd., Suite 260
Folsom, CA 95630
Phone: (916) 608-2212
Fax: (916) 608-2232

SAN JOAQUIN AREA FLOOD CONTROL AGENCY

FUTURE DEVELOPMENT CONDITIONS FOR FRENCH CAMP SLOUGH WATERSHED

FIGURE

6-9

6.6. STORM CENTERINGS

Four storm centerings were analyzed for the French Camp Slough watershed:

- The *Upper Watershed* centering was analyzed to stress the foothill region of the study area which could produce flash flooding.

- The *Farmington* centering was placed directly above Farmington Reservoir and was analyzed to stress Farmington Dam.

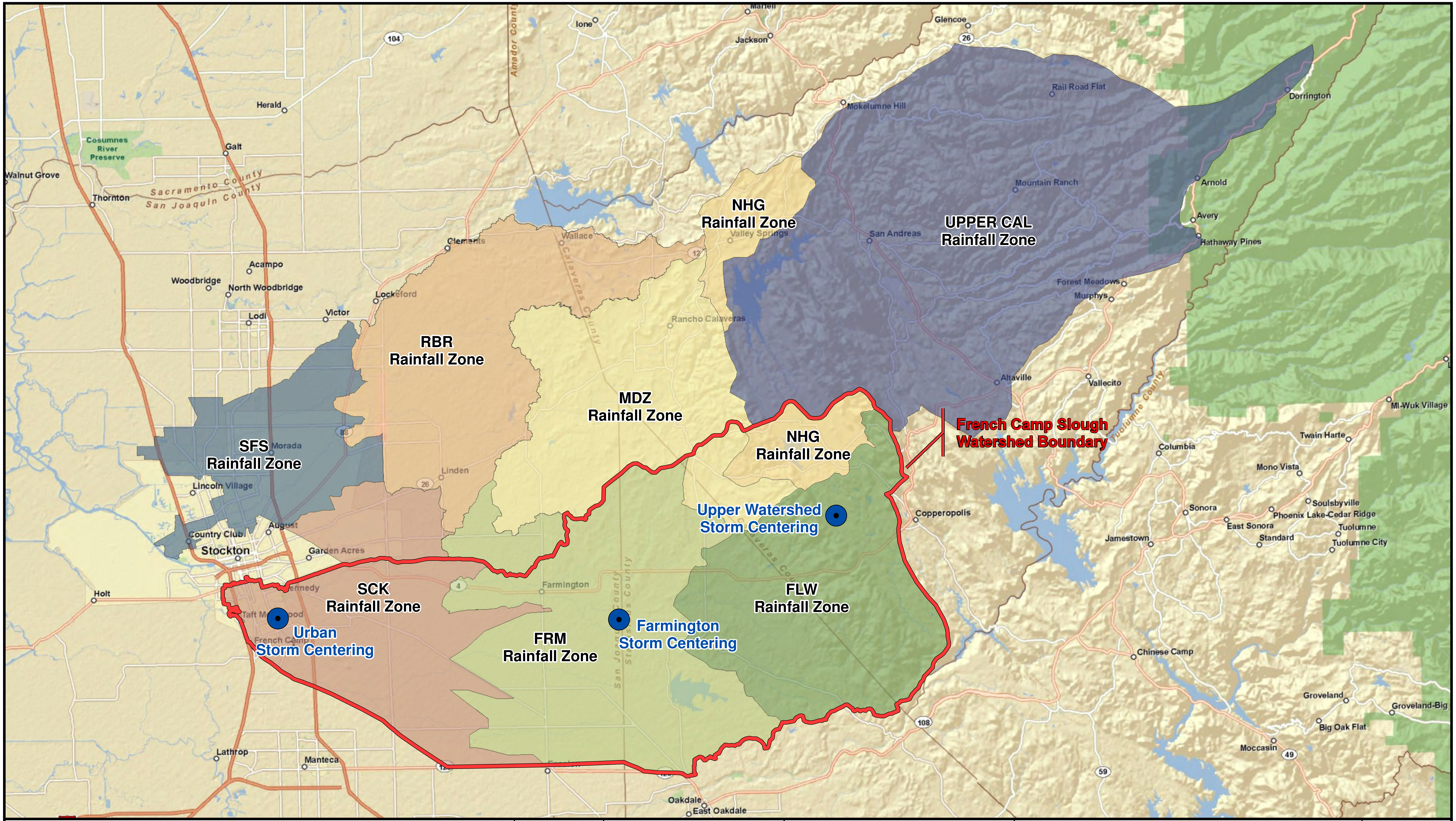
- The *"Average"* centering took the average of the *Upper Watershed* and *Farmington* area reduction factors to come up with rainfall depths. This centering is considered the official LSJRFS design storm for the French Camp Slough production runs.

  Reservoir flows from the Ford analysis were reported for an "Average" storm centering and are most applicable for this scenario. The "Average" storm centering scenario is considered the LSJRFS design storm scenario for French Camp Slough production runs and therefore are the only flows reported in Table 6- 6 and Table 6-7.

- The *Urban* centering was analyzed for interior drainage purposes and is directly centered over the urban areas in the lower watershed.

Eight design storms with the 1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, and 1/500 AEP events will be produced for each centering. This selection of design storms provides a wide range of scenarios that can be used for planning purposes.

Calculated area reduction factors and resulting area-reduced rainfall depths for each rainfall zone are provided in Attachment 6-G for all frequency-duration-storm centering combinations.



N

0 5 Miles
1 : 300,000

JUNE 24, 2011

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

FRENCH CAMP SLOUGH WATERSHED STORM CENTERINGS

FIGURE
6-10

6.7. MODEL SIMULATIONS

French Camp Slough production runs include 64 scenarios with unique combinations of development conditions, storm frequencies, and storm centerings.

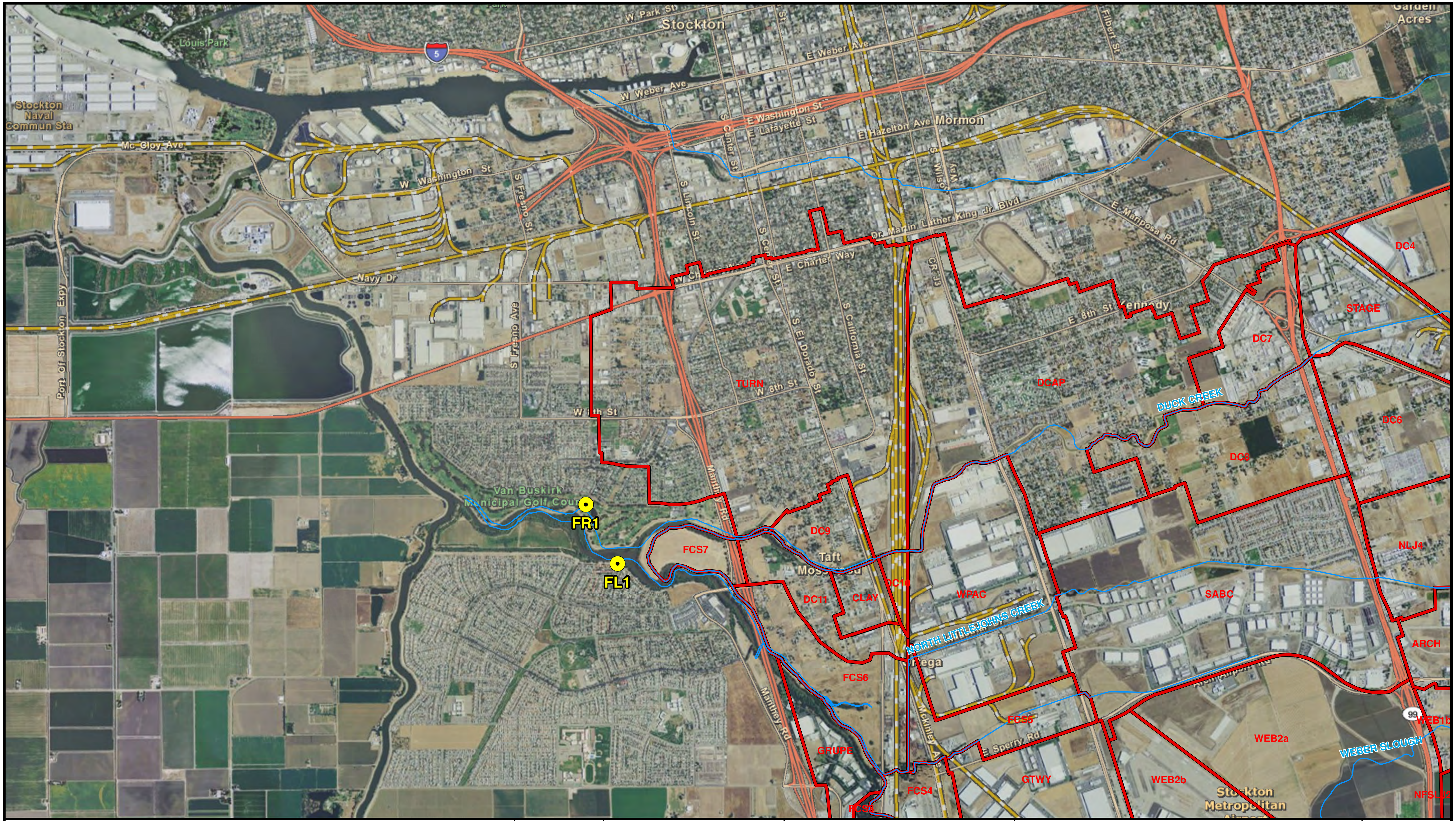
Table 6- 5. French Camp Slough production run scenarios.

Development Conditions	Storm Centerings	AEP Events
Existing Conditions	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Farmington	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	"Average"	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Upper Watershed	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
Future-Without-Project Conditions (2070)	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Farmington	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	"Average"	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Upper Watershed	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500

6.7.1. Summary of Results

Peak flow results were extracted from HEC-HMS at each LSJRFS index point. Locations of LSJRFS index points within the French Camp Slough watershed are shown in Figure 6- 11. Table 6- 6 and Table 6-8 summarize “Average” storm centering peak flows for ‘Existing Conditions’ runs and for ‘Future-Without-Project Conditions’ runs, respectively.

Reservoir flows from the Ford analysis were reported for an “Average” storm centering and are most applicable for this scenario. The “Average” storm centering scenario is considered the LSJRFS design storm scenario for the Calaveras River production runs and therefore are the only flows reported in Table 6- 6 and Table 6-8.



🟥 Subshed Boundary

⊙ LSJRFS Index Point

N

0 0.125 0.25 0.5 Miles

1 inch = 1/2 mile

JUNE 20, 2012

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260
Folsom, CA 95630

Phone: (916) 608-2212
Fax: (916) 608-2232

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

FRENCH CAMP SLOUGH WATERSHED INDEX POINTS

FIGURE

6-11

Table 6-6. Peak Flow Results for French Camp Slough - Existing Conditions [cfs]

LSJRFS Index Point ID	Description	Average Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
--	Littlejohns Creek at Town of Farmington[1]	1,400	2,170	2,370	2,620	3,740	9,900	12,900	16,600
--	Duck Creek at Hwy 99	410	500	740	1,050	1,310	1,570	1,800	2,140
--	North Littlejohns Creek at Hwy 99	20	50	130	250	360	460	560	750
--	North Fork- South LJ Creek at Hwy 99	650	1,010	1,110	1,200	1,340	1,700	1,910	1,970
--	South Fork- South LJ Creek at Hwy 99	760	1,190	1,350	1,500	1,890	3,860	5,900	7,730
--	FCS at UPRR	1,440	2,540	2,860	3,170	3,590	5,030	6,070	7,020
D7 & D8	FCS at Duck Creek Confluence	1,790	3,030	3,860	4,710	5,500	6,490	7,090	7,800

[1] Flows for Littlejohns Creek at Town of Farmington were Provided by USACE. All upstream model flows were superseded by USACE hydrographs at Farmington.

Table 6-7. Peak Flow Results for French Camp Slough - Future Conditions [cfs]

LSJRFS Index Point ID	Description	Average Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
--	Littlejohns Creek at Town of Farmington[1]	1,400	2,170	2,370	2,620	3,740	9,900	12,900	16,600
--	Duck Creek at Hwy 99	1,140	1,470	1,530	1,590	1,740	1,770	1,790	1,990
--	North Littlejohns Creek at Hwy 99	240	300	340	470	470	480	550	750
--	North Fork- South LJ Creek at Hwy 99	720	1,090	1,200	1,300	1,390	1,730	1,920	1,980
--	South Fork- South LJ Creek at Hwy 99	820	1,270	1,450	1,590	1,900	3,850	5,890	7,720
--	FCS at UPRR	1,590	2,700	3,030	3,340	3,730	5,060	6,090	7,030
D7 & D8	FCS at Duck Creek Confluence	2,050	3,290	4,060	4,870	5,660	6,610	7,200	7,840

[1] Flows for Littlejohns Creek at Town of Farmington were Provided by USACE. All upstream model flows were superseded by USACE hydrographs at Farmington.

6.7.2. Uncertainty Parameters

For the purposes of the LSJRFS, uncertainty parameters for each flow-frequency dataset can be estimated within HEC-FDA during the project's economic analysis. HEC-FDA defines uncertainty in terms of confidence intervals or standard deviations given inputs of flow-frequency data (provided in Table 6- 6 and Table 6-8) and an equivalent record length.

The equivalent record length is an estimate of the overall "worth" or "quality" of the flow-frequency function, expressed as the number of years-of-record[19]. For probability functions derived at ungaged locations using model or other data, the equivalent record length is based on a judgment of the quality of that model or data. EM 1110-2-1619[20] provides guidelines for assigning equivalent record lengths.

The French Camp Slough model wasn't calibrated to an observed event; however the parameters were adjusted based on the calibration of the neighboring Calaveras River model (see Section 5.4). Because of this, there is slightly less confidence in the French Camp flows compared to flows calculated for the other LSJRFS watersheds. EM 1619 estimates that the equivalent record length is 10-30 years for a rainfall-runoff model with *regional* adjustments made to its parameters.

7.0 REFERENCES

1. National Oceanic and Atmospheric Administration (NOAA), *Atlas 14 Precipitation Frequency Study for California*, April 2011.
2. U.S. Army Corps of Engineers, Sacramento District, *Downtown Guadalupe River Project, Appendix D: Documentation of Meteorology Calculations*, November 2009**.**
3. United States Geological Survey (USGS), Digital Elevation Model (DEM) Datasets. Published on September 17, 2001.
4. California Department of Water Resources, LiDAR Datasets for the San Joaquin River Basin, Central Valley Floodplain Evaluation and Delineation Program (CVFED), March 2010 Deliverable.
5. U.S. Army Corps of Engineers, HEC-HMS Version 3.4 User's Manual, August 2009.
6. U.S. Army Corps of Engineers, HEC-GeoHMS Version 4.2 User's Manual, May 2009.
7. City of Stockton, Pump Data Acquired via Email Correspondence from Chris Retzius, August 5, 2010.
8. HDR, Final Technical Memorandum #1, Hydrologic Report prepared for San Joaquin Area Flood Control Agency, January 1998.
9. U.S. Army Corps of Engineers, HEC-HMS for the Sacramento and San Joaquin River Basins Comprehensive Study, August 2001.
10. County of San Joaquin, Draft Hydrology Manual for San Joaquin County, September 1997.
11. City of Stockton, Conceptual Storm Drain Master Plan, July 2008.
12. City of Stockton, City of Stockton 2035 General Plan, December 2007.
13. National Resource Conservation Service (NRCS), Soils data acquired from the NRCS Soils Datamart, August 23, 2010.
14. University of California Cooperative Extension, Soils data acquired from Calaveras County Soil-Vegetation Maps, August 30, 2010.
15. Maidment, David R., Handbook of Hydrology, McGraw-Hill, 1993.

16. County of San Joaquin, ALERT System Data, Data Acquired September 22, 2010.

17. County of San Joaquin, County of San Joaquin General Plan, July 1992.

18. U.S. Army Corps of Engineers, *Engineer Manual 1110-2-1417: Flood-Runoff Analysis*, 31 August 1994.

19. U.S. Army Corps of Engineers, HEC-FDA Version 1.2.4 User's Manual, November 2008.

20. U.S. Army Corps of Engineers, *Engineer Manual 1110-2-1619: Risk Based Analysis for Flood Damage Reduction Studies*, 01 August 1996.

21. David Ford Consulting Engineers, *Lower San Joaquin River Feasibility Study: Calaveras River Frequency Analysis and Hydrographs*, 20 June 2011.

22. U.S. Army Corps of Engineers, *Draft Memorandum for Record: Lower San Joaquin River Feasibility Study, Bellota and Farmington Regulated Flow Hydrographs,* 07 February 2012.

23. David Ford Consulting Engineers, *Lower San Joaquin River Feasibility Study: Littlejohn Creek Frequency Analysis and Hydrographs*, 23 June 2011.

24. California Data Exchange Center (CDEC), Data Acquired September 10, 2010.

25. Domenichelli & Associates, Inc., Conditional Letter of Map Revision (CLOMR) for Tidewater Crossing Flood Control Project, April 2007.

26. Pacific Advanced Civil Engineering (PACE), Mariposa Lakes Off-Site Regional Hydrologic Investigation, 7 April 2006.

27. U.S. Army Corps of Engineers, CORPSCON v6.0.1, August 2004.

28. Federal Emergency Management Agency (FEMA), San Joaquin County Flood Insurance Study, 16 October 2009.

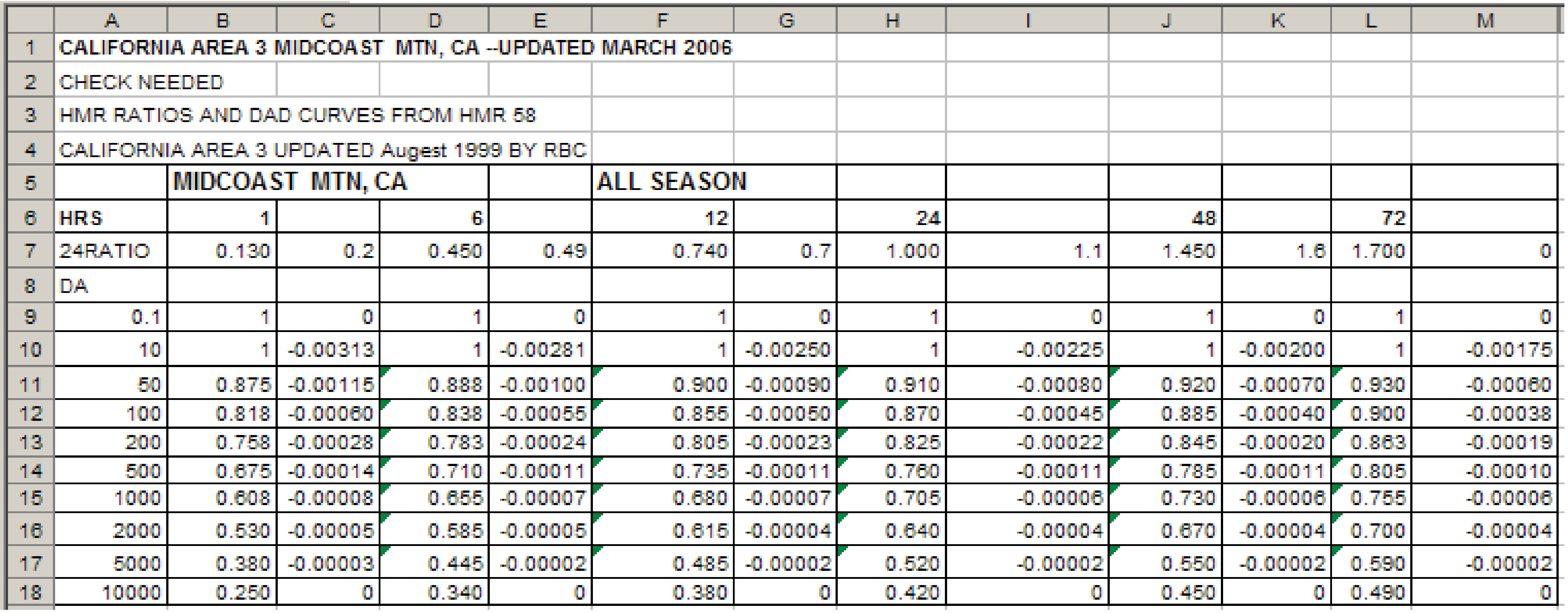
8.0 ATTACHMENTS

Attachment 2- A. Procedure for Calculating Area Reduction Factors: USACE Guadalupe River Hydrology Report[2]

Documentation of Meteorology Calculations

The purpose of this document is to outline how the 3-day statistical precipitation patterns for the sub-basins were calculated. The following will go through the calculations performed in the Excel spreadsheet created by the Sacramento District which linearly interpolates between the Depth Area Reduction Factors (DARFs) as presented in HMR 59 for various drainage areas, computes the statistical 1, 6, 12, 24, 48, and 72-hour cumulative precipitation for each sub-basin, and creates custom hyetographs.

Depth Area Reduction Factors (DARFs)



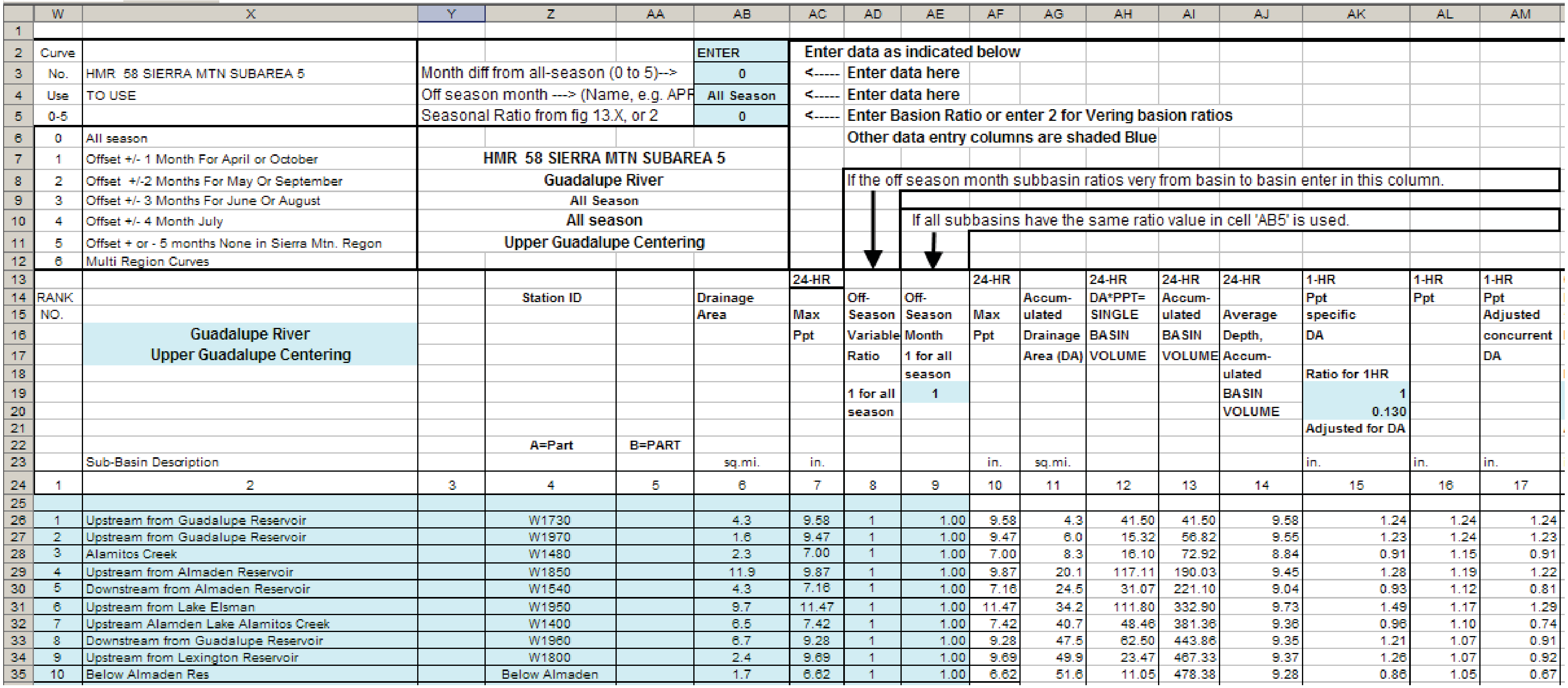
	A	B	C	D	E	F	G	H	I	J	K	L	M
1	CALIFORNIA AREA 3 MIDCOAST MTN, CA --UPDATED MARCH 2006												
2	CHECK NEEDED												
3	HMR RATIOS AND DAD CURVES FROM HMR 58												
4	CALIFORNIA AREA 3 UPDATED Augest 1999 BY RBC												
5		MIDCOAST MTN, CA				ALL SEASON							
6	HRS	1		6		12		24		48		72	
7	24RATIO	0.130	0.2	0.450	0.49	0.740	0.7	1.000	1.1	1.450	1.6	1.700	0
8	DA												
9	0.1	1	0	1	0	1	0	1	0	1	0	1	0
10	10	1	-0.00313	1	-0.00281	1	-0.00250	1	-0.00225	1	-0.00200	1	-0.00175
11	50	0.875	-0.00115	0.888	-0.00100	0.900	-0.00090	0.910	-0.00080	0.920	-0.00070	0.930	-0.00060
12	100	0.818	-0.00060	0.838	-0.00055	0.855	-0.00050	0.870	-0.00045	0.885	-0.00040	0.900	-0.00038
13	200	0.758	-0.00028	0.783	-0.00024	0.805	-0.00023	0.825	-0.00022	0.845	-0.00020	0.863	-0.00019
14	500	0.675	-0.00014	0.710	-0.00011	0.735	-0.00011	0.760	-0.00011	0.785	-0.00011	0.805	-0.00010
15	1000	0.608	-0.00008	0.655	-0.00007	0.680	-0.00007	0.705	-0.00006	0.730	-0.00006	0.755	-0.00006
16	2000	0.530	-0.00005	0.585	-0.00005	0.615	-0.00004	0.640	-0.00004	0.670	-0.00004	0.700	-0.00004
17	5000	0.380	-0.00003	0.445	-0.00002	0.485	-0.00002	0.520	-0.00002	0.550	-0.00002	0.590	-0.00002
18	10000	0.250	0	0.340	0	0.380	0	0.420	0	0.450	0	0.490	0

The Depth Area Reduction Factors (DARFs) from HMR 59 are represented in the Excel spreadsheet as shown above. Columns B, D, F, H, J, and L contain the DARFs for the 1, 6, 12, 24, 48, and 72-hour durations, respectively. The columns in-between (Columns C, E, G, I, K, and M) calculate the incremental change in DARF for a given unit of area (1 square mile). These numbers are used to linearly interpolate between DARF values for drainage areas in-between those specified in column A.

The row labeled "24RATIO" (Row 7) refers to the ratio to 24-hour cumulative precipitation as derived in HMR 59. These ratios are used to convert the 24-hour precipitation (main input parameter) to 1, 6, 12, 48, and 72-hour precipitation for each sub-basin.

For the purpose of this study, the California Area 3 Midcoast Mountain, California DARFs were selected, as noted in Row 1.

Cumulative Precipitation Calculations



	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI	AJ	AK	AL	AM
1																	
2	Curve					ENTER	Enter data as indicated below										
3	No.	HMR 58 SIERRA MTN SUBAREA 5	Month diff from all-season (0 to 5)-->			0	<-----	Enter data here									
4	Use	TO USE	Off season month ---> (Name, e.g. APR			All Season	<-----	Enter data here									
5	0-5		Seasonal Ratio from fig 13.X, or 2			0	<-----	Enter Basion Ratio or enter 2 for Vering basion ratios									
6	0	All season						Other data entry columns are shaded Blue									
7	1	Offset +/- 1 Month For April or October		HMR 58 SIERRA MTN SUBAREA 5													
8	2	Offset +/-2 Months For May Or September		Guadalupe River				If the off season month subbasin ratios very from basin to basin enter in this column.									
9	3	Offset +/- 3 Months For June Or August		All Season													
10	4	Offset +/- 4 Month July		All season					If all subbasins have the same ratio value in cell 'AB5' is used.								
11	5	Offset + or - 5 months None in Sierra Mtn. Regon		Upper Guadalupe Centering													
12	6	Multi Region Curves															
13							24-HR			24-HR		24-HR	24-HR	24-HR	1-HR	1-HR	1-HR
14	RANK			Station ID		Drainage		Off-	Off-		Accum-	DA*PPT=	Accum-		Ppt	Ppt	Ppt
15	NO.					Area	Max	Season	Season	Max	ulated	SINGLE	ulated	Average	specific		Adjusted
16		Guadalupe River					Ppt	Variable	Month	Ppt	Drainage	BASIN	BASIN	Depth,	DA		concurrent
17		Upper Guadalupe Centering						Ratio	1 for all		Area (DA)	VOLUME	VOLUME	Accum-			DA
18									season					ulated	Ratio for 1HR		
19								1 for all	1					BASIN	1		
20								season						VOLUME	0.130		
21															Adjusted for DA		
22				A=Part	B=PART												
23		Sub-Basin Description				sq.mi.	in.			in.	sq.mi.				in.	in.	in.
24	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
25																	
26	1	Upstream from Guadalupe Reservoir		W1730		4.3	9.58	1	1.00	9.58	4.3	41.50	41.50	9.58	1.24	1.24	1.24
27	2	Upstream from Guadalupe Reservoir		W1970		1.6	9.47	1	1.00	9.47	6.0	15.32	56.82	9.55	1.23	1.24	1.23
28	3	Alamitos Creek		W1480		2.3	7.00	1	1.00	7.00	8.3	16.10	72.92	8.84	0.91	1.15	0.91
29	4	Upstream from Almaden Reservoir		W1850		11.9	9.87	1	1.00	9.87	20.1	117.11	190.03	9.45	1.28	1.19	1.22
30	5	Downstream from Almaden Reservoir		W1540		4.3	7.16	1	1.00	7.16	24.5	31.07	221.10	9.04	0.93	1.12	0.81
31	6	Upstream from Lake Elsman		W1950		9.7	11.47	1	1.00	11.47	34.2	111.80	332.90	9.73	1.49	1.17	1.29
32	7	Upstream Alamden Lake Alamitos Creek		W1400		6.5	7.42	1	1.00	7.42	40.7	48.46	381.36	9.36	0.96	1.10	0.74
33	8	Downstream from Guadalupe Reservoir		W1960		6.7	9.28	1	1.00	9.28	47.5	62.50	443.86	9.35	1.21	1.07	0.91
34	9	Upstream from Lexington Reservoir		W1800		2.4	9.69	1	1.00	9.69	49.9	23.47	467.33	9.37	1.26	1.07	0.92
35	10	Below Almaden Res		Below Almaden		1.7	6.62	1	1.00	6.62	51.6	11.05	478.38	9.28	0.86	1.05	0.67

Fields shaded in light blue represent cells in which user input is possible or required.

Row 3, Column AB is populated with a “0” to represent “all-season” conditions. Seasonal variations in precipitation patterns are not accounted for in this study.

Row 26 and below:

- Column W is populated with the sub-basin “rank number”. Each sub-basin is assigned a rank, starting with the storm center and ending at the mouth of the basin.
- Column X is populated with a description of each sub-basin.

- Column Z includes the sub-basin ID, as used in the HMS model.
- Column AB includes the individual drainage area of each sub-basin.
- Column AC is populated with the calculated 24-hour cumulative precipitation for each sub-basin given a particular event frequency (i.e. 2,5,10,25, 50, 100, 200 or 500-year).
- Columns AD and AE are set at 1, for "all seasons."
- The remaining Columns (not shaded in light blue) conduct various calculations on the inputted data.
    - Column AF remains the same as Column AC for "all seasons."
    - Column AG calculates the cumulative drainage area. This is done by summing up all of the drainage areas at a given sub-basin (rank x) from rank 1 to rank x. This number is used to estimate the drainage area upstream of a particular sub-basin. This number is used in selecting an appropriate set of DARFs for a given sub-basin. For example, for sub-basin W1850 with a "rank number" of 4, the cumulative drainage area is 20.1. The DARFs are then calculated using the table previously presented using a linear interpolation between the DARFs for 10 square miles and the DARFs for 50 square miles.
    - Column AH calculates the volume of water allotted to a given sub-basin during the maximum 24-hour duration.
    - Column AI calculates the cumulative volume of water at a given sub-basin (rank x) from rank 1 to rank x. This is used to make sure that the volume of water over the entire basin is conserved.
    - Column AJ calculates the average depth of water for the cumulative drainage area above a given sub-basin. This is calculated by dividing the cumulative water volume (Column AI) by the cumulative drainage area (Column AG). This number is used in subsequent calculations to estimate the specific and concurrent precipitation at a given index location (at the outlet of a given sub-basin) with respect to the entire drainage area above the index location.
    - Columns AK through AM conduct calculations for the 1-hour duration that are subsequently carried out for the remaining durations (6, 12, 24, 48, and 72-hours) to estimate the "specific" and "concurrent" precipitation:
        - Column AK uses the ratio to 24-hour precipitation, 0.130 (see previous table), to estimate the maximum cumulative 1-hour specific precipitation.

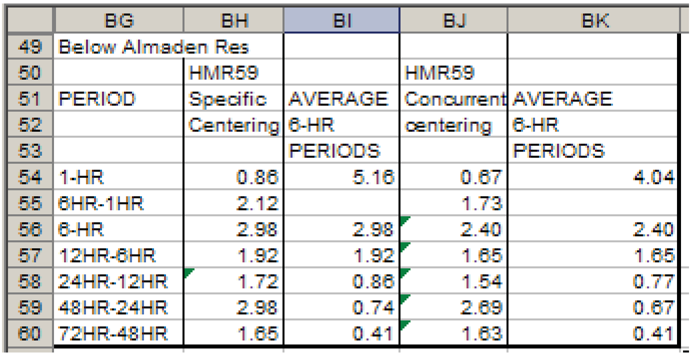
- Column AL represents an intermediary calculation necessary to calculate the concurrent precipitation. This field multiplies the average depth for the cumulative drainage area above a given sub-basin (Column AJ) by the ratio to 24-hour precipitation (0.130), and applies the appropriate DARF using the aforementioned table.

- Column AM calculates the concurrent cumulative precipitation by subtracting the product of the accumulated drainage area (Column AG) and Column AL from the previous sub-basin (by rank) from the product of Column AG and Column AL of the given sub-basin, and dividing this difference by the drainage area of the given sub-basin. This calculates the precipitation depth for a particular sub-basin while taking into account its placement in the larger basin area with respect to the storm center. In other words:

  $PptCon10 = (CumDA10*PptSp10 - CumDAR9*PptSp9)/DA10$

  PptCon10 = Concurrent cumulative precipitation for a sub-basin of a given rank (say, rank = 10).
  CumDA10 = Cumulative Drainage Area of the sub-basin (rank = 10)
  PptSp10 = Specific cumulative precipitation for the sub-basin, previously calculated using the average rainfall depth over all of the previous sub-basins (ranks 1 through 10).
  CumDAR9 = Cumulative drainage area for the previous sub-basin (rank = 9).
  PptSp9 = Specific cumulative precipitation for the previous sub-basin (rank = 9), previously calculated.
  DA10 = Individual drainage area for the sub-basin (rank = 10).

6-Hour Average Precipitation



	BG	BH	BI	BJ	BK
49	Below Almaden Res				
50		HMR59		HMR59	
51	PERIOD	Specific	AVERAGE	Concurrent	AVERAGE
52		Centering	6-HR	centering	6-HR
53			PERIODS		PERIODS
54	1-HR	0.86	5.16	0.67	4.04
55	6HR-1HR	2.12		1.73	
56	6-HR	2.98	2.98	2.40	2.40
57	12HR-6HR	1.92	1.92	1.65	1.65
58	24HR-12HR	1.72	0.86	1.54	0.77
59	48HR-24HR	2.98	0.74	2.69	0.67
60	72HR-48HR	1.65	0.41	1.63	0.41

These calculations will be used in distributing the calculated 1, 6, 12, 24, 48, and 72-hour specific and concurrent rainfall over the chosen precipitation pattern (“Pattern A”).

- Columns BH and BJ present the previously calculated specific and concurrent rainfall depths, and various differences between depths.

- Columns BI and BK calculate the average 6-hour rainfall of a given period (when applicable, no 6-hour average calculated for Row 55).

Creating Precipitation Patterns

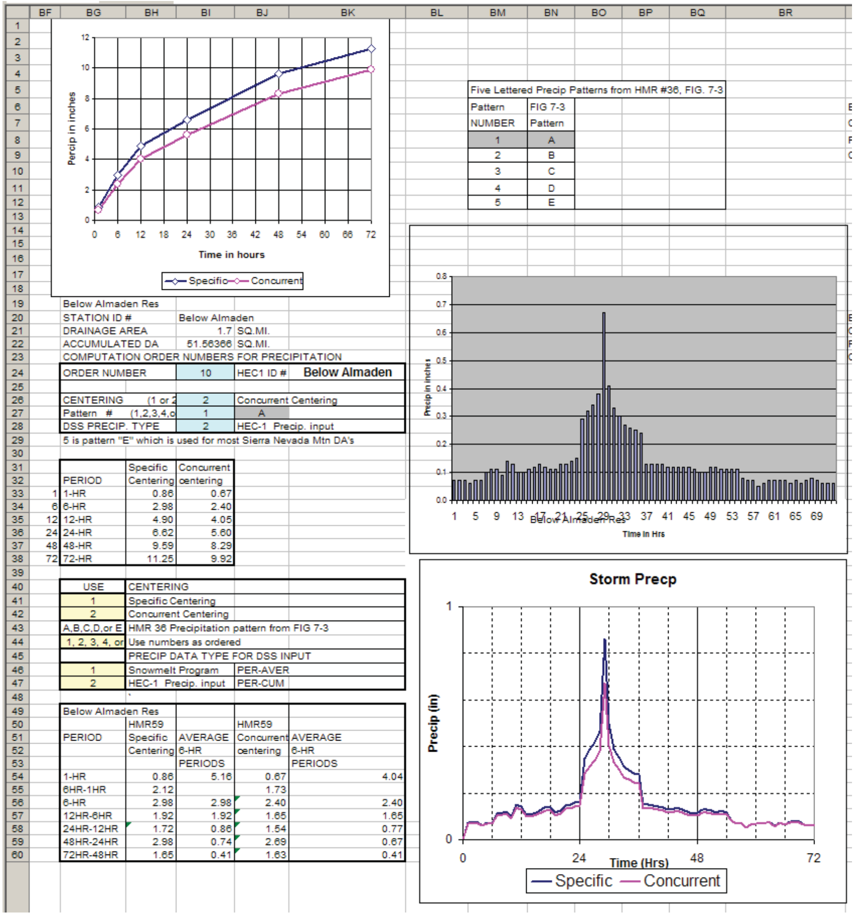
	W	X	Y	Z	AA	AB	AC	AD	AE
40						PERIOD	HMR56	BASIN	
41						COUNT	PMP	PATTERN	PATTERN
42							PATTERN	HMR56	HMR55
43		HMR# 56 PATTERN "A" = adjustments						Specific	Concurrent
44								Centering	Centering
45									
46		1	specific	0.41	72	1	0.30	0.071	0.070
47		2	conc	0.41	72	2	0.30	0.071	0.070
48	9	3	pattern total	1.74	72	3	0.30	0.071	0.070
49		4	Specific sum	0.41	72	4	0.25	0.059	0.059
50		5	concur.sum	0.41	72	5	0.30	0.071	0.070
51		6			72	6	0.29	0.069	0.068
52		1	specific	0.74	48	7	0.34	0.114	0.103
53		2	conc	0.67	48	8	0.35	0.117	0.106
54	7	3	pattern total	2.22	48	9	0.35	0.117	0.106
55		4	Specific sum	0.74	48	10	0.30	0.101	0.091
56		5	concur.sum	0.67	48	11	0.45	0.151	0.136
57		6			48	12	0.43	0.144	0.130
58		1	specific	0.74	48	13	0.42	0.109	0.099
59		2	conc	0.67	48	14	0.42	0.109	0.099
60	5	3	pattern total	2.86	48	15	0.45	0.117	0.106
61		4	Specific sum	0.74	48	16	0.50	0.130	0.118
62		5	concur.sum	0.67	48	17	0.54	0.141	0.127
63		6			48	18	0.53	0.139	0.125
64		1	specific	0.86	24	19	0.52	0.118	0.106
65		2	conc	0.77	24	20	0.55	0.124	0.112
66	3	3	pattern total	3.80	24	21	0.65	0.147	0.132
67		4	Specific sum	0.86	24	22	0.66	0.149	0.134
68		5	concur.sum	0.77	24	23	0.70	0.158	0.142
69		6			24	24	0.72	0.163	0.146
70		1	specific	2.96	6	25	0.75	0.349	0.285
71		2	conc	2.40	6	26	0.83	0.386	0.316
72	1	3	pattern total	5.70	6	27	0.90	0.419	0.342
73		4	Specific sum	2.96	6	28	1.00	0.465	0.380
74		5	concur.sum	2.40	6	29	1.15	0.860	0.674
75		6			6	30	1.07	0.498	0.407
76		1	specific	1.92	12	31	0.89	0.386	0.332
77		2	conc	1.65	12	32	0.81	0.352	0.302
78	2	3	pattern total	4.42	12	33	0.72	0.313	0.269
79		4	Specific sum	1.92	12	34	0.69	0.300	0.258
80		5	concur.sum	1.65	12	35	0.66	0.287	0.246
81		6			12	36	0.65	0.282	0.243
82		1	specific	0.86	24	37	0.57	0.149	0.134
83		2	conc	0.77	24	38	0.57	0.149	0.134
84	4	3	pattern total	3.23	24	39	0.56	0.147	0.132
85		4	Specific sum	0.86	24	40	0.55	0.144	0.129
86		5	concur.sum	0.77	24	41	0.53	0.139	0.125
87		6			24	42	0.50	0.131	0.118
88		1	specific	0.74	48	43	0.44	0.130	0.118
89		2	conc	0.67	48	44	0.46	0.136	0.123
90	6	3	pattern total	2.51	48	45	0.44	0.130	0.118
91		4	Specific sum	0.74	48	46	0.40	0.119	0.107
92		5	concur.sum	0.67	48	47	0.38	0.113	0.102
93		6			48	48	0.39	0.116	0.104
94		1	specific	0.74	48	49	0.34	0.129	0.117
95		2	conc	0.67	48	50	0.34	0.129	0.117
96	8	3	pattern total	1.96	48	51	0.33	0.125	0.113
97		4	Specific sum	0.74	48	52	0.31	0.118	0.106
98		5	concur.sum	0.67	48	53	0.33	0.125	0.113
99		6			48	54	0.31	0.116	0.106
100		1	specific	0.41	72	55	0.31	0.082	0.080
101		2	conc	0.41	72	56	0.27	0.071	0.070
102	9	3	pattern total	1.57	72	57	0.28	0.074	0.073
103		4	Specific sum	0.41	72	58	0.20	0.053	0.052
104		5	concur.sum	0.41	72	59	0.25	0.066	0.065
105		6			72	60	0.26	0.068	0.067
106		1	specific	0.41	72	61	0.24	0.070	0.069
107		2	conc	0.41	72	62	0.25	0.073	0.072
108	9	3	pattern total	1.41	72	63	0.25	0.073	0.072
109		4	Specific sum	0.41	72	64	0.20	0.059	0.058
110		5	concur.sum	0.41	72	65	0.25	0.073	0.072
111		6			72	66	0.22	0.065	0.064
112		1	specific	0.41	72	67	0.23	0.075	0.074
113		2	conc	0.41	72	68	0.24	0.079	0.078
114	12	3	pattern total	1.26	72	69	0.23	0.075	0.074
115		4	Specific sum	0.41	72	70	0.20	0.066	0.065
116		5	concur.sum	0.41	72	71	0.18	0.059	0.058
117		6			72	72	0.18	0.059	0.058
118									
119							32.73	11.25	9.92

The specific and concurrent precipitation depths for the 1, 6, 12, 24, 48, and 72-hour durations are used to create a unique rainfall pattern for each sub-basin based on a selected pattern. For the purpose of this study, "Pattern A" from HMR 59 was selected, as the critical 1, 12, and 24-hour cumulative precipitation is distributed in a fashion similar to large storms in and around the study area. All of the patterns presented in HMR 59 were taken from recorded storm events throughout California.

The calculations presented in this table distribute the specific and concurrent precipitation for the 1, 6, 12, 24, 48, and 72-hour durations while maintaining the shape of the selected pattern. The rainfall pattern (Pattern A) is broken up into 6-hour increments; the 6-hour average precipitation values previously calculated for various durations are used to estimate the precipitation for each time-step.

- Specific and concurrent rainfall is distributed based average precipitation estimated for a given 6-hour increment. Within a given 6-hour increment, Column Z contains several calculations:
    - ▪ Specific and Concurrent: pulls numbers from the 6-Hour Average Precipitation Table (presented previously) depending on which duration is represented in a given 6-hour time increment.
    - ▪ Pattern Total, Specific Sum, and Concurrent Sum: Cumulative rainfall for a given 6-hour increment from Columns AC, AD, and AE, respectively.
- Column AA signifies which data from the "6-hour precipitation" table will be used in the calculations for a given 6-hour increment. For example:
    - o The beginning of the rainfall pattern (Rows 46 and 51) is characterized by the rainfall between the maximum 48-hour and 72-hour precipitation pattern. As a result, the values in Row 60 of the "6-hour precipitation" table (72HR-48HR) are used to estimate the average "specific" and "concurrent" precipitation.
    - o The portion of the hyetograph between Rows 70-75 represents the maximum 6-hour pattern. As a result, the values in Row 56 of the "6-hour precipitation" table (6-HR) are used to estimate the average specific and concurrent precipitation.
- Column AC presents the selected hyetograph (Pattern A).
- Column AD displays the calculated specific hyetograph for a given sub-basin.
- Column AE displays the calculated concurrent hydrograph for a given sub-basin.
- The hyetograph displayed in Column AE is the primary output from this spreadsheet. This hyetograph can be copied and pasted directly into the meteorologic model in HMS for a particular sub-basin. In this example the output for sub-basin "Below Almaden", rank = 10, is displayed.

Spreadsheet Output



Below Almaden Res

STATION ID # Below Almaden

DRAINAGE AREA 1.7 SQ.MI.

ACCUMULATED DA 51.56366 SQ.MI.

COMPUTATION ORDER NUMBERS FOR PRECIPITATION

ORDER NUMBER	10	HEC1 ID #	**Below Almaden**
CENTERING (1 or 2	2	Concurrent Centering	
Pattern # (1,2,3,4,o	1	A	
DSS PRECIP. TYPE	2	HEC-1 Precip. input	

5 is pattern "E" which is used for most Sierra Nevada Mtn DA's

	PERIOD	Specific Centering	Concurrent centering
1	1-HR	0.86	0.67
6	6-HR	2.98	2.40
12	12-HR	4.90	4.05
24	24-HR	6.62	5.60
48	48-HR	9.59	8.29
72	72-HR	11.25	9.92

USE	CENTERING	
1	Specific Centering	
2	Concurrent Centering	
A,B,C,D,or E	HMR 36 Precipitation pattern from FIG 7-3	
1, 2, 3, 4, or	Use numbers as ordered	
	PRECIP DATA TYPE FOR DSS INPUT	
1	Snowmelt Program	PER-AVER
2	HEC-1 Precip. input	PER-CUM

Below Almaden Res PERIOD	HMR59 Specific Centering	AVERAGE 6-HR PERIODS	HMR59 Concurrent centering	AVERAGE 6-HR PERIODS
1-HR	0.86	5.16	0.67	4.04
6HR-1HR	2.12		1.73	
6-HR	2.98	2.98	2.40	2.40
12HR-6HR	1.92	1.92	1.65	1.65
24HR-12HR	1.72	0.86	1.54	0.77
48HR-24HR	2.98	0.74	2.69	0.67
72HR-48HR	1.65	0.41	1.63	0.41

Five Lettered Precip Patterns from HMR #36, FIG. 7-3

Pattern NUMBER	FIG 7-3 Pattern
1	A
2	B
3	C
4	D
5	E

The spreadsheet calculates the specific and concurrent hyetographs for a given sub-basin by specifying a particular “order number” or “rank” in Column BI, Row 24.

Concurrent centering is selected by placing the number “2” in Column BI, Row 26 (“1” is for specific centering only).

The rainfall pattern (Pattern A, B, C, D, or E) is selected by specifying 1, 2, 3, 4, or 5 (respectively) in Column BI, Row 27.

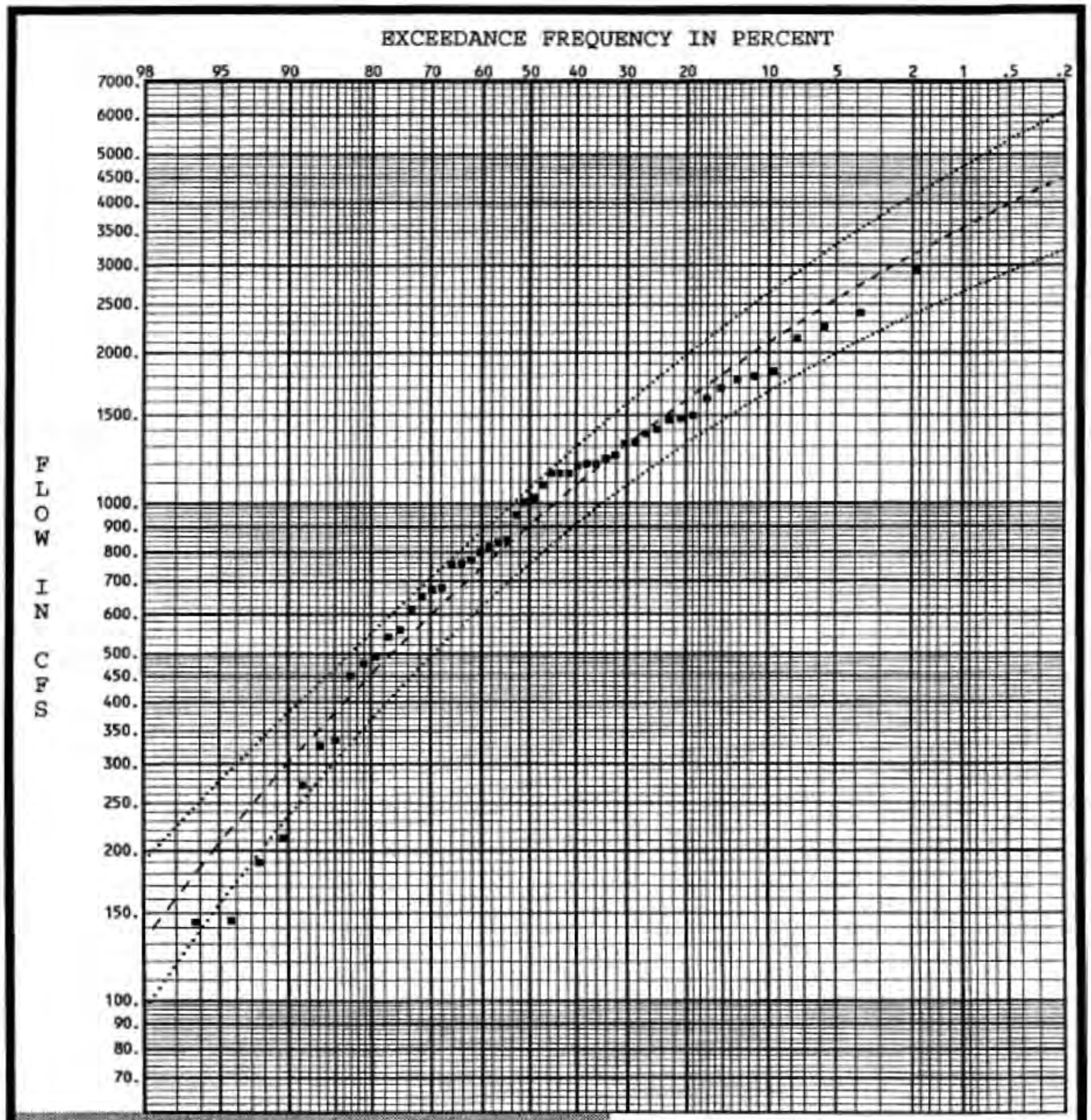
Once a desired hyetograph has been created, the spreadsheet is able to conduct various graphical representations and comparisons of the specific and concurrent precipitation patterns.

Attachment 3- A. Bear Creek Watershed Comparison of Subbasin Parameters: 1998 SJAFCA HEC-1 Model vs. 2010 PBI HEC-HMS Model

	AREA [sq. mi]			LAG TIME [hrs]		
Subbasin	1998 SJAFCA HEC-1 Model	2010 PBI HEC-HMS Model	% Difference	1998 SJAFCA HEC-1 Model	2010 PBI HEC-HMS Model	% Difference
B7	26.7	30.24	13%	7.80	12.59	61%
B6	13.5	11.73	-13%	5.06	9.29	84%
B5	5.6	4.04	-28%	2.97	5.83	96%
B4	2.6	1.53	-41%	2.60	4.47	72%
B10	11.7	12.01	3%	5.08	5.97	18%
B12	4.6	4.41	-4%	5.42	7.17	32%
B11	3.2	2.60	-19%	4.36	4.43	2%
B9	3.1	3.88	25%	4.84	8.73	80%
B8	1.4	0.95	-32%	3.31	3.70	12%
B13	4.71	5.28	12%	5.54	6.34	14%
B3	2.3	2.84	24%	4.76	6.29	32%
B2	1.71	2.06	21%	3.17	5.11	61%
B1	1.4	2.30	64%	2.32	3.78	63%
M3	5.67	5.76	2%	6.10	6.55	7%
M2	1.45	1.02	-29%	3.37	4.26	27%
M1	2.85	3.54	24%	4.58	4.42	-4%
LB15	0.5	0.35	-30%	0.50	0.20	-61%
LB10	1.21	0.54	-55%	0.54	0.20	-63%
LB20	1.14	0.83	-27%	0.55	0.33	-39%
LB30	0.86	0.50	-41%	0.65	0.24	-63%
LB35	1.25	0.85	-32%	0.90	0.23	-74%
LB40	1.15	1.69	47%	0.62	0.25	-60%
PX1	5	7.46	49%	6.10	5.87	-4%
LP10	1.92	1.25	-35%	2.72	2.65	-3%
LP20	1.3	0.82	-37%	2.61	2.92	12%
LP30	2.06	2.09	1%	3.57	0.41	-88%
LP31	0.79	1.10	39%	4.70	0.49	-90%
LP34[a]	--	1.25	--	--	0.32	--
LP32	1.02	0.40	-61%	1.80	0.22	-88%
LP33[a]	--	0.32	--	--	0.21	--
LB50	0.78	1.54	97%	0.48	0.34	-30%
LB55	0.06	0.28	361%	0.39	0.21	-47%
LB60	0.77	0.57	-26%	0.29	0.22	-23%
LB70	0.2	0.26	32%	0.22	0.16	-29%

[a] Subbasin parameters for LP33 and LP34 are not listed in 1998 SJAFCA HEC-1 documentation.

Attachment 3- B. Flow-Frequency for Bear Creek at Lockeford Stream Gage Used in 1998 HEC-1 Calibration



- - - Flow Frequency (with Exp. Prob.)
■ Weibull Plotting Positions
······· 5% and 95% Confidence Limits

FREQUENCY STATISTICS

LOG TRANSFORM OF FLOW, CFS		NUMBER OF EVENTS	
MEAN	2.9237	HISTORIC EVENTS	0
STANDARD DEV	.3255	HIGH OUTLIERS	0
SKEW	-.9101	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	-.6000	SYSTEMATIC EVENTS	52

BEAR CREEK, NEAR LOCKEFORD

BASIN AREA = 47.6 sq. mi.
WATER YEARS IN RECORD
1932-1975,1978-1985

Source: HDR, Final Technical Memorandum #1, Hydrologic Report prepared for San Joaquin Area Flood Control Agency, January 1998.

Attachment 3- C. Bear Creek Subbasin Soil Groups and Loss Rates

Soil Group:	A (0.35 in/hr)	B (0.2 in/hr)	C (0.1 in/hr)	D (0.025 in/hr)	Composite Loss Rate	Adjusted Loss Rate (Adjustment Factor = 0.80)
Subbasin	[sq. mi.]	[sq. mi.]	[sq. mi.]	[sq. mi.]	[in/hr]	[in/hr]
B7	0.07	6.18	3.13	20.44	0.070	0.056
B6	0.17	1.58	2.06	7.83	0.067	0.053
B5	0.00	0.39	2.31	1.34	0.085	0.068
B4	0.00	0.15	0.70	0.61	0.078	0.063
B10	0.00	0.08	5.72	6.15	0.062	0.050
B12	0.00	0.00	0.99	3.42	0.042	0.033
B11	0.00	0.00	0.45	2.16	0.038	0.030
B9	0.00	0.37	0.92	2.46	0.060	0.048
B8	0.00	0.00	0.00	0.90	0.025	0.020
B13	0.00	0.00	0.85	4.22	0.038	0.030
B3	0.00	1.01	0.47	1.26	0.103	0.082
B2	0.00	0.01	0.28	1.65	0.037	0.030
B1	0.00	0.00	0.34	1.89	0.037	0.029
M3	0.00	0.38	0.49	4.89	0.043	0.034
M2	0.00	0.23	0.30	0.49	0.086	0.069
M1	0.00	0.14	1.35	2.00	0.061	0.049
LB15	0.00	0.00	0.04	0.31	0.033	0.026
LB10	0.00	0.00	0.00	0.52	0.025	0.020
LB20	0.00	0.00	0.00	0.82	0.025	0.020
LB30	0.00	0.00	0.00	0.50	0.025	0.020
LB35	0.00	0.00	0.00	0.81	0.025	0.020
LB40	0.00	0.00	0.03	1.61	0.027	0.021
PX1	0.00	4.92	0.72	1.83	0.148	0.118
LP10	0.00	0.47	0.41	0.37	0.115	0.092
LP20	0.03	0.24	0.43	0.12	0.126	0.101
LP30	0.00	0.04	2.02	0.00	0.102	0.081
LP31	0.00	0.00	0.30	0.80	0.045	0.036
LP34	0.00	0.00	1.24	0.00	0.100	0.080
LP32	0.00	0.00	0.34	0.05	0.090	0.072
LP33	0.00	0.00	0.25	0.06	0.085	0.068
LB50	0.00	0.00	0.51	0.99	0.051	0.040
LB55	0.00	0.00	0.19	0.07	0.079	0.064
LB60	0.00	0.00	0.55	0.00	0.100	0.080
LB70	0.00	0.00	0.23	0.00	0.100	0.080

Attachment 3- D. Bear Creek Subbasin Characteristics – Existing Conditions

Basin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss Rate	Constant Loss Rate	Impervious %	Associated Pump Station	Pump Station Capacity
		n	L			Lc	S	Lg						
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]		[cfs]
B7	30.24	0.2	14.49	950	131	6.57	56.52	12.59	FH	0.5	0.056	0	--	--
B6	11.73	0.2	8.50	466	114	4.30	41.39	9.29	FH	0.5	0.053	0	--	--
B5	4.04	0.18	5.05	227	97	2.21	25.72	5.83	VU	1.5	0.068	2	--	--
B4	1.53	0.15	1.96	96	93	1.11	1.53	4.47	VU	1.5	0.063	5	--	--
B10	12.01	0.18	5.37	302	111	2.61	35.59	5.97	FH	0.5	0.050	2	--	--
B12	4.41	0.18	5.32	165	80	2.85	15.99	7.17	FH	0.5	0.033	2	--	--
B11	2.60	0.18	2.98	150	88	1.64	20.84	4.43	FH	0.5	0.030	2	--	--
B9	3.88	0.18	5.44	111	71	3.17	7.35	8.73	VU	1.5	0.048	2	--	--
B8	0.95	0.18	1.68	80	75	0.68	2.98	3.70	VU	1.5	0.020	2	--	--
B13	5.28	0.15	5.87	140	71	2.59	11.76	6.34	VU	1.5	0.030	5	--	--
B3	2.84	0.18	3.79	93	69	1.78	6.33	6.29	VU	1.5	0.082	2	--	--
B2	2.06	0.18	2.81	70	55	1.28	5.35	5.11	VU	1.5	0.030	2	--	--
B1	2.30	0.18	1.86	55	45	0.87	5.37	3.78	VU	1.5	0.029	2	--	--
M3	5.76	0.18	4.49	166	95	2.64	15.81	6.55	FH	0.5	0.034	2	--	--
M2	1.02	0.18	2.18	92	75	1.24	7.80	4.26	VU	1.5	0.069	2	--	--
M1	3.54	0.15	2.76	75	55	1.68	7.25	4.42	VU	1.5	0.049	5	--	--
LB15	0.35	0.18	1.02	43	37	0.48	5.90	2.35	VU	1.5	0.026	2	--	--
LB10	0.54	0.18	1.04	45	37	0.57	7.68	2.40	VU	1.5	0.020	2	--	--
LB20	0.83	0.15	1.69	40	34	0.89	3.54	3.31	VU	1.5	0.020	5	--	--
LB30	0.50	0.18	1.38	35	25	0.68	7.25	2.89	VU	1.5	0.020	2	--	--
LB35	0.85	0.15	1.53	34	16	0.68	11.73	2.30	VU	1.5	0.020	2	--	--
LB40	1.69	0.18	1.87	29	13	0.57	8.57	2.94	VU	1.5	0.021	2	--	--
PX1	7.46	0.18	5.75	79	45	3.07	5.91	9.18	VU	1.5	0.118	2	--	--
LP10	1.25	0.15	1.27	45	40	0.69	3.92	2.65	VU	1.5	0.092	5	--	--
LP20	0.82	0.18	0.98	40	38	0.52	2.04	2.92	VU	1.5	0.101	2	--	--
LP30	2.09	0.18	2.58	38	24	1.29	5.43	4.95	VU	1.5	0.081	2	--	--
LP31	1.10	0.18	3.17	40	25	1.56	4.74	5.90	VU	1.5	0.036	2	--	--
LP32	0.40	0.18	0.80	21	20	0.37	1.24	2.61	VU	1.5	0.072	2	--	--
LP33	0.32	0.015	0.80	22	20	0.47	2.49	0.21	VD	0.2	0.068	60	Pixley PS	90.8
LB50	1.54	0.015	1.87	20	10	1.02	5.34	0.34	VD	0.2	0.040	60	Thornton PS	431
LB55	0.28	0.015	1.10	12	10	0.30	1.83	0.21	VD	0.2	0.064	60		
LB60	0.57	0.015	1.15	12	5	0.61	6.11	0.22	VD	0.2	0.080	60	I-5 PS	46.8
LB70	0.26	0.015	0.77	7	1	0.42	7.75	0.16	VD	0.2	0.080	60		

Notes: VU = Valley Undeveloped; VD = Valley Developed; FH = Foothill

Attachment 3- E. Bear Creek Subbasin Characteristics – Future Conditions

Basin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss Rate	Constant Loss Rate	Impervious %	Associated Pump Station	Pump Station Capacity
		n	L			Lc	S	Lg						
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]		[cfs]
B7	30.24	0.2	14.49	950.00	131.00	6.57	56.52	12.59	FH	0.5	0.056	0	--	--
B6	11.73	0.2	8.50	466.00	114.00	4.30	41.39	9.29	FH	0.5	0.053	0	--	--
B5	4.04	0.18	5.05	227.00	97.00	2.21	25.72	5.83	VU	1.5	0.068	2	--	--
B4	1.53	0.15	1.96	96.00	93.00	1.11	1.53	4.47	VU	1.5	0.063	5	--	--
B10	12.01	0.18	5.37	302.00	111.00	2.61	35.59	5.97	FH	0.5	0.050	2	--	--
B12	4.41	0.18	5.32	165.00	80.00	2.85	15.99	7.17	FH	0.5	0.033	2	--	--
B11	2.60	0.18	2.98	150.00	88.00	1.64	20.84	4.43	FH	0.5	0.030	2	--	--
B9	3.88	0.18	5.44	111.00	71.00	3.17	7.35	8.73	VU	1.5	0.048	2	--	--
B8	0.95	0.18	1.68	80.00	75.00	0.68	2.98	3.70	VU	1.5	0.020	2	--	--
B13	5.28	0.15	5.87	140.00	71.00	2.59	11.76	6.34	VU	1.5	0.030	5	--	--
B3	2.84	0.18	3.79	93.00	69.00	1.78	6.33	6.29	VU	1.5	0.082	2	--	--
B2	2.06	0.18	2.81	70.00	55.00	1.28	5.35	5.11	VU	1.5	0.030	2	--	--
B1	2.30	0.18	1.86	55.00	45.00	0.87	5.37	3.78	VU	1.5	0.029	2	--	--
M3	5.76	0.18	4.49	166.00	95.00	2.64	15.81	6.55	FH	0.5	0.034	2	--	--
M2	1.02	0.18	2.18	92.00	75.00	1.24	7.80	4.26	VU	1.5	0.069	2	--	--
M1	3.54	0.15	2.76	75.00	55.00	1.68	7.25	4.42	VU	1.5	0.049	5	--	--
LB15	0.35	0.015	1.02	43.00	37.00	0.48	5.90	0.20	VD	0.2	0.026	60	PLB15	83
LB10	0.54	0.015	1.04	45.00	37.00	0.57	7.68	0.20	VD	0.2	0.020	60	PLB10	128
LB20	0.83	0.015	1.69	40.00	34.00	0.89	3.54	0.33	VD	0.2	0.020	60	PLB20	197
LB30	0.50	0.015	1.38	35.00	25.00	0.68	7.25	0.24	VD	0.2	0.020	60	PLB30	118
LB35	0.85	0.015	1.53	34.00	16.00	0.68	11.73	0.23	VD	0.2	0.020	60	PLB35	201
LB40	1.69	0.015	1.87	29.00	13.00	0.57	8.57	0.25	VD	0.2	0.021	60	PLB40	400
PX1	7.46	0.115	5.75	79.00	45.00	3.07	5.91	5.87	VU	1.5	0.118	2	--	--
LP10	1.25	0.15	1.27	45.00	40.00	0.69	3.92	2.65	VU	1.5	0.092	5	--	--
LP20	0.82	0.18	0.98	40.00	38.00	0.52	2.04	2.92	VU	1.5	0.101	2	--	--
LP30	2.09	0.015	2.58	38.00	24.00	1.29	5.43	0.41	VD	0.2	0.081	60	PLP30	495
LP31	1.10	0.015	3.17	40.00	25.00	1.56	4.74	0.49	VD	0.2	0.036	60	PLP31	260
LP34	1.25	0.015	1.30	22.00	21.00	0.50	0.77	0.32	VD	0.2	0.080	60	PLP34	296
LP32	0.40	0.015	0.80	21.00	20.00	0.37	1.24	0.22	VD	0.2	0.072	60	PLP32	95
LP33	0.32	0.015	0.80	22.00	20.00	0.47	2.49	0.21	VD	0.2	0.068	60	Pixley PS	90.8
LB50	1.54	0.015	1.87	20.00	10.00	1.02	5.34	0.34	VD	0.2	0.040	60	Thornton PS	431
LB55	0.28	0.015	1.10	12.00	10.00	0.30	1.83	0.21	VD	0.2	0.064	60		
LB60	0.57	0.015	1.15	12.00	5.00	0.61	6.11	0.22	VD	0.2	0.080	60	I-5 PS	46.8
LB70	0.26	0.015	0.77	7.00	1.00	0.42	7.75	0.16	VD	0.2	0.080	60		

Notes: VU = Valley Undeveloped; VD = Valley Developed; FH = Foothill

Attachment 3- F. Bear Creek Depth-Duration-Frequency Tables

BEAR CREEK WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: SFS

Calculated Average Point Rainfall Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
10 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
15 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
30 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
60 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
3 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
6 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
12 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
24 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
48 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
72 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
96 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.120	0.157	0.187	0.227	0.257	0.288	0.320	0.362
10 min	0.173	0.225	0.268	0.325	0.369	0.413	0.459	0.519
15 min	0.208	0.273	0.325	0.393	0.446	0.500	0.554	0.628
30 min	0.286	0.375	0.445	0.540	0.612	0.686	0.761	0.862
60 min	0.396	0.518	0.616	0.747	0.848	0.949	1.053	1.193
3 hour	0.645	0.797	0.923	1.097	1.235	1.379	1.531	1.746
6 hour	0.879	1.067	1.224	1.443	1.617	1.799	1.993	2.267
12 hour	1.168	1.426	1.639	1.932	2.161	2.396	2.643	2.982
24 hour	1.603	1.983	2.292	2.711	3.032	3.358	3.694	4.149
48 hour	2.038	2.519	2.905	3.418	3.806	4.195	4.590	5.117
72 hour	2.345	2.893	3.330	3.909	4.342	4.775	5.211	5.791
96 hour	2.581	3.184	3.660	4.289	4.758	5.224	5.691	6.310

BEAR CREEK WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: SFS

Calculated Average Point Rainfall Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Upper Watershed Storm Centering Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.733	0.738	0.738	0.736	0.736	0.736	0.736	0.735
10 min	0.735	0.736	0.738	0.736	0.737	0.735	0.734	0.734
15 min	0.735	0.736	0.736	0.737	0.737	0.736	0.735	0.733
30 min	0.734	0.737	0.737	0.736	0.736	0.736	0.734	0.732
60 min	0.735	0.738	0.738	0.738	0.738	0.736	0.736	0.735
3 hour	0.735	0.735	0.735	0.735	0.734	0.733	0.733	0.732
6 hour	0.735	0.734	0.734	0.733	0.733	0.732	0.732	0.732
12 hour	0.735	0.734	0.734	0.733	0.733	0.733	0.733	0.733
24 hour	0.735	0.735	0.735	0.735	0.735	0.735	0.735	0.735
48 hour	0.736	0.735	0.735	0.735	0.735	0.735	0.736	0.736
72 hour	0.736	0.736	0.735	0.735	0.735	0.735	0.736	0.736
96 hour	0.736	0.735	0.735	0.735	0.735	0.735	0.735	0.736

Upper Watershed Storm Centering Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.098	0.130	0.154	0.187	0.212	0.237	0.263	0.298
10 min	0.142	0.185	0.221	0.268	0.304	0.340	0.377	0.426
15 min	0.171	0.224	0.267	0.324	0.368	0.411	0.456	0.515
30 min	0.235	0.309	0.367	0.445	0.504	0.565	0.625	0.706
60 min	0.326	0.427	0.508	0.617	0.700	0.782	0.867	0.981
3 hour	0.531	0.655	0.759	0.902	1.014	1.130	1.256	1.430
6 hour	0.723	0.876	1.005	1.183	1.326	1.473	1.632	1.856
12 hour	0.961	1.171	1.345	1.584	1.772	1.964	2.167	2.445
24 hour	1.318	1.630	1.885	2.229	2.493	2.761	3.037	3.411
48 hour	1.678	2.071	2.388	2.810	3.129	3.449	3.779	4.213
72 hour	1.931	2.382	2.738	3.213	3.570	3.926	4.290	4.768
96 hour	2.125	2.617	3.009	3.526	3.912	4.295	4.679	5.195

BEAR CREEK WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: RBR

Calculated Average Point Rainfall Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.140	0.179	0.212	0.259	0.295	0.333	0.374	0.431
10 min	0.201	0.257	0.305	0.371	0.423	0.478	0.536	0.618
15 min	0.243	0.311	0.368	0.448	0.512	0.578	0.648	0.747
30 min	0.337	0.433	0.512	0.623	0.711	0.803	0.901	1.039
60 min	0.456	0.585	0.692	0.842	0.962	1.086	1.218	1.404
3 hour	0.751	0.929	1.080	1.294	1.467	1.649	1.846	2.125
6 hour	1.029	1.260	1.454	1.728	1.948	2.178	2.424	2.773
12 hour	1.377	1.692	1.952	2.312	2.593	2.884	3.187	3.606
24 hour	1.882	2.331	2.695	3.188	3.565	3.947	4.338	4.867
48 hour	2.368	2.936	3.388	3.988	4.438	4.886	5.338	5.935
72 hour	2.718	3.372	3.888	4.565	5.068	5.564	6.061	6.712
96 hour	2.997	3.722	4.290	5.030	5.576	6.112	6.645	7.339

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.767	0.764	0.764	0.765	0.765	0.765	0.765	0.766
10 min	0.766	0.765	0.764	0.765	0.765	0.766	0.766	0.767
15 min	0.766	0.765	0.765	0.765	0.765	0.765	0.766	0.767
30 min	0.766	0.765	0.765	0.765	0.765	0.765	0.766	0.767
60 min	0.766	0.764	0.764	0.764	0.764	0.765	0.765	0.766
3 hour	0.766	0.766	0.766	0.766	0.767	0.767	0.767	0.768
6 hour	0.766	0.766	0.766	0.767	0.767	0.767	0.767	0.768
12 hour	0.766	0.766	0.767	0.767	0.767	0.767	0.767	0.767
24 hour	0.766	0.766	0.766	0.766	0.766	0.766	0.766	0.766
48 hour	0.765	0.766	0.766	0.766	0.766	0.766	0.766	0.765
72 hour	0.765	0.766	0.766	0.766	0.766	0.766	0.765	0.765
96 hour	0.765	0.766	0.766	0.766	0.766	0.766	0.766	0.765

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.107	0.137	0.162	0.198	0.226	0.255	0.286	0.330
10 min	0.154	0.197	0.233	0.284	0.324	0.366	0.411	0.474
15 min	0.186	0.238	0.282	0.343	0.392	0.442	0.496	0.573
30 min	0.258	0.331	0.392	0.477	0.544	0.614	0.690	0.797
60 min	0.349	0.447	0.529	0.643	0.735	0.831	0.932	1.075
3 hour	0.575	0.712	0.827	0.991	1.125	1.265	1.416	1.632
6 hour	0.788	0.965	1.114	1.325	1.494	1.671	1.859	2.130
12 hour	1.055	1.296	1.497	1.773	1.989	2.212	2.444	2.766
24 hour	1.442	1.786	2.064	2.442	2.731	3.023	3.323	3.728
48 hour	1.812	2.249	2.595	3.055	3.400	3.743	4.089	4.540
72 hour	2.079	2.583	2.978	3.497	3.882	4.262	4.637	5.135
96 hour	2.293	2.851	3.286	3.853	4.271	4.682	5.090	5.614

BEAR CREEK WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: RBR

Calculated Average Point Rainfall Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.140	0.179	0.212	0.259	0.295	0.333	0.374	0.431
10 min	0.201	0.257	0.305	0.371	0.423	0.478	0.536	0.618
15 min	0.243	0.311	0.368	0.448	0.512	0.578	0.648	0.747
30 min	0.337	0.433	0.512	0.623	0.711	0.803	0.901	1.039
60 min	0.456	0.585	0.692	0.842	0.962	1.086	1.218	1.404
3 hour	0.751	0.929	1.080	1.294	1.467	1.649	1.846	2.125
6 hour	1.029	1.260	1.454	1.728	1.948	2.178	2.424	2.773
12 hour	1.377	1.692	1.952	2.312	2.593	2.884	3.187	3.606
24 hour	1.882	2.331	2.695	3.188	3.565	3.947	4.338	4.867
48 hour	2.368	2.936	3.388	3.988	4.438	4.886	5.338	5.935
72 hour	2.718	3.372	3.888	4.565	5.068	5.564	6.061	6.712
96 hour	2.997	3.722	4.290	5.030	5.576	6.112	6.645	7.339

Upper Watershed Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
10 min	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
15 min	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
30 min	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
60 min	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
3 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
6 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
12 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
24 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
48 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
72 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
96 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848

Upper Watershed Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.119	0.152	0.180	0.220	0.250	0.282	0.317	0.365
10 min	0.170	0.218	0.259	0.315	0.359	0.405	0.455	0.524
15 min	0.206	0.264	0.312	0.380	0.434	0.490	0.550	0.633
30 min	0.286	0.367	0.434	0.528	0.603	0.681	0.764	0.881
60 min	0.387	0.496	0.587	0.714	0.816	0.921	1.033	1.191
3 hour	0.637	0.788	0.916	1.097	1.244	1.398	1.565	1.802
6 hour	0.873	1.068	1.233	1.465	1.652	1.847	2.056	2.352
12 hour	1.168	1.435	1.655	1.961	2.199	2.446	2.703	3.058
24 hour	1.596	1.977	2.285	2.703	3.023	3.347	3.679	4.127
48 hour	2.008	2.490	2.873	3.382	3.763	4.143	4.527	5.033
72 hour	2.305	2.859	3.297	3.871	4.298	4.718	5.140	5.692
96 hour	2.541	3.156	3.638	4.265	4.728	5.183	5.635	6.223

Attachment 3- G. ITR Comment Forms for Bear Creek HEC-HMS Modeling

Lower San Joaquin River Feasibility Study

HYDROLOGIC MODELING REVIEW – BEAR CREEK WATERSHED

Task Order 6 – LSJRFS Work-In-Kind Hydrology

Reviewer:	Domenichelli & Associates
Review Date:	8-23-10
PBI Response Date:	9-24-10
Domenichelli Backcheck:	10-8-10

Backcheck Comments:

1. All previous comments were addressed adequately. No back check comments on previous comments.

2. New calibration information (Section 3.5): In the text it is stated that the new calibration gage data in Figure 8 corresponds to HMS model element MSRTN. The Figure shows a peak 100-yr event flow of approx 4,100cfs, however the model provided shows a peak flow of 6,300cfs at element MSRTN. Why is there a different result in the model?

   PBI Response: Figure 8 displays the model calibration results at MSRTN. The calibration utilized an observed rainfall event taken from historical gage records. The model provided to D&A includes the 100-year rainfall event taken from the 1998 SJAFCA HEC-1 model. Results shown in Figure 8 are expected to differ from the reviewed model due to differing rainfall inputs.

PETERSON . BRUSTAD . INC .
ENGINEERING . CONSULTING

Lower San Joaquin River Feasibility Study

HYDROLOGIC MODELING REVIEW – BEAR CREEK WATERSHED

Task Order 6 – LSJRFS Work-In-Kind Hydrology

Reviewer:	Domenichelli & Associates
Review Date:	8-23-10
PBI Response Date:	9-24-10

Note to Reviewer: After the original Draft TM was sent to D&A for ITR, PBI was able to obtain more detailed calibration data. Section 3.5 is now updated to describe the latest calibration methodology. Although new comments are not usually part of the backcheck process, comments on Section 3.5 are welcome if needed.

Memorandum Comments:

1. *Section 3.2 Model Development - This would be a good place early in the TM to describe the "Existing Conditions" and "Future Without Project" model assumptions and parameters.*

PBI Response: Section 3.2 now includes a mention of the 'Existing' and 'Future Without Project' model runs. However, this section was not intended to include significant details on the assumptions and parameters of the model runs and was only meant to provide an outline of the TM. Instead, Sections 3.6.1 and 3.6.2 include the relevant details and were referenced.

2. *Section 3.2 , Page 2, Item 3.- Provide Reference…*

PBI Response: Agreed. See Section 3.2 , Page 2, Item 3.

3. *Section 3.2.1, Paragraph 3 - Remove "was used" after "method".*

PBI Response: Agreed. See Section 3.2.1 , Paragraph 3.

4. *Section 3.3, Design Storms - Provide reasoning for 3-day storm and reason for (use of) so many (8) design frequency storms.*

PBI Response: Additional reasoning was added to Section 3.3 which now reads: "…A 72-hour storm was selected to stress the basin from both a peak flow and volume standpoint.…"
-and-

"The selection of eight design storms provides a wide range of scenarios that can be used for planning purposes."

5. *Table 2, Consider providing a column for routing reach description (ie..Basin B7 to B6).*

PBI Response: Agreed. See Section 3.4.5, Page 12, Table 2.

6. *Section 3.4.6, Loss Rates- The loss rates will usually vary with different soils type, natural cover, etc. Only one initial and constant rate combination is used. Is the entire Bear Creek watershed Type D soil? May want to include a soils map somewhere for confirmation.*

PBI Response: The method for assigning constant loss rates has now been modified. GIS soils layers were obtained from the NRCS. The percentage of hydrologic soil groups (A, B, C, or D) contained within each subbasin was determined through GIS calculations. Soil groups were each assigned a loss rate based on published studies and a weighted loss rate was calculated for each subbasin.

7. *Section 3.4.7 Impervious Percentages- May consider more intermediate values of impervious percentage to meet varied land uses (especially under existing, non built out conditions). Some adjacent upper sheds change between 2% to 10% with only small changes in current development.*

PBI Response: After further discussions with Domenichelli & Associates, impervious percentages for the 'Agricultural with Rural Development' land use classification were changed from 10% to 5% to ensure that no subbasins east of the CCTR are assigned impervious percentages greater than 5%.

8. *Section 3.6.2 Future Without Project Conditions- Is 2070 a typo error or does some document estimate the level of development for that time (seems like an odd number.)*

PBI Response: 2070 is the agreed upon Future-Without-Project date. The 2035 general plan gives the best possible estimates available for land use conditions given that a 2070 general plan does not exist.

9. *Attachment A – There is no reference to this table in the text. Will it be referenced under 3.7 Model Results after the models are updated with new rainfall data? Will a table for existing conditions be provided at that time also?*

PBI Response: Attachment A is referenced in Section 3.2.2, Page 4, Paragraph 4. It is meant to compare the SJAFCA HEC-1 model's results to the results produced by an "interim" HEC-HMS model which was produced after directly converting from the HEC-1 model. Note that this interim HMS model is not the most updated model which includes the initial/constant loss rate method, etc. Instead, these interim HMS results reflect a model that uses the same methods (Curve Numbers, etc) as the SJAFCA HEC-1 model and are meant to show that the direct conversion went smoothly.

Once the models are updated with the new rainfall data produced from the NOAA study, both ‘Existing’ and ‘Future Without Project’ production runs will be performed and results tables with be provided in a subsequent TM.

10. *Attachment A –label Table as “Future Without Project” for clarity. May also consider adding a column to describe the model elements.*

PBI Response: See PBI response to Item 9.

Once ‘Existing’ and ‘Future-Without-Project’ production runs are performed, complete results tables will be provided as Attachments. In addition, summary tables of model results will be provided in Section 3.7 of the final report. These tables will include peak flows produced at key locations in the watershed along with a description of model element locations.

11. *Attachment A – How do the PBI, HMS sub-basin flows shown in the table match precisely with the SJAFCA sub-basin results, even though a different loss rate method was used and more precise topo is used for lag time calculations? Would have expected some deviation in these comparisons. (See following comment 12)*

PBI Response: See PBI response to Item 9.

12. *Latest Model Results - Attachment A results in the table do not match the results provided in the latest HMS model.*

PBI Response: See PBI response to Item 9.

13. *Latest Model Results- Latest HMS results are significantly lower than the SJAFCA results at the downstream end of the system (approximately 18% lower), even though the flow at the Lockford gage was only 2% lower and one more sub-basin (LP34) is added to the new model at the downstream end. What is/are the reason(s) for this difference. The difference should be explained in the text.*

PBI Response: The HMS model was calibrated to give the same results at the Lockeford gage. However this location only includes 3 subbasins in its drainage area. The remaining 29 subbasins are expected to give differing results from the HEC-1 model for the following reasons:

1. The loss method was changed which fundamentally changes the calculations for infiltration.
2. The lag times were re-calculated for the PBI Model and, in many cases, are significantly different from those entered in the SJAFCA HEC-1 model. Both the magnitude and timing of peak flows are affected by the change in lag times. Therefore the peak flow contributions from the subbasins are expected to arrive at the model outlet with different timing and magnitudes than what occurred in the SJAFCA HEC-1 model.
3. PBI's 'Future-Without-Project' model has 5 additional pumps compared to the SJAFCA HEC-1 model. These pumps are set to discharge at 0.37 cfs/acre of tributary area (roughly a 1/10-AEP flow) and regulate flows for 5 additional subbasins which would otherwise contribute much higher peak flows.

14. *Figure 2 – Text box reads "LP24" should read "LP34"*

PBI Response: Agreed. See Figure 2.

Attachment 3- H. SPK Comment Forms for Bear Creek HEC-HMS Modeling

Corps of Engineers, Hydrology Section

Review of Bear Creek HEC-1 to HEC-HMS model conversion and preliminary report.
12 November 2010 with Responses 03 December 2010, SFH

Steven F. Holmstrom, P.E.

The Technical Memorandum for the Lower San Joaquin River Feasibility Study Bear Creek HEC-HMS modeling DRAFT hydrology report has been reviewed and the following comments are provided.

1. Section 3.2.1, SJAFCA HEC-1 model, states that the 1/100 AEP rainfall event matched the 1/100 AEP peak flow from the Bear Creek at Lockeford stream gage. Information on the period of record and statistics (mean, SD, skew), should be shown in the report to help quantify the uncertainty in the period of record. If additional data is available, the frequency curve should be updated to reduce the uncertainty in the estimate. The frequency curve used should be included in the report.

   PBI Response: The Lockeford stream gage was used in the calibration of the 1998 SJAFCA HEC-1 model. The frequency curve and statistics for this gage are now provided in Attachment B.

   SPK backcheck: OK

2. In section 3.3 Design Storms, the fourth paragraph states that two storm centerings will be analyzed. A third storm centering will be required to compute the flow above the New Hogan Dam for the 8 design storm frequencies.

   PBI Response: It is PBI's understanding that the third storm centering above New Hogan Dam will only be analyzed for the Calaveras River watershed. No changes were made to this section.

   SPK backcheck: OK

3. Section 3.4.1, subbasins, states that DWR LiDAR^2 data were used to confirm and revise subbasin boundaries and drainage areas. In addition, the study states that other HMS parameters were adjusted to calibrate the runoff at the gage. The study must provide a table showing the adjusted drainage area, and the differences in input parameters between the HEC-1 and re-calibrated HMS models.

   PBI Response: Parameters from the 1998 SJAFCA HEC-1 model are now included as Attachment A.

SPK backcheck: Attachment A appears to have changed from a comparison of peak flows from HEC-1 to HMS to a tabulation of Watershed parameters from the 1998 report. What I am looking for are 3 additional columns in the table comparing peak flows, that show the drainage area used in the 1998 study and the REVISED drainage area (which accounts for a difference in peak flow) to be used in the current study. Please restore the table in attachment B from the first draft and add columns representing DA from 1998, DA from GeoHMS and percent difference.

PBI Response to Backcheck: Attachment A now includes a subbasin parameter comparison between the 1998 HEC-1 model and the 2010 PBI HEC-HMS Model. When I included the HEC-1 vs. HEC-HMS peak flow results in the original Attachment A that you referenced, it seemed to cause quite a bit of confusion both in the ITR Review and SPK Review.

To clarify, the HEC-HMS model that produced the results listed in that Attachment used the same subbasin areas, same lag times, same curve number method, etc. as the 1998 HEC-1 model. There were virtually no differences between the peak flows listed in the two columns. This table was intended to show that the HEC-1 data cards were uploaded correctly into HEC-HMS.

In an attempt to eliminate any further confusion, I took the peak flow comparison table out of the report because most reviewers mistakenly thought that the peak flow results listed in the HEC-HMS column were from the 2010 PBI HEC-HMS Model.

Further Comparisons of HEC-1 and HEC-HMS results can be included once the final NOAA14 rainfall data are incorporated into the 2010 PBI HEC-HMS Model.

4. On figure 2, the Bear Creek at Lockeford stream gage location should be identified, or noted to be co-located with the ALERT gage. In addition, the location of flows diverted from Mosher Creek should be identified as input to Bear Creek.

   PBI Response: The Bear Creek at Lockeford stream gage is now labeled in Figure 2. Note that the Lockeford gage and the ALERT gage are not the same gage. The Bear Creek ALERT gage on is located approximately 7 miles downstream from the Lockeford gage.

   SPK backcheck: OK

5. In section 3.4.3, Diversions, the study should discuss the possibility of the diversions being overwhelmed during very high flows that is greater than the 1% flood.

   PBI Response: Section 3.4.3 has been updated.

   SPK backcheck: OK

6. On figure 6, Bear Creek subbasin flow paths, the location where Mosher Slough subbasin 1105 (M1, M2, M3) is diverted into Bear Creek must be shown. The subbasin boundary should also be shown in the Bear Creek figures.

   PBI Response: The Mosher Slough diversion point is now labeled in Figures 2 and 6. Note that Bear Creek's subbasins M1, M2, and M3 and Mosher Slough's subbasin 1105 do not cover the same drainage area; they are adjacent to one another. Mosher Slough subbasin 1105 was therefore not included in Figure 6.

   SPK backcheck: OK

7. The subbasin lag times defined in section 3.4.4, unit hydrograph S-graph and lag times, and listed in attachment C, subbasin characteristics, use a basin 'n in the calculations that appears to be high. Figure E-2 in the San Joaquin Hydrology Manual, notes that a basin 'n' of 0.20 is appropriate where "the groundcover consists of cultivated crops", and where the "surface characteristics are such that channelization does not occur". It appears that the choice of high basin 'n' values may result in very low (0.02 "/hour) loss rates. The study must review the relationship between basin 'n' values and loss rates through sensitivity analysis to derive more rational values for each parameter.

   PBI Response: As noted in Figure E-2 in San Joaquin's Hydrology Manual, a basin 'n' of 0.2 is appropriate for areas with cultivated crops where channelization does not occur. PBI assigned a basin 'n' value of 0.2 for all agricultural lands that have relatively flat slopes. When looking at the descriptions in the Hydrology Manual, 0.2 would appear to be the most appropriate basin 'n' value for flat, agricultural land. This land has "cultivated crops" and "channelization does not occur" due to its flat surface. If, on the other hand, the land had steeper slopes or was not cultivated, a basin 'n' value of 0.2 would be too high.

   Note that the assignment of basin 'n' values and loss rates are independent of each other. Basin 'n' values are assigned based on the land use type, etc. and are used in calculations for basin lag times (see Section 3.4.4). Loss rates are assigned based on the soil makeup within each subbasin (see Section 3.4.6). The low loss rates are a result of the abundance of Type D soils seen throughout the watershed.

   SPK backcheck: OK

8. The frequency of the Jan-Feb 1998 event noted in section 3.5, model calibration, must be shown. This information may be included on the Bear Creek at Lockeford frequency curve mentioned in item 1 above.

   PBI Response: A mention of the frequency of the calibration event is now included in Section 3.5. Note that the calibration location is approximately 7 miles downstream from the Lockeford gage. The Lockeford frequency curve was adjusted based on the proportional relationship between its drainage area and the drainage area at the calibration location. This adjusted curve was then used to estimate that the calibration event is approximately a 1/10 AEP event.

SPK backcheck: Clarify the duration of "effective rainfall" that produced the peak flow, in section 3.5. A 12-day duration sounds high relative to the 3-day (72-hour) duration selected for the current study.
PBI Response to Backcheck: Section 3.5 has been updated.

9. In section 3.6.1, Model Simulations, Existing Conditions, the statement is made that "any subbasin flows exceeding pump station capacities would result in temporary ponding within the subbasin." The study must be clear that this condition is not related to exterior stages in the receiving stream. Or a coincidence analysis must be performed to relate interior and exterior stages.

   PBI Response: Section 3.6.1 has been updated.

   SPK backcheck: OK

10. Subbasin 1105 (M1, M2, M3) from Mosher Slough must be added to the table in attachment A which compares the HEC1 and HMS results. This (1105) or these (M1, M2, M3) subbasin(s) should also be included in the final results tables.

    PBI Response: As noted above in Response #6, Bear Creek subbasins M1, M2, and M3 do not cover the same area as Mosher Slough subbasin 1105; these subbasins are adjacent to one another. Mosher Slough subbasin 1105 was therefore not included in any of the final results table because it is not part of the Bear Creek model.

    SPK backcheck: OK

11. The HMS model transmitted with the report does not appear to match either the results from the 1998 study or the calibration done for the current study. The report must clarify the purpose and the state of the input parameters contained in the model supplied.

    PBI Response: The HMS model transmitted with the report is not expected to match the results from the 1998 study or from the calibration run.

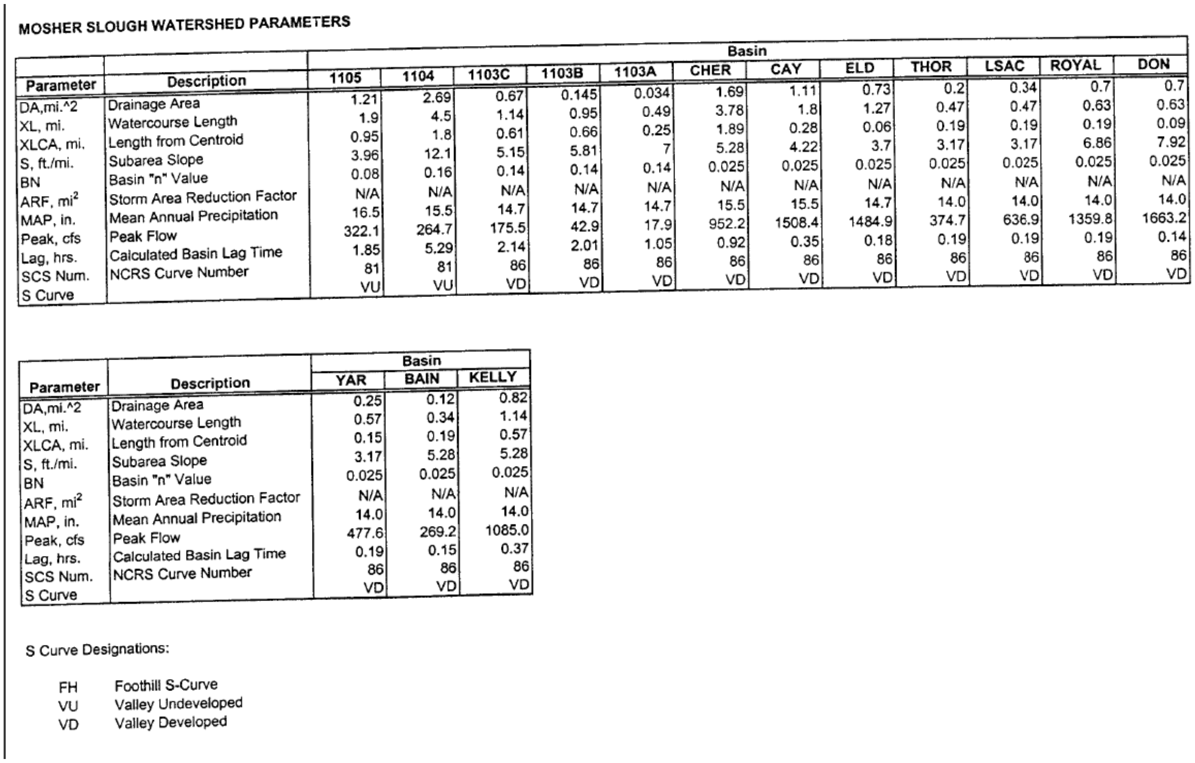
    The results table originally included in Attachment A was meant to show that the conversion from the HEC-1 model to HEC-HMS went smoothly. The HEC-HMS results listed in that table were for a model that still used the old methodology from the 1998 HEC-1 study (curve numbers, etc). This table has now been excluded from PBI's report so as to avoid any confusion.

The HMS model transmitted will also not produce the same results as the calibration shown in Figure 8. The calibration was run using observed rainfall data from a historical storm whereas the transmitted model is coded with the 100-year design storms that were used in the 1998 HEC-1 model.

The input parameters for the transmitted model are listed in Attachment D & Attachment E. Once the NOAA14 design storms are determined, they will be coded into the PBI Model.

SPK backcheck: OK

Attachment 4- A. Mosher Slough Watershed Subbasin Parameters Used in the 1998 SJAFCA HEC-1 Model



MOSHER SLOUGH WATERSHED PARAMETERS

		Basin											
Parameter	**Description**	**1105**	**1104**	**1103C**	**1103B**	**1103A**	**CHER**	**CAY**	**ELD**	**THOR**	**LSAC**	**ROYAL**	**DON**
DA,mi.^2	Drainage Area	1.21	2.69	0.67	0.145	0.034	1.69	1.11	0.73	0.2	0.34	0.7	0.7
XL, mi.	Watercourse Length	1.9	4.5	1.14	0.95	0.49	3.78	1.8	1.27	0.47	0.47	0.63	0.63
XLCA, mi.	Length from Centroid	0.95	1.8	0.61	0.66	0.25	1.89	0.28	0.06	0.19	0.19	0.19	0.09
S, ft./mi.	Subarea Slope	3.96	12.1	5.15	5.81	7	5.28	4.22	3.7	3.17	3.17	6.86	7.92
BN	Basin "n" Value	0.08	0.16	0.14	0.14	0.14	0.025	0.025	0.025	0.025	0.025	0.025	0.025
ARF, mi^2	Storm Area Reduction Factor	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
MAP, in.	Mean Annual Precipitation	16.5	15.5	14.7	14.7	14.7	15.5	15.5	14.7	14.0	14.0	14.0	14.0
Peak, cfs	Peak Flow	322.1	264.7	175.5	42.9	17.9	952.2	1508.4	1484.9	374.7	636.9	1359.8	1663.2
Lag, hrs.	Calculated Basin Lag Time	1.85	5.29	2.14	2.01	1.05	0.92	0.35	0.18	0.19	0.19	0.19	0.14
SCS Num.	NCRS Curve Number	81	81	86	86	86	86	86	86	86	86	86	86
S Curve		VU	VU	VD	VD	VD	VD	VD	VD	VD	VD	VD	VD

		Basin		
Parameter	**Description**	**YAR**	**BAIN**	**KELLY**
DA,mi.^2	Drainage Area	0.25	0.12	0.82
XL, mi.	Watercourse Length	0.57	0.34	1.14
XLCA, mi.	Length from Centroid	0.15	0.19	0.57
S, ft./mi.	Subarea Slope	3.17	5.28	5.28
BN	Basin "n" Value	0.025	0.025	0.025
ARF, mi^2	Storm Area Reduction Factor	N/A	N/A	N/A
MAP, in.	Mean Annual Precipitation	14.0	14.0	14.0
Peak, cfs	Peak Flow	477.6	269.2	1085.0
Lag, hrs.	Calculated Basin Lag Time	0.19	0.15	0.37
SCS Num.	NCRS Curve Number	86	86	86
S Curve		VD	VD	VD

S Curve Designations:

FH	Foothill S-Curve
VU	Valley Undeveloped
VD	Valley Developed

Source: HDR, Final Technical Memorandum #1, Hydrologic Report prepared for San Joaquin Area Flood Control Agency, January 1998.

Attachment 4- B. Mosher Slough Subbasin Soil Groups and Loss Rates

Soil Group:	A (0.35 in/hr)	B (0.2 in/hr)	C (0.1 in/hr)	D (0.025 in/hr)	Composite Loss Rate	Adjusted Loss Rate (Adjustment Factor = 0.80)
Subbasin	[sq. mi.]	[sq. mi.]	[sq. mi.]	[sq. mi.]	[in/hr]	[in/hr]
1104	0.00	0.01	0.43	2.82	0.035	0.028
1105	0.00	0.00	0.67	1.07	0.054	0.043
1103C	0.00	0.00	0.00	1.00	0.025	0.020
1103B	0.00	0.00	0.00	0.24	0.025	0.020
1103D	0.00	0.00	0.00	1.17	0.025	0.020
1103A	0.00	0.00	0.00	0.10	0.025	0.020
CHER	0.00	0.00	0.00	1.78	0.025	0.020
CAY	0.00	0.00	0.00	1.17	0.025	0.020
ELD	0.00	0.00	0.00	0.71	0.025	0.020
THOR	0.00	0.00	0.00	0.47	0.025	0.020
ROYAL	0.00	0.00	0.00	0.72	0.025	0.020
LSAC	0.00	0.00	0.00	0.35	0.025	0.020
DON	0.00	0.00	0.44	0.52	0.059	0.047
BAIN	0.00	0.00	0.14	0.00	0.100	0.080
KELLY	0.00	0.00	0.67	0.10	0.090	0.072
YAR	0.00	0.00	0.28	0.02	0.095	0.076
TCREEKS	0.00	0.00	0.17	0.00	0.100	0.080
ATLAS	0.00	0.00	0.47	0.00	0.100	0.080

Attachment 4- C. Mosher Slough Subbasin Characteristics – Existing Conditions

Basin	Area [Sq. Mi.]	Basin 'n' n	Watercourse Length L [miles]	Upstream Elevation [feet]	Downstream Elevation [feet]	Length from Centroid Lc [miles]	Watercourse Slope S [ft/mile]	Lag Time Lg [hrs]	S-Graph	Initial Loss [inches]	Constant Loss Rate [in/hour]	Impervious % [%]	Pump Station Capacity [cfs]
1104	3.25	0.15	3.66	54	35	1.44	5.20	4.95	VU	1.5	0.028	10	85.3
1105	1.75	0.2	2.26	64	50	0.77	6.18	4.19	VU	1.5	0.043	2	
1103C	1.04	0.015	2.18	40	31	1.27	4.13	0.40	VD	1.5	0.020	60	
1103B	0.24	0.015	1.07	35	30	0.60	4.68	0.23	VD	1.5	0.020	60	
1103D	1.17	0.015	1.19	30	26	0.44	3.36	0.22	VD	1.5	0.020	60	
1103A	0.10	0.015	0.59	30	26	0.31	6.82	0.13	VD	1.5	0.020	60	
CHER	1.78	0.015	1.50	25	20	0.48	3.34	0.25	VD	1.5	0.020	60	199.5
CAY	1.17	0.015	2.01	21	20	0.63	0.50	0.45	VD	1.5	0.020	60	269.2
ELD	0.71	0.015	1.55	20	16	0.77	2.57	0.32	VD	1.5	0.020	60	188.5
THOR	0.47	0.015	0.86	11	10	0.47	1.16	0.25	VD	1.5	0.020	60	26.8
ROYAL	0.73	0.015	0.88	19	13	0.17	6.80	0.12	VD	1.5	0.020	60	204.5
LSAC	0.35	0.015	0.75	20	15	0.40	6.67	0.16	VD	1.5	0.020	60	19.0
DON	0.96	0.015	1.26	14	1	0.55	10.28	0.20	VD	1.5	0.047	60	77.7
BAIN	0.14	0.015	0.69	2	1	0.37	1.46	0.20	VD	1.5	0.080	60	43.5
KELLY	0.79	0.015	1.16	10	1	0.61	7.76	0.21	VD	1.5	0.072	60	152.6
YAR	0.30	0.015	1.05	5	1	0.60	3.83	0.23	VD	1.5	0.076	60	82.1
TCREEKS	0.17	0.015	0.73	1	0	0.23	1.37	0.17	VD	1.5	0.080	60	34.8
ATLAS	0.51	0.115	1.09	1	0	0.59	0.92	2.37	VU	1.5	0.080	2	--

Notes: VU = Valley Undeveloped; VD = Valley Developed

Attachment 4- D. Mosher Slough Subbasin Characteristics – Future Conditions

Basin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
1104	3.25	0.15	3.66	54	35	1.44	5.20	4.95	VU	1.5	0.028	10	85.3
1105	1.75	0.20	2.26	64	50	0.77	6.18	4.19	VU	1.5	0.043	2	
1103C	1.04	0.015	2.18	40	31	1.27	4.13	0.40	VD	1.5	0.020	60	
1103B	0.24	0.015	1.07	35	30	0.60	4.68	0.23	VD	1.5	0.020	60	
1103D	1.17	0.015	1.19	30	26	0.44	3.36	0.22	VD	1.5	0.020	60	
1103A	0.10	0.015	0.59	30	26	0.31	6.82	0.13	VD	1.5	0.020	60	
CHER	1.78	0.015	1.50	25	20	0.48	3.34	0.25	VD	1.5	0.020	60	199.5
CAY	1.17	0.015	2.01	21	20	0.63	0.50	0.45	VD	1.5	0.020	60	269.2
ELD	0.71	0.015	1.55	20	16	0.77	2.57	0.32	VD	1.5	0.020	60	188.5
THOR	0.47	0.015	0.86	11	10	0.47	1.16	0.25	VD	1.5	0.020	60	26.8
ROYAL	0.73	0.015	0.88	19	13	0.17	6.80	0.12	VD	1.5	0.020	60	204.5
LSAC	0.35	0.015	0.75	20	15	0.40	6.67	0.16	VD	1.5	0.020	60	19.0
DON	0.96	0.015	1.26	14	1	0.55	10.28	0.20	VD	1.5	0.047	60	77.7
BAIN	0.14	0.015	0.69	2	1	0.37	1.46	0.20	VD	1.5	0.080	60	43.5
KELLY	0.79	0.015	1.16	10	1	0.61	7.76	0.21	VD	1.5	0.072	60	152.6
YAR	0.30	0.015	1.05	5	1	0.60	3.83	0.23	VD	1.5	0.076	60	82.1
TCREEKS	0.17	0.015	0.73	1	0	0.23	1.37	0.17	VD	1.5	0.080	60	34.8
ATLAS	0.51	0.015	1.09	1	0	0.59	0.92	0.31	VD	1.5	0.080	60	120.8

Notes: VU = Valley Undeveloped; VD = Valley Developed

Attachment 4- E. Mosher Slough Depth-Duration-Frequency Tables

MOSHER SLOUGH WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: SFS

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Urban Storm Centering Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
10 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
15 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
30 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
60 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
3 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
6 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
12 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
24 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
48 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
72 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
96 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894

Urban Storm Centering Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.120	0.157	0.187	0.227	0.257	0.288	0.320	0.362
10 min	0.173	0.225	0.268	0.325	0.369	0.413	0.459	0.519
15 min	0.208	0.273	0.325	0.393	0.446	0.500	0.554	0.628
30 min	0.286	0.375	0.445	0.540	0.612	0.686	0.761	0.862
60 min	0.396	0.518	0.616	0.747	0.848	0.949	1.053	1.193
3 hour	0.645	0.797	0.923	1.097	1.235	1.379	1.531	1.746
6 hour	0.879	1.067	1.224	1.443	1.617	1.799	1.993	2.267
12 hour	1.168	1.426	1.639	1.932	2.161	2.396	2.643	2.982
24 hour	1.603	1.983	2.292	2.711	3.032	3.358	3.694	4.149
48 hour	2.038	2.519	2.905	3.418	3.806	4.195	4.590	5.117
72 hour	2.345	2.893	3.330	3.909	4.342	4.775	5.211	5.791
96 hour	2.581	3.184	3.660	4.289	4.758	5.224	5.691	6.310

Attachment 4- F. ITR Comment Forms for Mosher Slough HEC-HMS Modeling

Lower San Joaquin River Feasibility Study

HYDROLOGIC MODELING REVIEW – MOSHER SLOUGH WATERSHED

Task Order 6 – LSJRFS Work-In-Kind Hydrology

Reviewer:	Domenichelli & Associates
Review Date:	9-14-10
PBI Response Date:	9-28-10
D&A Backcheck:	10-8-10

Backcheck Comments:

1. No additional comments on the revised memo and model. All comments have been addressed adequately.

2. A general observation about the Mosher Slough project is that interior drainage behind the levees (and floodwalls) was not thoroughly address in the original SJAFCA project. For the 100-yr event, the pumps will not keep up and significant storage occurs behind the levees (approx 50 ac-ft at Don Ave PS). May need to consider analyzing interior drainage and mapping potential interior floodplains later in the process.

   PBI Response: Agreed. Additional analysis would be required to assess the dynamics of interior drainage within Mosher Slough subbasins.

PETERSON . BRUSTAD . INC .
ENGINEERING . CONSULTING

Lower San Joaquin River Feasibility Study

HYDROLOGIC MODELING REVIEW – MOSHER SLOUGH WATERSHED

Task Order 6 – LSJRFS Work-In-Kind Hydrology

Reviewer: Domenichelli & Associates
Review Date: 9-14-10
PBI Response Date: 9-28-10

Note to Reviewer: Two subbasins (ATLAS & TCREEKS) were added to the PBI Model just west of Interstate-5. This extends the model to Mosher Slough's confluence with Bear Creek. All methodology for subbasin parameterization remained consistent with the rest of the subbasins.

Memorandum Comments:

1. *Page 7, Paragraph 1- Complete the Sentence*

PBI Response: Agreed.

2. *Section 4.4.2 – For a 72-hour event, the detention basins will likely fill requiring pumping back into the slough with the pump station capacities designed for the project. Be sure that both are modeled with pumping.*

PBI Response: Agreed. See Section 4.4.2.

3. *For more accurate modeling of the detention basins inflow and outflow, the final modeler should consider using the HEC-RAS Un-Steady State modeling routine. Side weirs and pumping rate information can be more accurately input into the RAS model than what is used in the HEC-HMS model.*

PBI Response: Agreed. The features in HEC-RAS can be used to perform analyses that cannot be completed in HMS. A HEC-RAS analysis is beyond the scope of this study. However, the hydraulics analysis for the LSJRFS planned for 2011 will include HEC-RAS unsteady modeling.

4. *Table 2, Consider providing a column for routing reach description (ie..Basin B7 to B6).*

PBI Response: Agreed.

6. *Section 4.4.6, Loss Rates- The initial loss rate is conservative (low) per last paragraph but the constant rate is at the mid to upper limit. Is this due to calibration? Why not be consistent with the Bear Creek rates. Again, may want to include a soils map somewhere for confirmation.*

PBI Response: The methodology for selecting constant loss rates has been modified. See Section 4.4.6.

7. *Section 4.5 Calibration- Calibrating to 790cfs at the location indicated provides results that are much higher than the SJAFCA model. Regional Equations are not very accurate and we would not recommend using them as a means for calibration. A translation of the parameters from the Bear Creek calibration seems more appropriate.*

PBI Response: Agreed. See Section 4.5.

Model Comments:

8. *Looking at the model results using the loss rates in Section 4.4.6 we would not expect the results to be almost double the SJAFCA model results for the sheds 1104 and 1105. We cannot find the reason for this large discrepancy. Please provide an explanation. Without the original SJAFCA model we cannot compare the input.*

PBI Response: With the model no longer being calibrated to 1/100 AEP peak flows calculated with the regression equation, combined peak flows coming from subbasins 1104 and 1105 are now only ~45% higher than the SJAFCA HEC-1 Model. This 45% difference is due not only to the different loss rate methodology used in the PBI Model, but also because subbasin areas and lag times were re-calculated for the PBI Model and differ from the SJAFCA HEC-1 Model.

9. *As stated in Comment #2 using the longer (72-hr) duration event and higher peak flows (per comment 8) and volumes, the volume of flow and timing into the detention basin must be checked. For the 100-yr event, Detention Basin #1 may fill at such a time that more than 230cfs will pass down Mosher Slough. Confirm that there is adequate storage to maintain the peak flow passing at 230cfs or if not how the peak 100-yr flows downstream will be impacted.*

PBI Response: Agreed. See Section 4.4.

Attachment 4- G. SPK Comment Forms for Mosher Slough HEC-HMS Modeling

Corps of Engineers, Hydrology Section, Review of Mosher Slough HEC-1 to HEC-HMS model conversion and preliminary report.

12 November 2010

Steven F. Holmstrom, P.E.

The Technical Memorandum for the Lower San Joaquin River Feasibility Study Mosher Slough HEC-HMS modeling DRAFT hydrology report has been reviewed and the following comments are provided.

12. In section 4.3 Design Storms, the fourth paragraph states that two storm centerings will be analyzed. A third storm centering will be required to compute the flow above the New Hogan Dam for the 8 design storm frequencies.

    PBI Response: It is PBI's understanding that the third storm centering above New Hogan Dam will only be analyzed for the Calaveras River watershed. No changes were made to this section.

13. In Section 4.4.2 Reservoirs and Pumps, it must be made clear that the pumps discharge into the receiving channel above the highest stage to be expected so that there is independence between the exterior and interior areas. If that is not the case then a coincidence analysis must be performed to determine the modified interior pond stage-frequency curve considering the exterior-interior stage conditions. This is explained in EM1110-2-1413, Hydrologic Analysis of Interior Areas.

    PBI Response: Section 4.4.2 has been updated.

14. In figure 5, Mosher Slough subbasin flowpaths, the blue line representing the flowpath should exit into Bear Creek as the entire subarea is diverted for all flow frequencies.

    PBI Response: The flowpath for the Atlas Tract has been changed and now exits into Bear Creek. The lag time calculation has also been updated for this subbasin.

15. In section 4.6, Model Simulations, Existing Conditions, the statement is made that "any subbasin flows exceeding pump station capacities would result in temporary ponding within the subbasin." The study must be clear that this condition is not related to exterior stages in the receiving stream. Or a coincidence analysis must be performed to relate interior and exterior stages.

    PBI Response: Section 4.6 has been updated.

16. Subbasin 1103D must be added to the table in attachment A which compares the HEC1 and HMS results.

    PBI Response: Subbasin 1103D was left out of the 1998 HEC-1 report's results table (perhaps unintentionally) and therefore was not able to be compared to the HEC-HMS results.

    The results table originally included in Attachment A was intended to show that the conversion from the HEC-1 model to HEC-HMS went smoothly. The HEC-HMS results listed in that table were for a model that still used the old methodology from the 1998 HEC-1 study (curve numbers, etc). This table has now been excluded from PBI's report so as to avoid any confusion. Attachment A now includes a table of the parameters used in the 1998 SJAFCA HEC-1 model.

17. The HMS model transmitted with the report does not appear to match either the results from the 1998 study or the calibration done for the current study. The report must clarify the purpose and the state of the input parameters contained in the model supplied.

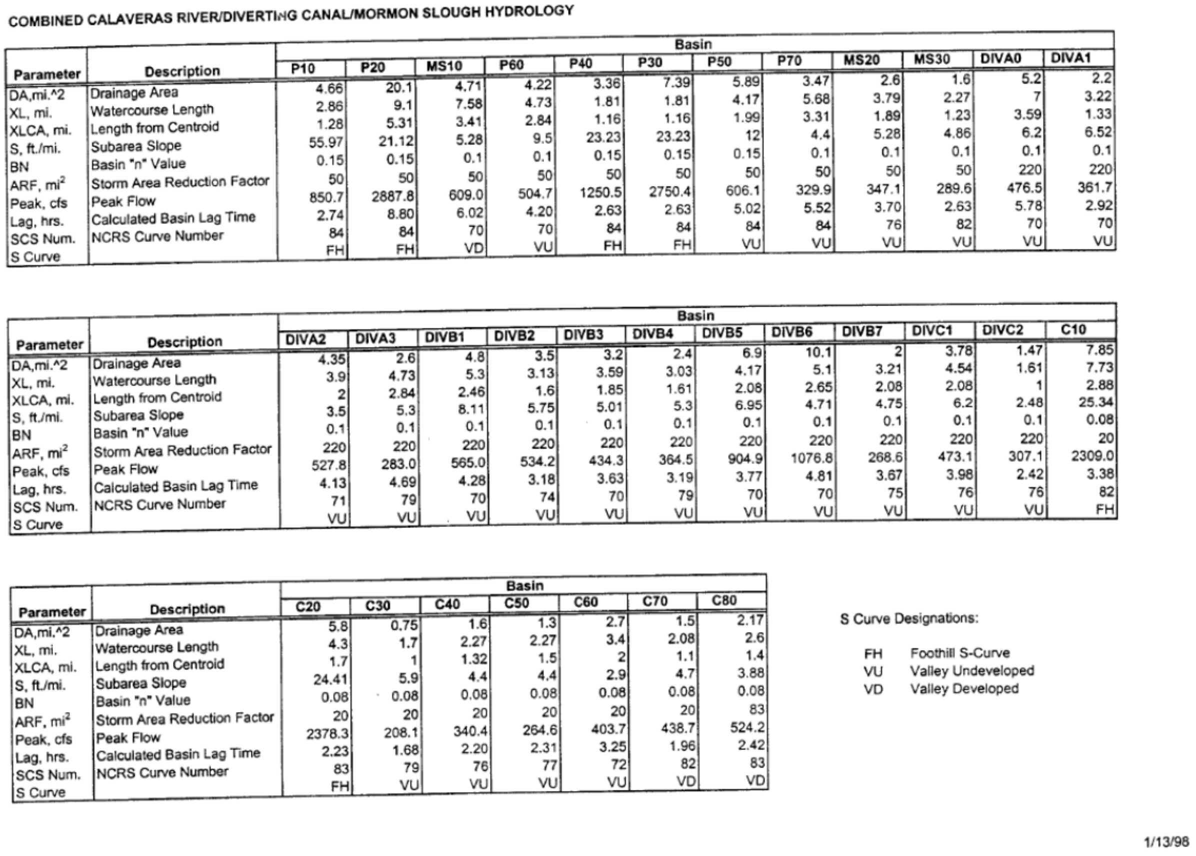
    PBI Response: The HMS model transmitted with the report is not expected to match the results from the 1998 study or from the calibration run.

    As previously mentioned, the results table originally included in Attachment A was meant to show that the conversion from the HEC-1 model to HEC-HMS went smoothly. The HEC-HMS results listed in that table were for a model that still used the old methodology from the 1998 HEC-1 study (curve numbers, etc). This table has now been excluded from PBI's report so as to avoid any confusion.

    Calibration flow results were not included in the Mosher Slough report. As mentioned in Section 4.5, this model was calibrated based on the loss rate adjustment factor determined in the Bear Creek calibration.

    The input parameters for the transmitted model are listed in Attachment C & Attachment D.

Attachment 5- A. Calaveras River Watershed Subbasin Parameters Used in the 1998 SJAFCA HEC-1 Model



COMBINED CALAVERAS RIVER/DIVERTING CANAL/MORMON SLOUGH HYDROLOGY

Parameter	Description	Basin											
		P10	P20	MS10	P60	P40	P30	P50	P70	MS20	MS30	DIVA0	DIVA1
DA,mi.^2	Drainage Area	4.66	20.1	4.71	4.22	3.36	7.39	5.89	3.47	2.6	1.6	5.2	2.2
XL, mi.	Watercourse Length	2.86	9.1	7.58	4.73	1.81	1.81	4.17	5.68	3.79	2.27	7	3.22
XLCA, mi.	Length from Centroid	1.28	5.31	3.41	2.84	1.16	1.16	1.99	3.31	1.89	1.23	3.59	1.33
S, ft./mi.	Subarea Slope	55.97	21.12	5.28	9.5	23.23	23.23	12	4.4	5.28	4.86	6.2	6.52
BN	Basin "n" Value	0.15	0.15	0.1	0.1	0.15	0.15	0.15	0.1	0.1	0.1	0.1	0.1
ARF, mi^2	Storm Area Reduction Factor	50	50	50	50	50	50	50	50	50	50	220	220
Peak, cfs	Peak Flow	850.7	2887.8	609.0	504.7	1250.5	2750.4	606.1	329.9	347.1	289.6	476.5	361.7
Lag, hrs.	Calculated Basin Lag Time	2.74	8.80	6.02	4.20	2.63	2.63	5.02	5.52	3.70	2.63	5.78	2.92
SCS Num.	NCRS Curve Number	84	84	70	70	84	84	84	84	76	82	70	70
S Curve		FH	FH	VD	VU	FH	FH	VU	VU	VU	VU	VU	VU

Parameter	Description	Basin											
		DIVA2	DIVA3	DIVB1	DIVB2	DIVB3	DIVB4	DIVB5	DIVB6	DIVB7	DIVC1	DIVC2	C10
DA,mi.^2	Drainage Area	4.35	2.6	4.8	3.5	3.2	2.4	6.9	10.1	2	3.78	1.47	7.85
XL, mi.	Watercourse Length	3.9	4.73	5.3	3.13	3.59	3.03	4.17	5.1	3.21	4.54	1.61	7.73
XLCA, mi.	Length from Centroid	2	2.84	2.46	1.6	1.85	1.61	2.08	2.65	2.08	2.08	1	2.88
S, ft./mi.	Subarea Slope	3.5	5.3	8.11	5.75	5.01	5.3	6.95	4.71	4.75	6.2	2.48	25.34
BN	Basin "n" Value	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.08
ARF, mi^2	Storm Area Reduction Factor	220	220	220	220	220	220	220	220	220	220	220	20
Peak, cfs	Peak Flow	527.8	283.0	565.0	534.2	434.3	364.5	904.9	1076.8	268.6	473.1	307.1	2309.0
Lag, hrs.	Calculated Basin Lag Time	4.13	4.69	4.28	3.18	3.63	3.19	3.77	4.81	3.67	3.98	2.42	3.38
SCS Num.	NCRS Curve Number	71	79	70	74	70	79	70	70	75	76	76	82
S Curve		VU	VU	VU	VU	VU	VU	VU	VU	VU	VU	VU	FH

Parameter	Description	Basin						
		C20	C30	C40	C50	C60	C70	C80
DA,mi.^2	Drainage Area	5.8	0.75	1.6	1.3	2.7	1.5	2.17
XL, mi.	Watercourse Length	4.3	1.7	2.27	2.27	3.4	2.08	2.6
XLCA, mi.	Length from Centroid	1.7	1	1.32	1.5	2	1.1	1.4
S, ft./mi.	Subarea Slope	24.41	5.9	4.4	4.4	2.9	4.7	3.88
BN	Basin "n" Value	0.08	0.08	0.08	0.08	0.08	0.08	0.08
ARF, mi^2	Storm Area Reduction Factor	20	20	20	20	20	20	83
Peak, cfs	Peak Flow	2378.3	208.1	340.4	264.6	403.7	438.7	524.2
Lag, hrs.	Calculated Basin Lag Time	2.23	1.68	2.20	2.31	3.25	1.96	2.42
SCS Num.	NCRS Curve Number	83	79	76	77	72	82	83
S Curve		FH	VU	VU	VU	VU	VD	VD

S Curve Designations:

FH	Foothill S-Curve
VU	Valley Undeveloped
VD	Valley Developed

1/13/98

Source: HDR, Final Technical Memorandum #1, Hydrologic Report prepared for San Joaquin Area Flood Control Agency, January 1998.

Attachment 5- B. Calaveras River Subbasin Characteristics

Basin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
BL10	72.63	0.15	14.81	700	142	7.83	37.69	10.99	FH	1.5	0.052	2	
CG10	21.35	0.15	9.65	1550	500	5.74	108.76	6.79	FH	1.5	0.059	5	
NH10	1.19	0.15	1.90	850	500	0.87	183.94	1.62	FH	1.5	0.065	2	
DUCK	9.76	0.15	4.21	430	200	2.35	54.63	4.02	FH	1.5	0.028	2	
MS10	4.09	0.2	6.90	123	81	3.81	6.08	11.80	VU	1.5	0.158	2	
P60	1.80	0.2	4.48	104	77	1.82	6.02	7.58	VU	1.5	0.114	2	
P20	19.85	0.2	9.82	320	101	5.23	22.30	11.89	FH	1.5	0.035	2	
P10	5.64	0.2	4.34	320	142	2.17	41.02	5.56	FH	1.5	0.033	2	
P50	10.62	0.2	5.63	100	75	1.80	4.44	8.72	VU	1.5	0.037	2	
P30	4.85	0.2	2.86	238	127	1.30	38.86	3.95	VU	1.5	0.024	2	
P40	4.05	0.2	2.91	224	105	0.75	40.90	3.19	VU	1.5	0.021	2	
MS20	3.10	0.2	4.32	81	51	2.42	6.94	8.11	VU	1.5	0.063	2	
P70	2.33	0.2	4.17	77	51	2.56	6.24	8.33	VU	1.5	0.021	2	
MS30	1.47	0.2	2.22	53	32	1.17	9.44	4.50	VU	1.5	0.021	2	
DIVA2	5.56	0.2	3.56	86	62	1.72	6.74	6.65	VU	1.5	0.096	2	16.0
DIVA1	2.09	0.2	2.92	103	80	0.95	7.87	4.77	VU	1.5	0.136	2	
DIVA3	3.96	0.15	5.17	69	30	2.57	7.54	6.55	VU	1.5	0.064	5	
DIVA0	3.37	0.2	5.82	73	34	2.60	6.70	9.39	VU	1.5	0.026	2	
DIVB6	6.87	0.2	4.36	70	46	2.04	5.51	7.96	VU	1.5	0.115	2	100.0
DIVB5	6.85	0.2	3.42	98	65	1.48	9.65	5.78	VU	1.5	0.130	2	
DIVB2	3.85	0.2	4.31	96	65	1.92	7.19	7.36	VU	1.5	0.114	2	
DIVB1	3.44	0.2	3.28	115	86	1.59	8.84	5.93	VU	1.5	0.148	2	
DIVB3	3.87	0.2	3.31	66	46	1.46	6.04	6.22	VU	1.5	0.100	2	
DIVB4	1.96	0.15	3.13	48	29	1.73	6.07	4.85	VU	1.5	0.047	5	
DIVB7	2.09	0.15	3.14	45	29	1.56	5.09	4.83	VU	1.5	0.063	5	
SANG	0.49	0.015	1.31	20	19	1.31	0.76	0.46	VD	1.5	0.045	60	92.2
C10	7.76	0.2	6.00	284	124	2.75	26.68	7.46	FH	1.5	0.031	0	
C20	7.11	0.2	6.08	195	95	3.26	16.45	8.77	VU	1.5	0.074	2	
C30	1.71	0.2	3.83	96	72	1.66	6.26	6.85	VU	1.5	0.128	2	

Basin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
C40	2.19	0.2	2.57	74	56	1.16	7.01	5.02	VU	1.5	0.108	2	
C60	3.75	0.15	3.61	53	33	1.52	5.54	4.97	VU	1.5	0.092	5	
C50	1.63	0.2	1.99	59	48	0.95	5.52	4.42	VU	1.5	0.078	2	
C70	2.26	0.1	2.88	41	30	1.38	3.82	3.14	VU	1.5	0.022	15	
HOLM	1.73	0.02	1.91	30	26	0.69	2.10	0.46	VD	1.5	0.028	30	140.1
DIVC1	2.39	0.15	3.13	48	1	1.60	15.01	3.97	VU	1.5	0.060	5	
DIVC2	1.21	0.15	1.80	29	1	0.88	15.53	2.54	VD	1.5	0.041	15	
WLN	0.71	0.015	1.55	25	20	0.65	3.23	0.29	VD	1.5	0.028	60	47.5
WLS	0.36	0.015	1.28	17	16	0.42	0.78	0.30	VD	1.5	0.021	60	254.0
BCHI	1.27	0.015	1.67	20	15	0.52	3.00	0.28	VD	1.5	0.021	60	176.9
SUT	0.67	0.015	1.32	17	15	0.52	1.52	0.29	VD	1.5	0.021	60	54.1
BSTG	0.45	0.015	1.09	12	8	0.55	3.67	0.23	VD	1.5	0.023	60	132.7
RWLK	0.02	0.015	0.18	9	7	0.05	10.99	0.04	VD	1.5	0.158	60	10.5
MBRK	0.61	0.015	0.99	1	0	0.32	1.01	0.23	VD	1.5	0.058	60	6.0
PLYM	0.10	0.015	0.57	1	0	0.26	1.77	0.16	VD	1.5	0.080	60	121.3
HGTY	0.10	0.015	0.57	6	5	0.26	1.77	0.16	VD	1.5	0.109	60	6.2
BRES	0.42	0.02	1.44	1	0	0.61	0.70	0.49	VD	1.5	0.085	40	39.0
KIRK	0.05	0.015	0.31	3	1	0.17	6.41	0.08	VD	1.5	0.085	60	14.5
WISC	1.14	0.015	1.56	5	3	0.37	1.28	0.28	VD	1.5	0.078	60	21.7

Notes: FH = Foothill; VU = Valley Undeveloped; VD = Valley Developed

Attachment 5- C. Calaveras River Subbasin Soil Groups and Loss Rates

Soil Group:	A (0.35 in/hr)	B (0.2 in/hr)	C (0.1 in/hr)	D (0.025 in/hr)	Composite Loss Rate	Adjusted Loss Rate (Adjustment Factor = 0.85)
Subbasin	[sq. mi.]	[sq. mi.]	[sq. mi.]	[sq. mi.]	[in/hr]	[in/hr]
BL10	0.47	12.31	3.19	55.57	0.061	0.052
CG10	0.00	4.47	1.83	14.16	0.070	0.059
NH10	0.00	0.10	0.54	0.49	0.076	0.065
DUCK	0.00	0.07	0.93	8.68	0.034	0.028
MS10	0.00	3.47	0.28	0.16	0.186	0.158
P60	0.00	0.72	0.93	0.14	0.134	0.114
P20	0.00	1.72	0.10	17.89	0.041	0.035
P10	0.15	0.00	0.39	4.98	0.039	0.033
P50	0.00	0.86	0.65	9.10	0.044	0.037
P30	0.00	0.10	0.00	4.75	0.028	0.024
P40	0.00	0.00	0.00	4.05	0.025	0.021
MS20	0.00	0.11	1.68	1.20	0.074	0.063
P70	0.00	0.00	0.01	2.32	0.025	0.021
MS30	0.00	0.00	0.00	1.41	0.025	0.021
DIVA2	0.00	1.06	4.04	0.46	0.113	0.096
DIVA1	0.00	1.26	0.83	0.00	0.160	0.136
DIVA3	0.00	0.02	2.63	1.29	0.076	0.064
DIVA0	0.00	0.00	0.24	3.07	0.030	0.026
DIVB6	0.00	2.44	4.42	0.01	0.135	0.115
DIVB5	0.00	3.62	3.23	0.00	0.153	0.130
DIVB2	0.00	1.29	2.56	0.00	0.134	0.114
DIVB1	0.00	2.63	0.70	0.11	0.174	0.148
DIVB3	0.00	0.69	3.18	0.00	0.118	0.100
DIVB4	0.00	0.00	0.78	1.14	0.055	0.047
DIVB7	0.00	0.00	1.23	0.67	0.074	0.063
SANG	0.00	0.00	0.18	0.30	0.053	0.045
C10	0.00	0.04	1.08	6.39	0.037	0.031
C20	0.00	1.83	1.45	3.68	0.087	0.074
C30	0.00	0.82	0.79	0.00	0.151	0.128
C40	0.00	0.78	1.08	0.28	0.127	0.108
C60	0.00	1.54	0.50	1.66	0.108	0.092
C50	0.00	0.37	0.56	0.67	0.091	0.078
C70	0.00	0.00	0.02	2.19	0.026	0.022
HOLM	0.00	0.00	0.18	1.54	0.033	0.028
DIVC1	0.00	0.04	1.30	0.95	0.070	0.060
DIVC2	0.00	0.03	0.19	0.60	0.049	0.041
WLN	0.00	0.00	0.08	0.63	0.033	0.028
WLS	0.00	0.00	0.00	0.36	0.025	0.021
BCHI	0.00	0.00	0.00	1.27	0.025	0.021
SUT	0.00	0.00	0.00	0.65	0.025	0.021
BSTG	0.00	0.00	0.01	0.44	0.027	0.023
RWLK	0.00	0.02	0.00	0.00	0.186	0.158
MBRK	0.00	0.00	0.35	0.25	0.069	0.058
PLYM	0.00	0.00	0.09	0.01	0.094	0.080
HGTY	0.00	0.03	0.07	0.00	0.128	0.109
BRES	0.00	0.00	0.40	0.00	0.100	0.085
KIRK	0.00	0.00	0.04	0.00	0.100	0.085
WISC	0.00	0.00	0.96	0.12	0.092	0.078

Attachment 5- D. Calaveras River Depth-Duration-Frequency Tables

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SFS

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
10 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
15 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
30 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
60 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
3 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
6 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
12 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
24 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
48 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
72 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
96 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.120	0.157	0.187	0.227	0.257	0.288	0.320	0.362
10 min	0.173	0.225	0.268	0.325	0.369	0.413	0.459	0.519
15 min	0.208	0.273	0.325	0.393	0.446	0.500	0.554	0.628
30 min	0.286	0.375	0.445	0.540	0.612	0.686	0.761	0.862
60 min	0.396	0.518	0.616	0.747	0.848	0.949	1.053	1.193
3 hour	0.645	0.797	0.923	1.097	1.235	1.379	1.531	1.746
6 hour	0.879	1.067	1.224	1.443	1.617	1.799	1.993	2.267
12 hour	1.168	1.426	1.639	1.932	2.161	2.396	2.643	2.982
24 hour	1.603	1.983	2.292	2.711	3.032	3.358	3.694	4.149
48 hour	2.038	2.519	2.905	3.418	3.806	4.195	4.590	5.117
72 hour	2.345	2.893	3.330	3.909	4.342	4.775	5.211	5.791
96 hour	2.581	3.184	3.660	4.289	4.758	5.224	5.691	6.310

CALAVERAS RIVER WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: SFS

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.497	0.510	0.507	0.504	0.505	0.504	0.504	0.502
10 min	0.503	0.504	0.507	0.504	0.504	0.503	0.502	0.500
15 min	0.503	0.504	0.505	0.506	0.505	0.504	0.502	0.500
30 min	0.502	0.505	0.506	0.504	0.505	0.503	0.501	0.498
60 min	0.504	0.507	0.508	0.507	0.506	0.505	0.504	0.501
3 hour	0.501	0.500	0.499	0.498	0.496	0.495	0.493	0.492
6 hour	0.496	0.494	0.493	0.491	0.490	0.489	0.488	0.487
12 hour	0.491	0.489	0.487	0.485	0.485	0.484	0.483	0.482
24 hour	0.486	0.484	0.483	0.482	0.481	0.481	0.481	0.481
48 hour	0.482	0.480	0.479	0.477	0.477	0.477	0.477	0.477
72 hour	0.480	0.477	0.475	0.474	0.474	0.474	0.473	0.474
96 hour	0.479	0.475	0.474	0.473	0.472	0.472	0.472	0.473

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.067	0.090	0.106	0.128	0.145	0.162	0.180	0.203
10 min	0.097	0.127	0.152	0.183	0.208	0.232	0.258	0.291
15 min	0.117	0.154	0.183	0.223	0.252	0.282	0.311	0.351
30 min	0.161	0.212	0.252	0.304	0.346	0.386	0.426	0.480
60 min	0.223	0.294	0.350	0.424	0.480	0.536	0.594	0.669
3 hour	0.362	0.446	0.515	0.611	0.685	0.763	0.845	0.961
6 hour	0.488	0.589	0.675	0.792	0.886	0.984	1.088	1.235
12 hour	0.642	0.780	0.893	1.048	1.172	1.297	1.428	1.608
24 hour	0.871	1.074	1.238	1.461	1.632	1.807	1.987	2.232
48 hour	1.099	1.353	1.556	1.824	2.031	2.238	2.449	2.730
72 hour	1.259	1.544	1.769	2.072	2.302	2.532	2.757	3.071
96 hour	1.383	1.691	1.941	2.269	2.512	2.758	3.005	3.338

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SFS

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.497	0.510	0.507	0.504	0.505	0.504	0.504	0.502
10 min	0.503	0.504	0.507	0.504	0.504	0.503	0.502	0.500
15 min	0.503	0.504	0.505	0.506	0.505	0.504	0.502	0.500
30 min	0.502	0.505	0.506	0.504	0.505	0.503	0.501	0.498
60 min	0.504	0.507	0.508	0.507	0.506	0.505	0.504	0.501
3 hour	0.501	0.500	0.499	0.498	0.496	0.495	0.493	0.492
6 hour	0.496	0.494	0.493	0.491	0.490	0.489	0.488	0.487
12 hour	0.491	0.489	0.487	0.485	0.485	0.484	0.483	0.482
24 hour	0.486	0.484	0.483	0.482	0.481	0.481	0.481	0.481
48 hour	0.482	0.480	0.479	0.477	0.477	0.477	0.477	0.477
72 hour	0.480	0.477	0.475	0.474	0.474	0.474	0.473	0.474
96 hour	0.479	0.475	0.474	0.473	0.472	0.472	0.472	0.473

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.067	0.090	0.106	0.128	0.145	0.162	0.180	0.203
10 min	0.097	0.127	0.152	0.183	0.208	0.232	0.258	0.291
15 min	0.117	0.154	0.183	0.223	0.252	0.282	0.311	0.351
30 min	0.161	0.212	0.252	0.304	0.346	0.386	0.426	0.480
60 min	0.223	0.294	0.350	0.424	0.480	0.536	0.594	0.669
3 hour	0.362	0.446	0.515	0.611	0.685	0.763	0.845	0.961
6 hour	0.488	0.589	0.675	0.792	0.886	0.984	1.088	1.235
12 hour	0.642	0.780	0.893	1.048	1.172	1.297	1.428	1.608
24 hour	0.871	1.074	1.238	1.461	1.632	1.807	1.987	2.232
48 hour	1.099	1.353	1.556	1.824	2.031	2.238	2.449	2.730
72 hour	1.259	1.544	1.769	2.072	2.302	2.532	2.757	3.071
96 hour	1.383	1.691	1.941	2.269	2.512	2.758	3.005	3.338

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SFS

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Average Storm Centering Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.497	0.510	0.507	0.504	0.505	0.504	0.504	0.502
10 min	0.503	0.504	0.507	0.504	0.504	0.503	0.502	0.500
15 min	0.503	0.504	0.505	0.506	0.505	0.504	0.502	0.500
30 min	0.502	0.505	0.506	0.504	0.505	0.503	0.501	0.498
60 min	0.504	0.507	0.508	0.507	0.506	0.505	0.504	0.501
3 hour	0.501	0.500	0.499	0.498	0.496	0.495	0.493	0.492
6 hour	0.496	0.494	0.493	0.491	0.490	0.489	0.488	0.487
12 hour	0.491	0.489	0.487	0.485	0.485	0.484	0.483	0.482
24 hour	0.486	0.484	0.483	0.482	0.481	0.481	0.481	0.481
48 hour	0.482	0.480	0.479	0.477	0.477	0.477	0.477	0.477
72 hour	0.480	0.477	0.475	0.474	0.474	0.474	0.473	0.474
96 hour	0.479	0.475	0.474	0.473	0.472	0.472	0.472	0.473

Average Storm Centering Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.067	0.090	0.106	0.128	0.145	0.162	0.180	0.203
10 min	0.097	0.127	0.152	0.183	0.208	0.232	0.258	0.291
15 min	0.117	0.154	0.183	0.223	0.252	0.282	0.311	0.351
30 min	0.161	0.212	0.252	0.304	0.346	0.386	0.426	0.480
60 min	0.223	0.294	0.350	0.424	0.480	0.536	0.594	0.669
3 hour	0.362	0.446	0.515	0.611	0.685	0.763	0.845	0.961
6 hour	0.488	0.589	0.675	0.792	0.886	0.984	1.088	1.235
12 hour	0.642	0.780	0.893	1.048	1.172	1.297	1.428	1.608
24 hour	0.871	1.074	1.238	1.461	1.632	1.807	1.987	2.232
48 hour	1.099	1.353	1.556	1.824	2.031	2.238	2.449	2.730
72 hour	1.259	1.544	1.769	2.072	2.302	2.532	2.757	3.071
96 hour	1.383	1.691	1.941	2.269	2.512	2.758	3.005	3.338

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.768	0.766	0.766	0.768	0.767	0.768	0.768	0.768
10 min	0.768	0.768	0.767	0.768	0.768	0.768	0.768	0.768
15 min	0.768	0.768	0.767	0.767	0.767	0.767	0.768	0.769
30 min	0.767	0.767	0.767	0.768	0.768	0.768	0.768	0.769
60 min	0.767	0.767	0.767	0.767	0.767	0.768	0.768	0.769
3 hour	0.765	0.766	0.766	0.766	0.767	0.768	0.769	0.769
6 hour	0.764	0.764	0.765	0.766	0.766	0.767	0.768	0.769
12 hour	0.763	0.764	0.765	0.766	0.766	0.767	0.767	0.768
24 hour	0.763	0.764	0.765	0.766	0.766	0.766	0.766	0.767
48 hour	0.762	0.763	0.763	0.764	0.764	0.765	0.765	0.765
72 hour	0.761	0.762	0.762	0.763	0.763	0.764	0.764	0.764
96 hour	0.761	0.761	0.762	0.762	0.763	0.763	0.763	0.764

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.101	0.131	0.156	0.191	0.218	0.245	0.273	0.313
10 min	0.144	0.188	0.224	0.274	0.312	0.352	0.392	0.449
15 min	0.174	0.227	0.271	0.331	0.377	0.424	0.475	0.544
30 min	0.239	0.312	0.373	0.455	0.518	0.584	0.652	0.747
60 min	0.330	0.432	0.515	0.628	0.716	0.807	0.902	1.033
3 hour	0.516	0.641	0.748	0.904	1.032	1.170	1.321	1.538
6 hour	0.682	0.838	0.975	1.171	1.333	1.509	1.701	1.981
12 hour	0.897	1.118	1.303	1.563	1.769	1.986	2.214	2.536
24 hour	1.237	1.566	1.834	2.197	2.473	2.754	3.043	3.438
48 hour	1.535	1.930	2.247	2.676	3.002	3.334	3.670	4.122
72 hour	1.730	2.163	2.511	2.982	3.338	3.703	4.071	4.568
96 hour	1.895	2.360	2.738	3.241	3.627	4.012	4.405	4.939

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.500	0.506	0.503	0.508	0.503	0.507	0.508	0.505
10 min	0.506	0.508	0.506	0.508	0.508	0.506	0.505	0.503
15 min	0.507	0.508	0.505	0.507	0.506	0.505	0.506	0.505
30 min	0.501	0.505	0.506	0.505	0.506	0.505	0.504	0.503
60 min	0.504	0.506	0.508	0.508	0.507	0.507	0.506	0.505
3 hour	0.492	0.493	0.493	0.493	0.494	0.495	0.496	0.498
6 hour	0.481	0.482	0.481	0.483	0.484	0.486	0.487	0.491
12 hour	0.473	0.473	0.474	0.475	0.476	0.477	0.479	0.480
24 hour	0.467	0.469	0.470	0.471	0.472	0.473	0.473	0.474
48 hour	0.457	0.458	0.458	0.459	0.460	0.461	0.462	0.464
72 hour	0.450	0.449	0.448	0.450	0.451	0.452	0.454	0.456
96 hour	0.447	0.445	0.445	0.446	0.447	0.449	0.450	0.453

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.066	0.087	0.103	0.126	0.143	0.162	0.181	0.206
10 min	0.095	0.124	0.148	0.181	0.206	0.232	0.258	0.294
15 min	0.115	0.150	0.178	0.219	0.248	0.279	0.313	0.357
30 min	0.156	0.206	0.246	0.299	0.342	0.384	0.428	0.488
60 min	0.217	0.285	0.341	0.416	0.474	0.533	0.594	0.678
3 hour	0.332	0.413	0.482	0.582	0.665	0.754	0.852	0.996
6 hour	0.430	0.529	0.613	0.739	0.842	0.956	1.079	1.265
12 hour	0.556	0.692	0.807	0.969	1.099	1.235	1.382	1.585
24 hour	0.757	0.961	1.127	1.351	1.524	1.700	1.879	2.124
48 hour	0.920	1.158	1.349	1.608	1.807	2.009	2.216	2.500
72 hour	1.023	1.275	1.476	1.759	1.973	2.191	2.419	2.726
96 hour	1.113	1.380	1.599	1.897	2.125	2.361	2.598	2.929

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.500	0.506	0.503	0.508	0.503	0.507	0.508	0.505
10 min	0.506	0.508	0.506	0.508	0.508	0.506	0.505	0.503
15 min	0.507	0.508	0.505	0.507	0.506	0.505	0.506	0.505
30 min	0.501	0.505	0.506	0.505	0.506	0.505	0.504	0.503
60 min	0.504	0.506	0.508	0.508	0.507	0.507	0.506	0.505
3 hour	0.492	0.493	0.493	0.493	0.494	0.495	0.496	0.498
6 hour	0.481	0.482	0.481	0.483	0.484	0.486	0.487	0.491
12 hour	0.473	0.473	0.474	0.475	0.476	0.477	0.479	0.480
24 hour	0.467	0.469	0.470	0.471	0.472	0.473	0.473	0.474
48 hour	0.457	0.458	0.458	0.459	0.460	0.461	0.462	0.464
72 hour	0.450	0.449	0.448	0.450	0.451	0.452	0.454	0.456
96 hour	0.447	0.445	0.445	0.446	0.447	0.449	0.450	0.453

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.066	0.087	0.103	0.126	0.143	0.162	0.181	0.206
10 min	0.095	0.124	0.148	0.181	0.206	0.232	0.258	0.294
15 min	0.115	0.150	0.178	0.219	0.248	0.279	0.313	0.357
30 min	0.156	0.206	0.246	0.299	0.342	0.384	0.428	0.488
60 min	0.217	0.285	0.341	0.416	0.474	0.533	0.594	0.678
3 hour	0.332	0.413	0.482	0.582	0.665	0.754	0.852	0.996
6 hour	0.430	0.529	0.613	0.739	0.842	0.956	1.079	1.265
12 hour	0.556	0.692	0.807	0.969	1.099	1.235	1.382	1.585
24 hour	0.757	0.961	1.127	1.351	1.524	1.700	1.879	2.124
48 hour	0.920	1.158	1.349	1.608	1.807	2.009	2.216	2.500
72 hour	1.023	1.275	1.476	1.759	1.973	2.191	2.419	2.726
96 hour	1.113	1.380	1.599	1.897	2.125	2.361	2.598	2.929

CALAVERAS RIVER WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.500	0.506	0.503	0.508	0.503	0.507	0.508	0.505
10 min	0.506	0.508	0.506	0.508	0.508	0.506	0.505	0.503
15 min	0.507	0.508	0.505	0.507	0.506	0.505	0.506	0.505
30 min	0.501	0.505	0.506	0.505	0.506	0.505	0.504	0.503
60 min	0.504	0.506	0.508	0.508	0.507	0.507	0.506	0.505
3 hour	0.492	0.493	0.493	0.493	0.494	0.495	0.496	0.498
6 hour	0.481	0.482	0.481	0.483	0.484	0.486	0.487	0.491
12 hour	0.473	0.473	0.474	0.475	0.476	0.477	0.479	0.480
24 hour	0.467	0.469	0.470	0.471	0.472	0.473	0.473	0.474
48 hour	0.457	0.458	0.458	0.459	0.460	0.461	0.462	0.464
72 hour	0.450	0.449	0.448	0.450	0.451	0.452	0.454	0.456
96 hour	0.447	0.445	0.445	0.446	0.447	0.449	0.450	0.453

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.066	0.087	0.103	0.126	0.143	0.162	0.181	0.206
10 min	0.095	0.124	0.148	0.181	0.206	0.232	0.258	0.294
15 min	0.115	0.150	0.178	0.219	0.248	0.279	0.313	0.357
30 min	0.156	0.206	0.246	0.299	0.342	0.384	0.428	0.488
60 min	0.217	0.285	0.341	0.416	0.474	0.533	0.594	0.678
3 hour	0.332	0.413	0.482	0.582	0.665	0.754	0.852	0.996
6 hour	0.430	0.529	0.613	0.739	0.842	0.956	1.079	1.265
12 hour	0.556	0.692	0.807	0.969	1.099	1.235	1.382	1.585
24 hour	0.757	0.961	1.127	1.351	1.524	1.700	1.879	2.124
48 hour	0.920	1.158	1.349	1.608	1.807	2.009	2.216	2.500
72 hour	1.023	1.275	1.476	1.759	1.973	2.191	2.419	2.726
96 hour	1.113	1.380	1.599	1.897	2.125	2.361	2.598	2.929

CALAVERAS RIVER WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: FRM

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.621	0.620	0.626	0.626	0.627	0.626	0.625	0.624
10 min	0.616	0.622	0.625	0.625	0.627	0.626	0.625	0.626
15 min	0.618	0.623	0.626	0.626	0.627	0.626	0.626	0.625
30 min	0.619	0.624	0.626	0.628	0.627	0.628	0.627	0.626
60 min	0.616	0.623	0.625	0.626	0.626	0.626	0.626	0.625
3 hour	0.616	0.622	0.626	0.628	0.629	0.630	0.630	0.630
6 hour	0.616	0.622	0.626	0.628	0.630	0.631	0.632	0.633
12 hour	0.616	0.624	0.628	0.631	0.633	0.634	0.635	0.636
24 hour	0.615	0.623	0.627	0.630	0.632	0.633	0.634	0.635
48 hour	0.613	0.620	0.624	0.626	0.628	0.628	0.629	0.629
72 hour	0.614	0.619	0.622	0.624	0.625	0.626	0.627	0.627
96 hour	0.612	0.617	0.619	0.621	0.622	0.623	0.623	0.623

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.079	0.108	0.132	0.163	0.187	0.210	0.233	0.265
10 min	0.112	0.155	0.189	0.233	0.268	0.300	0.334	0.380
15 min	0.136	0.188	0.228	0.282	0.324	0.364	0.404	0.459
30 min	0.189	0.261	0.318	0.394	0.450	0.507	0.564	0.639
60 min	0.256	0.353	0.431	0.532	0.608	0.685	0.762	0.865
3 hour	0.409	0.544	0.657	0.811	0.933	1.060	1.192	1.376
6 hour	0.548	0.723	0.872	1.076	1.240	1.410	1.589	1.841
12 hour	0.732	0.989	1.199	1.487	1.712	1.939	2.177	2.500
24 hour	0.996	1.364	1.660	2.054	2.352	2.648	2.951	3.353
48 hour	1.229	1.652	1.988	2.423	2.749	3.065	3.385	3.798
72 hour	1.404	1.859	2.218	2.682	3.024	3.361	3.697	4.126
96 hour	1.510	1.985	2.353	2.832	3.183	3.528	3.864	4.300

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.653	0.655	0.658	0.659	0.659	0.658	0.658	0.656
10 min	0.651	0.657	0.658	0.658	0.660	0.658	0.657	0.657
15 min	0.651	0.656	0.659	0.659	0.660	0.659	0.659	0.656
30 min	0.652	0.657	0.659	0.660	0.659	0.659	0.658	0.657
60 min	0.650	0.657	0.659	0.660	0.660	0.659	0.659	0.657
3 hour	0.649	0.654	0.657	0.659	0.659	0.660	0.660	0.660
6 hour	0.648	0.653	0.656	0.658	0.659	0.660	0.661	0.662
12 hour	0.647	0.654	0.657	0.660	0.662	0.663	0.664	0.665
24 hour	0.647	0.654	0.658	0.661	0.662	0.663	0.664	0.665
48 hour	0.645	0.651	0.655	0.657	0.658	0.659	0.660	0.661
72 hour	0.645	0.650	0.652	0.654	0.655	0.656	0.657	0.658
96 hour	0.643	0.647	0.649	0.651	0.652	0.653	0.654	0.654

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.083	0.114	0.139	0.171	0.196	0.220	0.245	0.278
10 min	0.118	0.164	0.199	0.245	0.282	0.316	0.351	0.399
15 min	0.143	0.197	0.241	0.297	0.341	0.383	0.426	0.482
30 min	0.200	0.275	0.335	0.414	0.473	0.532	0.592	0.671
60 min	0.270	0.373	0.454	0.561	0.642	0.721	0.803	0.909
3 hour	0.431	0.572	0.689	0.851	0.978	1.111	1.249	1.441
6 hour	0.576	0.759	0.914	1.128	1.297	1.474	1.662	1.925
12 hour	0.769	1.037	1.255	1.556	1.790	2.028	2.276	2.614
24 hour	1.047	1.432	1.742	2.155	2.464	2.774	3.090	3.511
48 hour	1.293	1.735	2.087	2.543	2.881	3.216	3.551	3.991
72 hour	1.474	1.953	2.325	2.811	3.170	3.522	3.874	4.330
96 hour	1.587	2.081	2.467	2.969	3.337	3.698	4.057	4.514

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.544	0.550	0.555	0.558	0.558	0.556	0.556	0.554
10 min	0.543	0.553	0.555	0.557	0.559	0.557	0.555	0.555
15 min	0.545	0.553	0.557	0.558	0.559	0.557	0.557	0.554
30 min	0.545	0.553	0.557	0.559	0.559	0.558	0.557	0.555
60 min	0.542	0.553	0.556	0.558	0.558	0.557	0.557	0.554
3 hour	0.534	0.543	0.548	0.551	0.553	0.554	0.555	0.555
6 hour	0.526	0.535	0.541	0.545	0.548	0.550	0.552	0.554
12 hour	0.519	0.531	0.538	0.544	0.546	0.549	0.551	0.552
24 hour	0.512	0.525	0.532	0.538	0.540	0.542	0.544	0.545
48 hour	0.501	0.512	0.518	0.522	0.525	0.526	0.527	0.529
72 hour	0.496	0.504	0.509	0.513	0.515	0.516	0.517	0.518
96 hour	0.490	0.498	0.501	0.504	0.506	0.508	0.509	0.510

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.069	0.096	0.117	0.145	0.166	0.186	0.207	0.235
10 min	0.099	0.138	0.168	0.208	0.239	0.267	0.296	0.337
15 min	0.120	0.166	0.203	0.252	0.288	0.324	0.360	0.407
30 min	0.167	0.231	0.283	0.350	0.401	0.451	0.501	0.567
60 min	0.225	0.314	0.383	0.474	0.542	0.609	0.678	0.767
3 hour	0.355	0.475	0.575	0.712	0.821	0.932	1.050	1.212
6 hour	0.468	0.622	0.754	0.934	1.078	1.229	1.388	1.611
12 hour	0.617	0.842	1.028	1.282	1.476	1.679	1.889	2.170
24 hour	0.829	1.150	1.409	1.754	2.010	2.268	2.532	2.878
48 hour	1.005	1.364	1.650	2.020	2.298	2.567	2.836	3.194
72 hour	1.134	1.514	1.815	2.205	2.492	2.770	3.048	3.409
96 hour	1.209	1.602	1.905	2.299	2.590	2.877	3.157	3.520

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.599	0.603	0.607	0.609	0.609	0.607	0.607	0.605
10 min	0.597	0.605	0.607	0.608	0.610	0.608	0.606	0.606
15 min	0.598	0.605	0.608	0.609	0.610	0.608	0.608	0.605
30 min	0.599	0.605	0.608	0.610	0.609	0.609	0.608	0.606
60 min	0.596	0.605	0.608	0.609	0.609	0.608	0.608	0.606
3 hour	0.592	0.599	0.603	0.605	0.606	0.607	0.608	0.608
6 hour	0.587	0.594	0.599	0.602	0.604	0.605	0.607	0.608
12 hour	0.583	0.593	0.598	0.602	0.604	0.606	0.608	0.609
24 hour	0.580	0.590	0.595	0.600	0.601	0.603	0.604	0.605
48 hour	0.573	0.582	0.587	0.590	0.592	0.593	0.594	0.595
72 hour	0.571	0.577	0.581	0.584	0.585	0.586	0.587	0.588
96 hour	0.567	0.573	0.575	0.578	0.579	0.581	0.582	0.582

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.076	0.105	0.128	0.158	0.181	0.203	0.226	0.257
10 min	0.109	0.151	0.183	0.227	0.260	0.292	0.324	0.368
15 min	0.132	0.182	0.222	0.275	0.315	0.353	0.393	0.444
30 min	0.183	0.253	0.309	0.382	0.437	0.492	0.547	0.619
60 min	0.247	0.343	0.419	0.518	0.592	0.665	0.741	0.839
3 hour	0.393	0.524	0.633	0.782	0.899	1.022	1.150	1.328
6 hour	0.522	0.691	0.834	1.032	1.189	1.352	1.527	1.768
12 hour	0.693	0.940	1.142	1.419	1.633	1.854	2.084	2.394
24 hour	0.939	1.292	1.576	1.956	2.237	2.523	2.811	3.194
48 hour	1.149	1.551	1.870	2.283	2.592	2.894	3.196	3.593
72 hour	1.305	1.733	2.072	2.510	2.831	3.146	3.461	3.870
96 hour	1.399	1.843	2.186	2.636	2.963	3.290	3.610	4.017

CALAVERAS RIVER WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: RBR

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.140	0.179	0.212	0.259	0.295	0.333	0.374	0.431
10 min	0.201	0.257	0.305	0.371	0.423	0.478	0.536	0.618
15 min	0.243	0.311	0.368	0.448	0.512	0.578	0.648	0.747
30 min	0.337	0.433	0.512	0.623	0.711	0.803	0.901	1.039
60 min	0.456	0.585	0.692	0.842	0.962	1.086	1.218	1.404
3 hour	0.751	0.929	1.080	1.294	1.467	1.649	1.846	2.125
6 hour	1.029	1.260	1.454	1.728	1.948	2.178	2.424	2.773
12 hour	1.377	1.692	1.952	2.312	2.593	2.884	3.187	3.606
24 hour	1.882	2.331	2.695	3.188	3.565	3.947	4.338	4.867
48 hour	2.368	2.936	3.388	3.988	4.438	4.886	5.338	5.935
72 hour	2.718	3.372	3.888	4.565	5.068	5.564	6.061	6.712
96 hour	2.997	3.722	4.290	5.030	5.576	6.112	6.645	7.339

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.717	0.715	0.715	0.715	0.716	0.715	0.715	0.716
10 min	0.716	0.715	0.714	0.715	0.714	0.715	0.716	0.717
15 min	0.715	0.715	0.716	0.715	0.715	0.716	0.716	0.716
30 min	0.718	0.715	0.715	0.716	0.715	0.716	0.716	0.717
60 min	0.716	0.714	0.714	0.714	0.714	0.715	0.715	0.716
3 hour	0.718	0.718	0.718	0.717	0.717	0.717	0.717	0.717
6 hour	0.719	0.719	0.719	0.719	0.719	0.718	0.718	0.717
12 hour	0.720	0.719	0.719	0.719	0.719	0.718	0.718	0.718
24 hour	0.719	0.719	0.718	0.718	0.718	0.717	0.717	0.717
48 hour	0.720	0.719	0.719	0.719	0.718	0.718	0.718	0.717
72 hour	0.720	0.720	0.720	0.719	0.719	0.719	0.718	0.718
96 hour	0.720	0.720	0.720	0.720	0.720	0.719	0.719	0.718

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.100	0.128	0.152	0.185	0.211	0.238	0.267	0.309
10 min	0.144	0.184	0.218	0.265	0.302	0.342	0.384	0.443
15 min	0.174	0.222	0.263	0.320	0.366	0.414	0.464	0.535
30 min	0.242	0.310	0.366	0.446	0.508	0.575	0.645	0.745
60 min	0.326	0.418	0.494	0.601	0.687	0.776	0.871	1.005
3 hour	0.539	0.667	0.775	0.928	1.052	1.182	1.324	1.524
6 hour	0.740	0.906	1.045	1.242	1.401	1.564	1.740	1.988
12 hour	0.991	1.217	1.403	1.662	1.864	2.071	2.288	2.589
24 hour	1.353	1.676	1.935	2.289	2.560	2.830	3.110	3.490
48 hour	1.705	2.111	2.436	2.867	3.186	3.508	3.833	4.255
72 hour	1.957	2.428	2.799	3.282	3.644	4.001	4.352	4.819
96 hour	2.158	2.680	3.089	3.622	4.015	4.395	4.778	5.269

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: RBR

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.140	0.179	0.212	0.259	0.295	0.333	0.374	0.431
10 min	0.201	0.257	0.305	0.371	0.423	0.478	0.536	0.618
15 min	0.243	0.311	0.368	0.448	0.512	0.578	0.648	0.747
30 min	0.337	0.433	0.512	0.623	0.711	0.803	0.901	1.039
60 min	0.456	0.585	0.692	0.842	0.962	1.086	1.218	1.404
3 hour	0.751	0.929	1.080	1.294	1.467	1.649	1.846	2.125
6 hour	1.029	1.260	1.454	1.728	1.948	2.178	2.424	2.773
12 hour	1.377	1.692	1.952	2.312	2.593	2.884	3.187	3.606
24 hour	1.882	2.331	2.695	3.188	3.565	3.947	4.338	4.867
48 hour	2.368	2.936	3.388	3.988	4.438	4.886	5.338	5.935
72 hour	2.718	3.372	3.888	4.565	5.068	5.564	6.061	6.712
96 hour	2.997	3.722	4.290	5.030	5.576	6.112	6.645	7.339

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.732	0.733	0.731	0.732	0.733	0.731	0.732	0.731
10 min	0.734	0.734	0.731	0.732	0.731	0.732	0.732	0.732
15 min	0.732	0.733	0.733	0.733	0.732	0.732	0.732	0.732
30 min	0.733	0.732	0.732	0.732	0.732	0.731	0.731	0.731
60 min	0.734	0.732	0.733	0.732	0.732	0.733	0.732	0.732
3 hour	0.734	0.733	0.733	0.732	0.732	0.732	0.731	0.731
6 hour	0.734	0.733	0.733	0.732	0.732	0.732	0.731	0.731
12 hour	0.734	0.733	0.733	0.732	0.732	0.732	0.732	0.731
24 hour	0.733	0.733	0.733	0.732	0.732	0.732	0.732	0.732
48 hour	0.734	0.733	0.733	0.733	0.733	0.733	0.733	0.733
72 hour	0.734	0.733	0.733	0.733	0.733	0.733	0.733	0.733
96 hour	0.734	0.733	0.733	0.733	0.733	0.733	0.733	0.733

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.102	0.131	0.155	0.190	0.216	0.243	0.274	0.315
10 min	0.148	0.189	0.223	0.272	0.309	0.350	0.392	0.452
15 min	0.178	0.228	0.270	0.328	0.375	0.423	0.474	0.547
30 min	0.247	0.317	0.375	0.456	0.520	0.587	0.659	0.760
60 min	0.335	0.428	0.507	0.616	0.704	0.796	0.892	1.028
3 hour	0.551	0.681	0.792	0.947	1.074	1.207	1.349	1.553
6 hour	0.755	0.924	1.066	1.265	1.426	1.594	1.772	2.027
12 hour	1.011	1.240	1.431	1.692	1.898	2.111	2.333	2.636
24 hour	1.380	1.709	1.975	2.334	2.610	2.889	3.175	3.563
48 hour	1.738	2.152	2.483	2.923	3.253	3.581	3.913	4.350
72 hour	1.995	2.472	2.850	3.346	3.715	4.078	4.443	4.920
96 hour	2.200	2.728	3.145	3.687	4.087	4.480	4.871	5.379

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: RBR

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.140	0.179	0.212	0.259	0.295	0.333	0.374	0.431
10 min	0.201	0.257	0.305	0.371	0.423	0.478	0.536	0.618
15 min	0.243	0.311	0.368	0.448	0.512	0.578	0.648	0.747
30 min	0.337	0.433	0.512	0.623	0.711	0.803	0.901	1.039
60 min	0.456	0.585	0.692	0.842	0.962	1.086	1.218	1.404
3 hour	0.751	0.929	1.080	1.294	1.467	1.649	1.846	2.125
6 hour	1.029	1.260	1.454	1.728	1.948	2.178	2.424	2.773
12 hour	1.377	1.692	1.952	2.312	2.593	2.884	3.187	3.606
24 hour	1.882	2.331	2.695	3.188	3.565	3.947	4.338	4.867
48 hour	2.368	2.936	3.388	3.988	4.438	4.886	5.338	5.935
72 hour	2.718	3.372	3.888	4.565	5.068	5.564	6.061	6.712
96 hour	2.997	3.722	4.290	5.030	5.576	6.112	6.645	7.339

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.600	0.603	0.601	0.604	0.604	0.602	0.602	0.603
10 min	0.602	0.603	0.602	0.603	0.603	0.603	0.603	0.603
15 min	0.601	0.603	0.604	0.604	0.603	0.603	0.603	0.603
30 min	0.603	0.602	0.603	0.603	0.604	0.603	0.603	0.603
60 min	0.602	0.601	0.603	0.603	0.603	0.603	0.603	0.602
3 hour	0.596	0.597	0.598	0.598	0.598	0.598	0.598	0.599
6 hour	0.592	0.592	0.592	0.593	0.593	0.594	0.594	0.594
12 hour	0.586	0.586	0.587	0.587	0.587	0.588	0.588	0.588
24 hour	0.580	0.580	0.581	0.581	0.581	0.581	0.581	0.581
48 hour	0.574	0.573	0.573	0.573	0.573	0.573	0.573	0.572
72 hour	0.570	0.569	0.568	0.568	0.568	0.568	0.568	0.568
96 hour	0.568	0.567	0.566	0.566	0.566	0.566	0.566	0.566

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.084	0.108	0.127	0.156	0.178	0.200	0.225	0.260
10 min	0.121	0.155	0.184	0.224	0.255	0.288	0.323	0.373
15 min	0.146	0.188	0.222	0.271	0.309	0.349	0.391	0.450
30 min	0.203	0.261	0.309	0.376	0.429	0.484	0.543	0.627
60 min	0.275	0.352	0.417	0.508	0.580	0.655	0.734	0.845
3 hour	0.448	0.555	0.646	0.774	0.877	0.986	1.104	1.273
6 hour	0.609	0.746	0.861	1.025	1.155	1.294	1.440	1.647
12 hour	0.807	0.992	1.146	1.357	1.522	1.696	1.874	2.120
24 hour	1.092	1.352	1.566	1.852	2.071	2.293	2.520	2.828
48 hour	1.359	1.682	1.941	2.285	2.543	2.800	3.059	3.395
72 hour	1.549	1.919	2.208	2.593	2.879	3.160	3.443	3.812
96 hour	1.702	2.110	2.428	2.847	3.156	3.459	3.761	4.154

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: RBR

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.140	0.179	0.212	0.259	0.295	0.333	0.374	0.431
10 min	0.201	0.257	0.305	0.371	0.423	0.478	0.536	0.618
15 min	0.243	0.311	0.368	0.448	0.512	0.578	0.648	0.747
30 min	0.337	0.433	0.512	0.623	0.711	0.803	0.901	1.039
60 min	0.456	0.585	0.692	0.842	0.962	1.086	1.218	1.404
3 hour	0.751	0.929	1.080	1.294	1.467	1.649	1.846	2.125
6 hour	1.029	1.260	1.454	1.728	1.948	2.178	2.424	2.773
12 hour	1.377	1.692	1.952	2.312	2.593	2.884	3.187	3.606
24 hour	1.882	2.331	2.695	3.188	3.565	3.947	4.338	4.867
48 hour	2.368	2.936	3.388	3.988	4.438	4.886	5.338	5.935
72 hour	2.718	3.372	3.888	4.565	5.068	5.564	6.061	6.712
96 hour	2.997	3.722	4.290	5.030	5.576	6.112	6.645	7.339

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.666	0.668	0.666	0.668	0.669	0.667	0.667	0.667
10 min	0.668	0.669	0.667	0.668	0.667	0.668	0.668	0.668
15 min	0.667	0.668	0.669	0.669	0.668	0.668	0.668	0.668
30 min	0.668	0.667	0.668	0.668	0.668	0.667	0.667	0.667
60 min	0.668	0.667	0.668	0.668	0.668	0.668	0.668	0.667
3 hour	0.665	0.665	0.666	0.665	0.665	0.665	0.665	0.665
6 hour	0.663	0.663	0.663	0.663	0.663	0.663	0.663	0.663
12 hour	0.660	0.660	0.660	0.660	0.660	0.660	0.660	0.660
24 hour	0.657	0.657	0.657	0.657	0.657	0.657	0.657	0.657
48 hour	0.654	0.653	0.653	0.653	0.653	0.653	0.653	0.653
72 hour	0.652	0.651	0.651	0.651	0.651	0.651	0.651	0.651
96 hour	0.651	0.650	0.650	0.650	0.650	0.650	0.650	0.650

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.093	0.120	0.141	0.173	0.197	0.222	0.249	0.287
10 min	0.134	0.172	0.203	0.248	0.282	0.319	0.358	0.413
15 min	0.162	0.208	0.246	0.300	0.342	0.386	0.433	0.499
30 min	0.225	0.289	0.342	0.416	0.475	0.536	0.601	0.693
60 min	0.305	0.390	0.462	0.562	0.643	0.725	0.814	0.936
3 hour	0.499	0.618	0.719	0.861	0.976	1.097	1.228	1.413
6 hour	0.682	0.835	0.964	1.146	1.292	1.444	1.607	1.838
12 hour	0.909	1.117	1.288	1.526	1.711	1.903	2.103	2.380
24 hour	1.236	1.531	1.771	2.095	2.342	2.593	2.850	3.198
48 hour	1.549	1.917	2.212	2.604	2.898	3.191	3.486	3.876
72 hour	1.772	2.195	2.531	2.972	3.299	3.622	3.946	4.370
96 hour	1.951	2.419	2.789	3.270	3.624	3.973	4.319	4.770

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.634	0.632	0.633	0.632	0.632	0.633	0.632	0.634
10 min	0.633	0.630	0.633	0.632	0.632	0.632	0.633	0.633
15 min	0.635	0.632	0.631	0.631	0.631	0.632	0.632	0.633
30 min	0.634	0.632	0.632	0.632	0.632	0.632	0.633	0.634
60 min	0.633	0.631	0.630	0.631	0.631	0.631	0.632	0.633
3 hour	0.634	0.633	0.632	0.632	0.633	0.633	0.633	0.633
6 hour	0.635	0.634	0.634	0.633	0.633	0.633	0.633	0.633
12 hour	0.635	0.634	0.634	0.633	0.632	0.632	0.632	0.632
24 hour	0.636	0.634	0.633	0.632	0.632	0.631	0.631	0.630
48 hour	0.636	0.634	0.634	0.633	0.632	0.632	0.632	0.631
72 hour	0.636	0.635	0.635	0.634	0.634	0.633	0.633	0.632
96 hour	0.637	0.636	0.636	0.635	0.635	0.635	0.634	0.634

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.094	0.121	0.144	0.175	0.201	0.228	0.256	0.297
10 min	0.134	0.173	0.206	0.252	0.288	0.326	0.367	0.425
15 min	0.163	0.209	0.248	0.304	0.348	0.394	0.444	0.514
30 min	0.228	0.293	0.348	0.425	0.486	0.551	0.621	0.719
60 min	0.303	0.390	0.462	0.566	0.648	0.734	0.827	0.958
3 hour	0.496	0.619	0.723	0.874	0.998	1.129	1.270	1.473
6 hour	0.681	0.842	0.979	1.172	1.329	1.494	1.671	1.922
12 hour	0.917	1.136	1.319	1.570	1.766	1.971	2.185	2.482
24 hour	1.259	1.567	1.817	2.153	2.411	2.668	2.935	3.290
48 hour	1.572	1.957	2.266	2.669	2.967	3.268	3.570	3.961
72 hour	1.811	2.259	2.612	3.067	3.405	3.732	4.061	4.484
96 hour	2.009	2.506	2.896	3.393	3.759	4.117	4.463	4.917

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
10 min	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
15 min	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
30 min	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
60 min	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
3 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
6 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
12 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
24 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
48 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
72 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
96 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.126	0.162	0.193	0.235	0.270	0.306	0.344	0.398
10 min	0.180	0.233	0.276	0.338	0.387	0.439	0.493	0.570
15 min	0.218	0.281	0.334	0.409	0.468	0.530	0.597	0.690
30 min	0.305	0.394	0.468	0.571	0.654	0.741	0.834	0.964
60 min	0.407	0.525	0.624	0.762	0.873	0.989	1.113	1.287
3 hour	0.666	0.831	0.972	1.176	1.340	1.516	1.706	1.978
6 hour	0.912	1.129	1.312	1.573	1.784	2.006	2.244	2.581
12 hour	1.227	1.523	1.769	2.109	2.376	2.651	2.939	3.338
24 hour	1.683	2.101	2.440	2.895	3.243	3.595	3.954	4.439
48 hour	2.101	2.624	3.038	3.584	3.991	4.395	4.801	5.335
72 hour	2.421	3.024	3.497	4.111	4.565	5.011	5.454	6.031
96 hour	2.681	3.350	3.870	4.542	5.032	5.511	5.983	6.593

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.588	0.592	0.595	0.597	0.595	0.596	0.595	0.596
10 min	0.589	0.589	0.596	0.595	0.597	0.596	0.595	0.595
15 min	0.593	0.593	0.592	0.595	0.595	0.594	0.596	0.596
30 min	0.591	0.593	0.595	0.596	0.597	0.597	0.597	0.596
60 min	0.588	0.590	0.591	0.593	0.594	0.594	0.594	0.594
3 hour	0.578	0.582	0.583	0.585	0.586	0.587	0.589	0.590
6 hour	0.569	0.573	0.575	0.577	0.579	0.580	0.581	0.583
12 hour	0.561	0.564	0.566	0.568	0.569	0.570	0.571	0.572
24 hour	0.552	0.554	0.555	0.557	0.557	0.557	0.557	0.557
48 hour	0.539	0.540	0.540	0.541	0.541	0.541	0.541	0.540
72 hour	0.534	0.534	0.533	0.533	0.533	0.533	0.532	0.532
96 hour	0.532	0.531	0.530	0.530	0.530	0.529	0.529	0.529

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.087	0.113	0.135	0.165	0.189	0.215	0.241	0.279
10 min	0.125	0.161	0.194	0.237	0.272	0.308	0.345	0.399
15 min	0.152	0.196	0.233	0.286	0.328	0.371	0.418	0.484
30 min	0.212	0.275	0.327	0.401	0.459	0.521	0.586	0.676
60 min	0.282	0.365	0.434	0.532	0.610	0.691	0.778	0.899
3 hour	0.453	0.569	0.667	0.809	0.924	1.047	1.182	1.373
6 hour	0.611	0.761	0.888	1.068	1.215	1.369	1.534	1.771
12 hour	0.810	1.011	1.178	1.409	1.590	1.778	1.975	2.246
24 hour	1.093	1.369	1.593	1.897	2.125	2.356	2.591	2.909
48 hour	1.332	1.667	1.930	2.281	2.540	2.798	3.056	3.390
72 hour	1.521	1.900	2.193	2.578	2.863	3.142	3.413	3.775
96 hour	1.678	2.093	2.413	2.832	3.138	3.430	3.724	4.103

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.719	0.721	0.723	0.724	0.723	0.723	0.723	0.723
10 min	0.720	0.720	0.723	0.723	0.724	0.723	0.723	0.723
15 min	0.722	0.722	0.721	0.723	0.723	0.722	0.723	0.723
30 min	0.721	0.722	0.723	0.723	0.724	0.724	0.724	0.723
60 min	0.719	0.720	0.721	0.722	0.722	0.722	0.722	0.722
3 hour	0.714	0.716	0.717	0.718	0.718	0.719	0.720	0.720
6 hour	0.710	0.712	0.713	0.714	0.715	0.715	0.716	0.717
12 hour	0.706	0.707	0.708	0.709	0.710	0.710	0.711	0.711
24 hour	0.701	0.702	0.703	0.704	0.704	0.704	0.704	0.704
48 hour	0.695	0.695	0.695	0.696	0.696	0.696	0.696	0.695
72 hour	0.692	0.692	0.692	0.692	0.692	0.692	0.691	0.691
96 hour	0.691	0.691	0.690	0.690	0.690	0.690	0.690	0.690

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.106	0.138	0.164	0.201	0.230	0.260	0.293	0.338
10 min	0.153	0.197	0.235	0.288	0.329	0.373	0.419	0.485
15 min	0.186	0.239	0.283	0.348	0.398	0.451	0.508	0.587
30 min	0.259	0.334	0.398	0.486	0.557	0.631	0.710	0.820
60 min	0.344	0.445	0.529	0.648	0.741	0.840	0.945	1.093
3 hour	0.559	0.700	0.820	0.993	1.132	1.283	1.445	1.675
6 hour	0.762	0.946	1.101	1.322	1.501	1.687	1.890	2.178
12 hour	1.019	1.267	1.473	1.759	1.984	2.214	2.459	2.792
24 hour	1.388	1.735	2.018	2.398	2.686	2.977	3.275	3.676
48 hour	1.718	2.145	2.484	2.934	3.268	3.599	3.931	4.363
72 hour	1.971	2.462	2.847	3.347	3.717	4.079	4.433	4.903
96 hour	2.179	2.723	3.142	3.687	4.085	4.473	4.857	5.352

CALAVERAS RIVER WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.619	0.618	0.617	0.617	0.618	0.621	0.621	0.622
10 min	0.618	0.616	0.617	0.618	0.618	0.620	0.621	0.623
15 min	0.618	0.618	0.617	0.618	0.619	0.620	0.621	0.623
30 min	0.619	0.618	0.617	0.619	0.619	0.620	0.621	0.623
60 min	0.617	0.616	0.616	0.617	0.618	0.619	0.620	0.622
3 hour	0.620	0.619	0.619	0.619	0.619	0.620	0.621	0.622
6 hour	0.622	0.622	0.621	0.621	0.621	0.622	0.622	0.622
12 hour	0.625	0.623	0.623	0.622	0.622	0.622	0.622	0.622
24 hour	0.627	0.625	0.624	0.623	0.622	0.622	0.622	0.621
48 hour	0.629	0.627	0.627	0.626	0.625	0.625	0.624	0.624
72 hour	0.630	0.629	0.629	0.628	0.627	0.627	0.626	0.625
96 hour	0.632	0.631	0.630	0.629	0.629	0.628	0.627	0.627

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.104	0.135	0.161	0.199	0.230	0.264	0.299	0.351
10 min	0.149	0.193	0.231	0.286	0.330	0.378	0.429	0.505
15 min	0.180	0.234	0.280	0.345	0.399	0.457	0.519	0.610
30 min	0.251	0.326	0.389	0.482	0.556	0.636	0.723	0.850
60 min	0.334	0.433	0.518	0.640	0.740	0.847	0.962	1.130
3 hour	0.558	0.697	0.818	0.995	1.141	1.302	1.479	1.737
6 hour	0.788	0.975	1.133	1.362	1.549	1.753	1.970	2.285
12 hour	1.094	1.349	1.564	1.862	2.099	2.346	2.608	2.976
24 hour	1.547	1.914	2.211	2.614	2.920	3.234	3.558	3.992
48 hour	2.006	2.479	2.858	3.351	3.715	4.081	4.443	4.930
72 hour	2.356	2.916	3.355	3.917	4.328	4.735	5.132	5.651
96 hour	2.647	3.276	3.757	4.373	4.825	5.254	5.676	6.230

CALAVERAS RIVER WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.618	0.619	0.617	0.617	0.617	0.622	0.622	0.623
10 min	0.619	0.616	0.616	0.618	0.618	0.620	0.621	0.623
15 min	0.618	0.618	0.616	0.618	0.619	0.621	0.622	0.624
30 min	0.619	0.618	0.617	0.618	0.619	0.621	0.621	0.623
60 min	0.617	0.615	0.616	0.617	0.618	0.619	0.620	0.623
3 hour	0.620	0.619	0.618	0.618	0.618	0.619	0.620	0.622
6 hour	0.624	0.622	0.621	0.620	0.621	0.621	0.621	0.622
12 hour	0.627	0.624	0.623	0.621	0.621	0.620	0.620	0.620
24 hour	0.630	0.626	0.624	0.623	0.622	0.621	0.621	0.621
48 hour	0.633	0.630	0.629	0.627	0.626	0.626	0.625	0.625
72 hour	0.635	0.633	0.631	0.630	0.629	0.629	0.628	0.627
96 hour	0.637	0.635	0.634	0.633	0.632	0.631	0.630	0.629

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.104	0.135	0.161	0.199	0.230	0.264	0.300	0.352
10 min	0.149	0.193	0.231	0.286	0.330	0.378	0.429	0.505
15 min	0.180	0.234	0.279	0.345	0.399	0.458	0.520	0.611
30 min	0.251	0.326	0.389	0.481	0.556	0.637	0.723	0.850
60 min	0.334	0.432	0.518	0.640	0.740	0.847	0.962	1.132
3 hour	0.558	0.697	0.817	0.994	1.140	1.300	1.476	1.737
6 hour	0.791	0.975	1.133	1.360	1.549	1.750	1.967	2.285
12 hour	1.098	1.351	1.564	1.859	2.096	2.339	2.600	2.966
24 hour	1.554	1.917	2.211	2.614	2.920	3.229	3.553	3.992
48 hour	2.019	2.491	2.867	3.356	3.721	4.088	4.450	4.938
72 hour	2.375	2.935	3.366	3.930	4.341	4.750	5.148	5.669
96 hour	2.668	3.296	3.781	4.401	4.848	5.279	5.703	6.250

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.639	0.645	0.645	0.648	0.648	0.652	0.651	0.652
10 min	0.640	0.641	0.645	0.647	0.649	0.650	0.651	0.653
15 min	0.641	0.645	0.645	0.649	0.650	0.651	0.652	0.653
30 min	0.642	0.645	0.646	0.649	0.651	0.652	0.652	0.653
60 min	0.637	0.640	0.644	0.646	0.647	0.649	0.650	0.651
3 hour	0.631	0.635	0.636	0.639	0.640	0.643	0.644	0.647
6 hour	0.627	0.629	0.631	0.633	0.636	0.637	0.639	0.641
12 hour	0.622	0.624	0.626	0.627	0.629	0.630	0.631	0.632
24 hour	0.618	0.619	0.619	0.620	0.620	0.620	0.620	0.621
48 hour	0.612	0.611	0.611	0.610	0.610	0.610	0.609	0.608
72 hour	0.610	0.608	0.607	0.606	0.605	0.605	0.604	0.603
96 hour	0.610	0.608	0.606	0.605	0.604	0.603	0.602	0.601

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.107	0.141	0.168	0.209	0.241	0.277	0.314	0.368
10 min	0.154	0.201	0.242	0.299	0.347	0.396	0.450	0.529
15 min	0.187	0.244	0.292	0.363	0.419	0.480	0.545	0.639
30 min	0.261	0.340	0.408	0.505	0.585	0.669	0.759	0.891
60 min	0.345	0.450	0.542	0.671	0.775	0.888	1.009	1.183
3 hour	0.568	0.715	0.841	1.028	1.180	1.350	1.533	1.807
6 hour	0.794	0.986	1.151	1.389	1.587	1.795	2.024	2.354
12 hour	1.089	1.351	1.572	1.877	2.123	2.376	2.646	3.023
24 hour	1.525	1.895	2.194	2.602	2.910	3.224	3.547	3.992
48 hour	1.952	2.416	2.785	3.265	3.626	3.983	4.336	4.804
72 hour	2.281	2.819	3.238	3.780	4.176	4.569	4.952	5.452
96 hour	2.555	3.156	3.614	4.207	4.633	5.045	5.449	5.972

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.629	0.632	0.631	0.633	0.633	0.637	0.637	0.638
10 min	0.630	0.629	0.631	0.633	0.634	0.635	0.636	0.638
15 min	0.630	0.632	0.631	0.634	0.635	0.636	0.637	0.639
30 min	0.631	0.632	0.632	0.634	0.635	0.637	0.637	0.638
60 min	0.627	0.628	0.630	0.632	0.633	0.634	0.635	0.637
3 hour	0.626	0.627	0.627	0.629	0.629	0.631	0.632	0.635
6 hour	0.626	0.626	0.626	0.627	0.629	0.629	0.630	0.632
12 hour	0.625	0.624	0.625	0.624	0.625	0.625	0.626	0.626
24 hour	0.624	0.623	0.622	0.622	0.621	0.621	0.621	0.621
48 hour	0.623	0.621	0.620	0.619	0.618	0.618	0.617	0.617
72 hour	0.623	0.621	0.619	0.618	0.617	0.617	0.616	0.615
96 hour	0.624	0.622	0.620	0.619	0.618	0.617	0.616	0.615

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.106	0.138	0.165	0.204	0.235	0.271	0.307	0.360
10 min	0.152	0.197	0.237	0.292	0.339	0.387	0.439	0.517
15 min	0.184	0.240	0.286	0.354	0.410	0.469	0.533	0.626
30 min	0.256	0.333	0.399	0.493	0.570	0.654	0.741	0.870
60 min	0.340	0.441	0.530	0.656	0.758	0.867	0.986	1.157
3 hour	0.563	0.706	0.829	1.011	1.160	1.325	1.505	1.774
6 hour	0.793	0.981	1.142	1.376	1.569	1.773	1.996	2.321
12 hour	1.094	1.351	1.569	1.868	2.109	2.358	2.625	2.995
24 hour	1.539	1.908	2.204	2.610	2.915	3.229	3.553	3.992
48 hour	1.987	2.455	2.826	3.314	3.673	4.036	4.393	4.875
72 hour	2.330	2.879	3.302	3.855	4.259	4.660	5.050	5.561
96 hour	2.613	3.229	3.698	4.304	4.741	5.162	5.576	6.111

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: UPPER CAL

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.176	0.221	0.260	0.314	0.357	0.403	0.451	0.521
10 min	0.252	0.317	0.372	0.450	0.512	0.577	0.647	0.747
15 min	0.305	0.384	0.450	0.544	0.619	0.698	0.782	0.904
30 min	0.420	0.529	0.620	0.749	0.853	0.962	1.078	1.246
60 min	0.576	0.725	0.851	1.028	1.169	1.319	1.479	1.708
3 hour	1.015	1.241	1.432	1.700	1.916	2.143	2.385	2.731
6 hour	1.487	1.801	2.062	2.428	2.718	3.020	3.340	3.788
12 hour	2.129	2.587	2.965	3.485	3.892	4.310	4.748	5.352
24 hour	3.103	3.816	4.396	5.183	5.787	6.401	7.035	7.896
48 hour	4.189	5.210	6.023	7.099	7.908	8.714	9.530	10.618
72 hour	4.980	6.240	7.228	8.519	9.476	10.420	11.367	12.613
96 hour	5.578	7.014	8.130	9.571	10.629	11.663	12.690	14.029

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.567	0.562	0.561	0.559	0.560	0.559	0.560	0.560
10 min	0.566	0.563	0.561	0.560	0.559	0.560	0.560	0.561
15 min	0.565	0.562	0.561	0.559	0.560	0.560	0.559	0.560
30 min	0.565	0.562	0.560	0.560	0.559	0.559	0.559	0.560
60 min	0.567	0.564	0.561	0.560	0.560	0.560	0.560	0.561
3 hour	0.573	0.570	0.569	0.568	0.567	0.566	0.565	0.564
6 hour	0.578	0.576	0.574	0.573	0.572	0.571	0.570	0.569
12 hour	0.582	0.580	0.578	0.577	0.576	0.575	0.575	0.574
24 hour	0.585	0.583	0.582	0.581	0.581	0.580	0.580	0.580
48 hour	0.589	0.589	0.588	0.588	0.587	0.587	0.587	0.587
72 hour	0.591	0.591	0.591	0.591	0.591	0.590	0.590	0.590
96 hour	0.592	0.592	0.592	0.592	0.592	0.592	0.592	0.592

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.100	0.124	0.146	0.176	0.200	0.225	0.253	0.292
10 min	0.143	0.178	0.209	0.252	0.286	0.323	0.362	0.419
15 min	0.172	0.216	0.252	0.304	0.347	0.391	0.437	0.506
30 min	0.237	0.297	0.347	0.419	0.477	0.538	0.603	0.698
60 min	0.327	0.409	0.477	0.576	0.655	0.739	0.828	0.958
3 hour	0.582	0.707	0.815	0.966	1.086	1.213	1.348	1.540
6 hour	0.859	1.037	1.184	1.391	1.555	1.724	1.904	2.155
12 hour	1.239	1.500	1.714	2.011	2.242	2.478	2.730	3.072
24 hour	1.815	2.225	2.558	3.011	3.362	3.713	4.080	4.580
48 hour	2.467	3.069	3.542	4.174	4.642	5.115	5.594	6.233
72 hour	2.943	3.688	4.272	5.035	5.600	6.148	6.707	7.442
96 hour	3.302	4.152	4.813	5.666	6.292	6.904	7.512	8.305

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: UPPER CAL

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.176	0.221	0.260	0.314	0.357	0.403	0.451	0.521
10 min	0.252	0.317	0.372	0.450	0.512	0.577	0.647	0.747
15 min	0.305	0.384	0.450	0.544	0.619	0.698	0.782	0.904
30 min	0.420	0.529	0.620	0.749	0.853	0.962	1.078	1.246
60 min	0.576	0.725	0.851	1.028	1.169	1.319	1.479	1.708
3 hour	1.015	1.241	1.432	1.700	1.916	2.143	2.385	2.731
6 hour	1.487	1.801	2.062	2.428	2.718	3.020	3.340	3.788
12 hour	2.129	2.587	2.965	3.485	3.892	4.310	4.748	5.352
24 hour	3.103	3.816	4.396	5.183	5.787	6.401	7.035	7.896
48 hour	4.189	5.210	6.023	7.099	7.908	8.714	9.530	10.618
72 hour	4.980	6.240	7.228	8.519	9.476	10.420	11.367	12.613
96 hour	5.578	7.014	8.130	9.571	10.629	11.663	12.690	14.029

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.610	0.607	0.606	0.603	0.604	0.604	0.604	0.604
10 min	0.609	0.607	0.605	0.605	0.604	0.604	0.604	0.604
15 min	0.608	0.606	0.605	0.604	0.604	0.604	0.603	0.604
30 min	0.608	0.606	0.605	0.604	0.603	0.603	0.603	0.604
60 min	0.610	0.608	0.606	0.605	0.605	0.604	0.604	0.605
3 hour	0.615	0.612	0.611	0.610	0.609	0.608	0.608	0.607
6 hour	0.618	0.616	0.615	0.614	0.613	0.612	0.611	0.610
12 hour	0.621	0.619	0.618	0.617	0.616	0.615	0.615	0.614
24 hour	0.624	0.622	0.621	0.620	0.620	0.620	0.619	0.619
48 hour	0.627	0.626	0.626	0.626	0.625	0.625	0.625	0.625
72 hour	0.629	0.628	0.628	0.628	0.628	0.628	0.628	0.628
96 hour	0.630	0.629	0.629	0.629	0.629	0.629	0.629	0.629

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.107	0.134	0.158	0.189	0.216	0.243	0.272	0.315
10 min	0.153	0.192	0.225	0.272	0.309	0.349	0.391	0.451
15 min	0.185	0.233	0.272	0.329	0.374	0.422	0.472	0.546
30 min	0.255	0.321	0.375	0.452	0.514	0.580	0.650	0.753
60 min	0.351	0.441	0.516	0.622	0.707	0.797	0.893	1.033
3 hour	0.624	0.759	0.875	1.037	1.167	1.303	1.450	1.658
6 hour	0.919	1.109	1.268	1.491	1.666	1.848	2.041	2.311
12 hour	1.322	1.601	1.832	2.150	2.397	2.651	2.920	3.286
24 hour	1.936	2.374	2.730	3.213	3.588	3.969	4.355	4.888
48 hour	2.627	3.261	3.770	4.444	4.943	5.446	5.956	6.636
72 hour	3.132	3.919	4.539	5.350	5.951	6.544	7.138	7.921
96 hour	3.514	4.412	5.114	6.020	6.686	7.336	7.982	8.824

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: UPPER CAL

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.176	0.221	0.260	0.314	0.357	0.403	0.451	0.521
10 min	0.252	0.317	0.372	0.450	0.512	0.577	0.647	0.747
15 min	0.305	0.384	0.450	0.544	0.619	0.698	0.782	0.904
30 min	0.420	0.529	0.620	0.749	0.853	0.962	1.078	1.246
60 min	0.576	0.725	0.851	1.028	1.169	1.319	1.479	1.708
3 hour	1.015	1.241	1.432	1.700	1.916	2.143	2.385	2.731
6 hour	1.487	1.801	2.062	2.428	2.718	3.020	3.340	3.788
12 hour	2.129	2.587	2.965	3.485	3.892	4.310	4.748	5.352
24 hour	3.103	3.816	4.396	5.183	5.787	6.401	7.035	7.896
48 hour	4.189	5.210	6.023	7.099	7.908	8.714	9.530	10.618
72 hour	4.980	6.240	7.228	8.519	9.476	10.420	11.367	12.613
96 hour	5.578	7.014	8.130	9.571	10.629	11.663	12.690	14.029

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
10 min	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
15 min	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
30 min	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
60 min	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
3 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
6 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
12 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
24 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
48 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
72 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
96 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.169	0.199	0.240	0.273	0.308	0.345	0.398
10 min	0.193	0.242	0.284	0.344	0.391	0.441	0.494	0.571
15 min	0.233	0.293	0.344	0.416	0.473	0.533	0.597	0.691
30 min	0.321	0.404	0.474	0.572	0.652	0.735	0.824	0.952
60 min	0.440	0.554	0.650	0.785	0.893	1.008	1.130	1.305
3 hour	0.775	0.948	1.094	1.299	1.464	1.637	1.822	2.086
6 hour	1.136	1.376	1.575	1.855	2.077	2.307	2.552	2.894
12 hour	1.627	1.976	2.265	2.663	2.973	3.293	3.627	4.089
24 hour	2.371	2.915	3.359	3.960	4.421	4.890	5.375	6.033
48 hour	3.200	3.980	4.602	5.424	6.042	6.657	7.281	8.112
72 hour	3.805	4.767	5.522	6.509	7.240	7.961	8.684	9.636
96 hour	4.262	5.359	6.211	7.312	8.121	8.911	9.695	10.718

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: UPPER CAL

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.176	0.221	0.260	0.314	0.357	0.403	0.451	0.521
10 min	0.252	0.317	0.372	0.450	0.512	0.577	0.647	0.747
15 min	0.305	0.384	0.450	0.544	0.619	0.698	0.782	0.904
30 min	0.420	0.529	0.620	0.749	0.853	0.962	1.078	1.246
60 min	0.576	0.725	0.851	1.028	1.169	1.319	1.479	1.708
3 hour	1.015	1.241	1.432	1.700	1.916	2.143	2.385	2.731
6 hour	1.487	1.801	2.062	2.428	2.718	3.020	3.340	3.788
12 hour	2.129	2.587	2.965	3.485	3.892	4.310	4.748	5.352
24 hour	3.103	3.816	4.396	5.183	5.787	6.401	7.035	7.896
48 hour	4.189	5.210	6.023	7.099	7.908	8.714	9.530	10.618
72 hour	4.980	6.240	7.228	8.519	9.476	10.420	11.367	12.613
96 hour	5.578	7.014	8.130	9.571	10.629	11.663	12.690	14.029

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.687	0.686	0.685	0.684	0.684	0.684	0.684	0.684
10 min	0.687	0.686	0.685	0.685	0.684	0.684	0.684	0.684
15 min	0.686	0.685	0.685	0.684	0.684	0.684	0.684	0.684
30 min	0.686	0.685	0.685	0.684	0.684	0.684	0.684	0.684
60 min	0.687	0.686	0.685	0.685	0.685	0.684	0.684	0.685
3 hour	0.690	0.688	0.688	0.687	0.687	0.686	0.686	0.686
6 hour	0.691	0.690	0.690	0.689	0.689	0.688	0.688	0.687
12 hour	0.693	0.692	0.691	0.691	0.690	0.690	0.690	0.689
24 hour	0.694	0.693	0.693	0.692	0.692	0.692	0.692	0.692
48 hour	0.696	0.695	0.695	0.695	0.695	0.695	0.695	0.695
72 hour	0.697	0.696	0.696	0.696	0.696	0.696	0.696	0.696
96 hour	0.697	0.697	0.697	0.697	0.697	0.697	0.697	0.697

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.121	0.152	0.178	0.215	0.244	0.276	0.308	0.356
10 min	0.173	0.217	0.255	0.308	0.350	0.395	0.443	0.511
15 min	0.209	0.263	0.308	0.372	0.423	0.477	0.535	0.618
30 min	0.288	0.362	0.425	0.512	0.583	0.658	0.737	0.852
60 min	0.396	0.497	0.583	0.704	0.801	0.902	1.012	1.170
3 hour	0.700	0.854	0.985	1.168	1.316	1.470	1.636	1.873
6 hour	1.028	1.243	1.423	1.673	1.873	2.078	2.298	2.602
12 hour	1.475	1.790	2.049	2.408	2.685	2.974	3.276	3.688
24 hour	2.153	2.644	3.046	3.587	4.005	4.429	4.868	5.464
48 hour	2.916	3.621	4.186	4.934	5.496	6.056	6.623	7.380
72 hour	3.471	4.343	5.031	5.929	6.595	7.252	7.911	8.779
96 hour	3.888	4.889	5.667	6.671	7.408	8.129	8.845	9.778

Attachment 5- E. ITR Comment Forms for Calaveras River HEC-HMS Modeling

Lower San Joaquin River Feasibility Study

HYDROLOGIC MODELING REVIEW – CALAVERAS RIVER WATERSHED

Task Order 6 – LSJRFS Work-In-Kind Hydrology

Reviewer:	Domenichelli & Associates
Review Date:	11-01-10
PBI Response Date:	11-03-10
DA Backcheck:	11-15-10

Memorandum Comments:

1. *Section 5.4.1 Subbasins – This would be a good place to talk about element HOLM and HOLM-PS. Element C80 from the HEC-1 model has been removed and replaced with HOLM and HOLM-PS.*

   PBI Response: An explanation for the re-naming of subbasin C80 has been added to *Section 5.4.1: Subbasins.*

   DA Backcheck: Accepted.

2. *In Attachment A the HEC-1 vs HEC-HMS comparison there is a model element RSRES and R60 but these elements are not found in Table 2. Summary of PBI Model routing elements or in the existing model. Add a paragraph explaining the removal of these elements or if the removal was unintentional add the elements to the model.*

   PBI Response: Three reach elements from the HEC-1 model (RSRES, R60, and RS4060) were initially combined into 1 reach (R4070) for the PBI Model. They have now been separated back into their original components for clarity purposes. See Table 2.

   DA Backcheck 1: R4070 is still in Table 2 from C40 to C70. RDV40, RSRES, and R60 are also in the model covering C40 to C70.

   DA Backcheck 2: The new length for RDV40, RSRES, and R60 is now 4000 feet more than the original length for R4070, which is correct?

   PBI Response to Backcheck 1: R4070 is a routing reach that represents the main channel of the Calaveras River from C40 to C70. RDV40, RSRES, and R60 are routing reaches that were added in to the model to represent overland flow from C40 to C70 for that portion of subbasin runoff that is prevented from entering the main channel of the Calaveras River due to levee barriers.

PBI Response to Backcheck 2: RDV40, RSRES, and R60 represent overland routing which takes a longer pathway than the main channel path of R4070.

Model Comments:

3. *In the model the impervious percentage of basin CG10 is listed as 0% impervious. From the figures in the memorandum it appears that the La Contenta community is within the basin. Consider using an impervious percentage of 2% or 5% for that basin.*

   PBI Response: Agreed. Subbasin CG10 is now assigned an impervious percentage of 5%.

   DA Backcheck: Accepted per attachment C.

Attachment 5- F. SPK Comment Forms for Calaveras River HEC-HMS Modeling

Corps of Engineers, Hydrology Section

Review of Calaveras HEC-1 to HEC-HMS model conversion and preliminary report.

22 November 2010 (Revised and transmitted 30 November 2010, sfh)

by Steven F. Holmstrom, P.E.

The Technical Memorandum for the Lower San Joaquin River Feasibility Study Calaveras HEC-HMS modeling DRAFT hydrology report has been reviewed and the following comments are provided.

18. In section 5.3 Design Storms, the fourth paragraph states that three storm centerings will be analyzed. A third storm centering will be required to compute the flow above the New Hogan Dam for the 8 design storm frequencies. Thank you for adding a third storm centering.

    PBI Response: No response necessary.

19. In Section 5.4.2 Reservoirs and Pumps, it must be made clear that the pumps discharge into the receiving channel above the highest stage to be expected so that there is independence between the exterior and interior areas. If that is not the case then a coincidence analysis must be performed to determine the modified interior pond stage-frequency curve considering the exterior-interior stage conditions. This is explained in EM1110-2-1413, Hydrologic Analysis of Interior Areas.

    PBI Response: Section 5.4.2 has been updated.

20. In figure 2, Calaveras River subbasins, the precipitation gage at Perry Ranch (PRY) should be shown, as it is mentioned in paragraph 5.5 model calibration. In addition, the stream gage at Duck Creek near Farmington that was mentioned in paragraph 5.2.1 SJAFCA HEC-1 model should be shown on the figure. A comparison of the 1/100 AEP results from the previous study and the current study will be interesting to see when the NOAA Atlas 14 document is published.

    PBI Response: Figure 2 now includes the mentioned gages.

    A comparison of the 1/100 AEP results from the 1998 HEC-1 study and the current study can be included once NOAA Atlas 14 precipitation is coded into the HEC-HMS model and production runs are completed.

21. The HMS model transmitted with this report failed at reservoir element “STPON”. Results from the report could not be compared with either the results from the 1998 study or the calibration done for the current study. The report must clarify the purpose and the state of the input parameters contained in the model supplied. It has been determined that the available storage for element “STPON” is marginal relative to the event simulated and events more rare. Additional storage should be coded into the model if available. Alternatively, an emergency flow path should be defined should the detention pond overflow.

    PBI Response: After discussions with the Corps, it was found that the transmitted model ran to completion when using HEC-HMS v3.4, but not when using the recently released HEC-HMS v3.5. The cause of this was determined to be an elevation-storage function for “STPON” that did not define the relationship for the upper limit of simulated storage conditions. To remedy this, the STPON function was inspected and extrapolated to handle a larger inflow event. All storage functions will be inspected once NOAA14 precipitation events are coded into the model to ensure they can handle the 500-year event (and beyond).

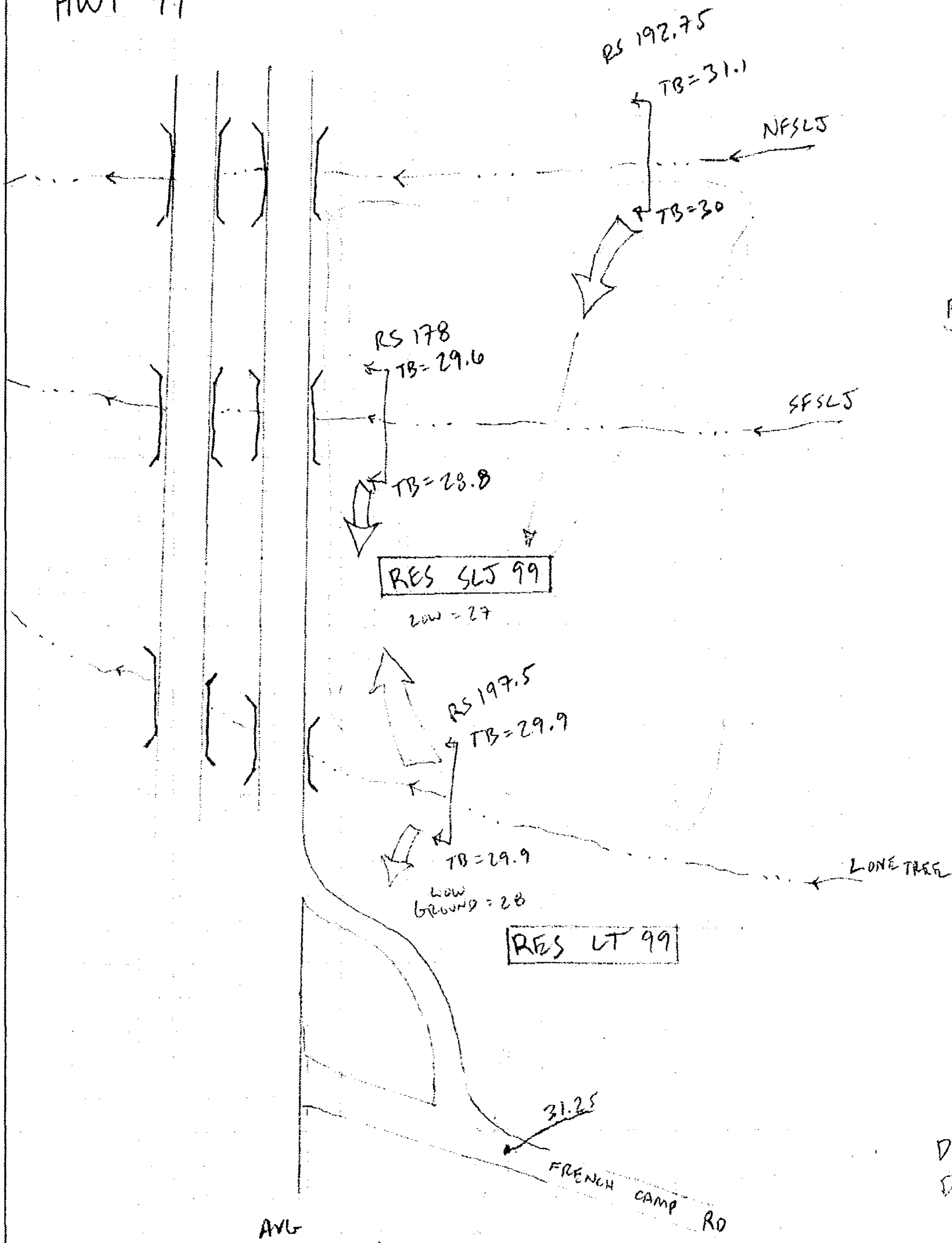
Attachment 6- A. Summary of Isolated Areas for French Camp Slough Subbasins

Subbasin	Measured Area [Sq. Mi.]	Percent of Area Estimated to be Isolated[a] [%]	Area Used in Model [sq. mi.]
LT A1	2.44	5%	2.32
LT A2	4.00	5%	3.80
LT A3	0.16	10%	0.15
LT B1	3.55	10%	3.20
LT B2	3.43	5%	3.26
LT B5	2.41	15%	2.05
LT C1	2.91	15%	2.47
LT C2	3.13	15%	2.66
LT C3	1.02	5%	0.97
LT C4a	1.87	10%	1.68
LT C4b	1.58	25%	1.19
LT D1	3.90	10%	3.51
LT D2	2.65	15%	2.25
TE A1	3.67	5%	3.49
TE B1	3.62	25%	2.72
TE B2	3.03	25%	2.27
TE B4	3.77	25%	2.83
TE C1	3.32	20%	2.66
TE D2	7.12	25%	5.34
TE D3	3.62	20%	2.90
TE F1	6.04	15%	5.13
TE F2	4.09	15%	3.47
TOTAL (Includes all 85 subbasins)	428.38	2.6%	417.35

[a] Percentages are based on field investigations conducted for the 2007 Tidewater Model

Attachment 6- B. Drawings and Hydraulic Calculations from the 2007 Tidewater Study

HWY 99



RS 192.75

Q	ELEV	DIV
1250	30.1	30
1530	30.6	200
1750	31.2	400
2000	31.8	550

RS 178

Q	ELEV	DIV
1750	29.0	10
2000	29.5	220
2300	30	450
2500	30.25	600

RS 197.5

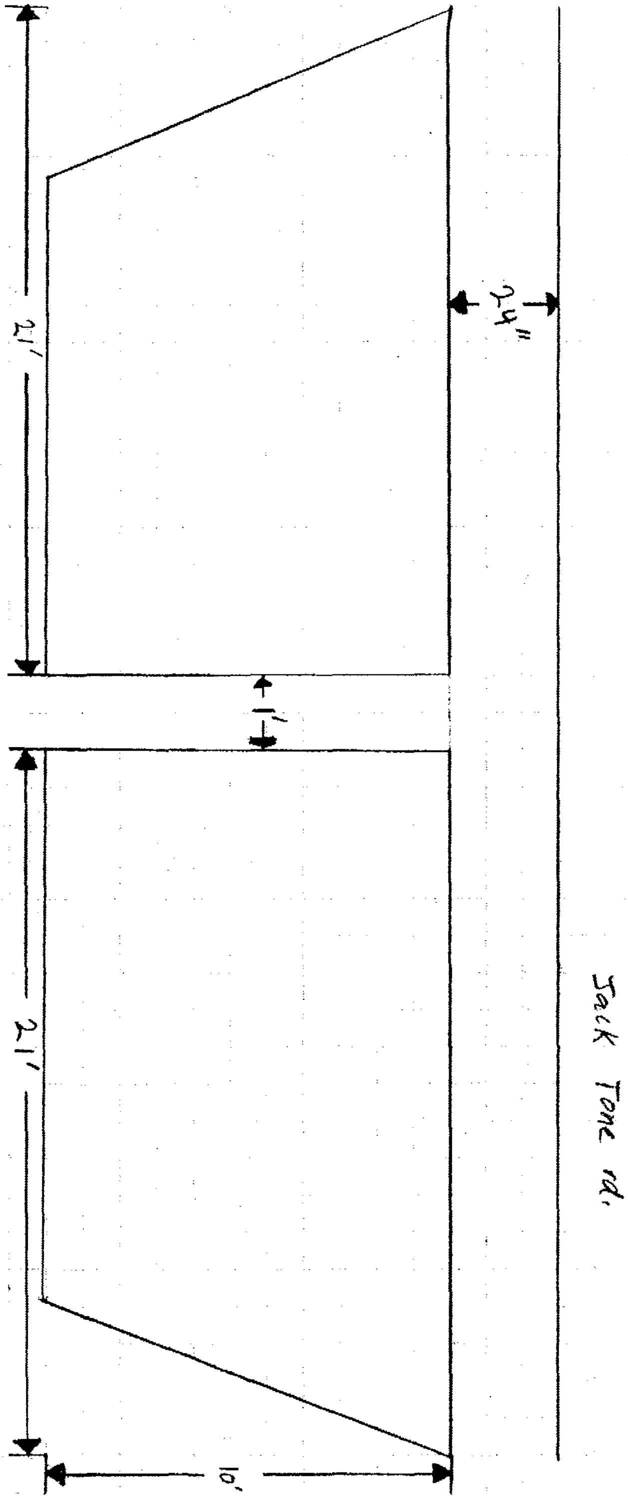
Q	ELEV	DIV
750	29.6	0
1000	30.4	150
1250	31.0	350
1500	31.6	550
1750	32.1	700
2000	32.6	850

DIV R = 0.75 DIV
DIV L = 0.25 DIV

	ELEV	AREA	AVG DEPTH	STORAGE	OUTFLOW
RES	29	160	2	320	10
	30	293	2.5	730	20
SLJ99	31	480	3	1440	35
	32	1215	3.3	4000	200
	33	—	4		—
	30	36	2	72	5
↗ 99	31	76	2.5	190	15
	32	115	3	345	400
	33	135	3.5	475	2000

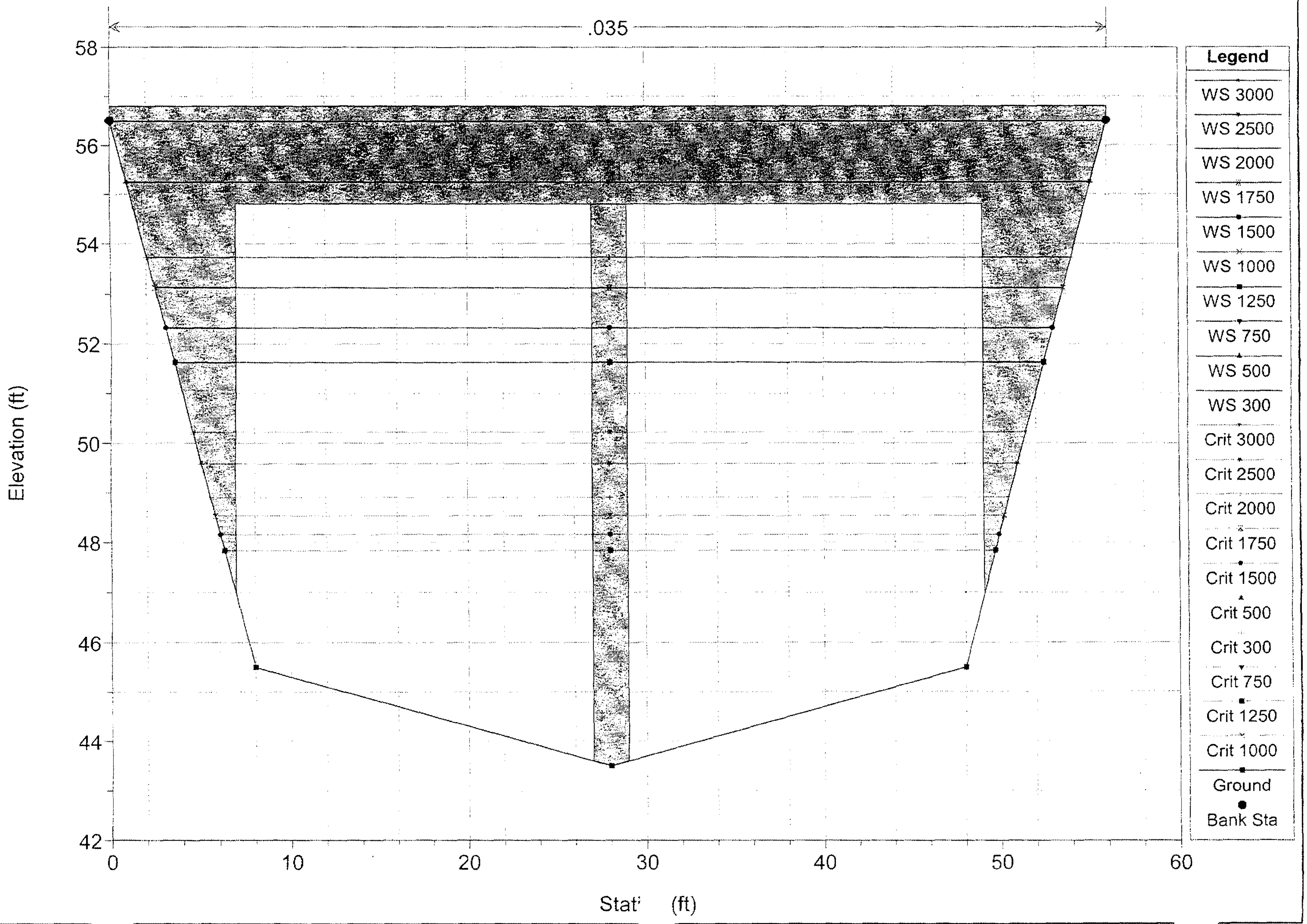
400 → OVER FRENCH CAMP 0.75'
1700' WEIR
UNDER 99?

NORTH FORK SOUTH LITTLEJOHNS CREEK
@ JACK TONE ROAD



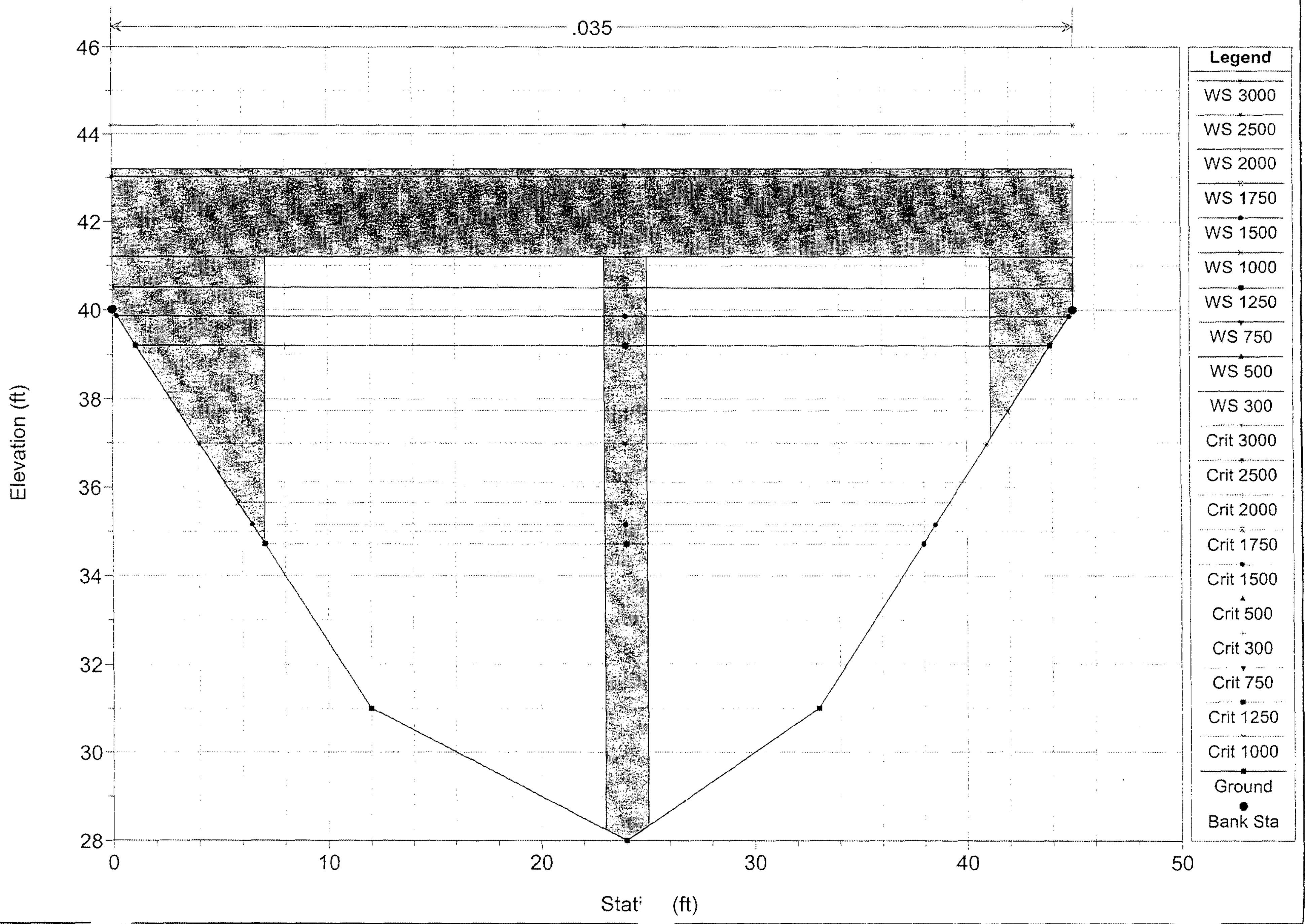
Upper Watershed Plan: Test

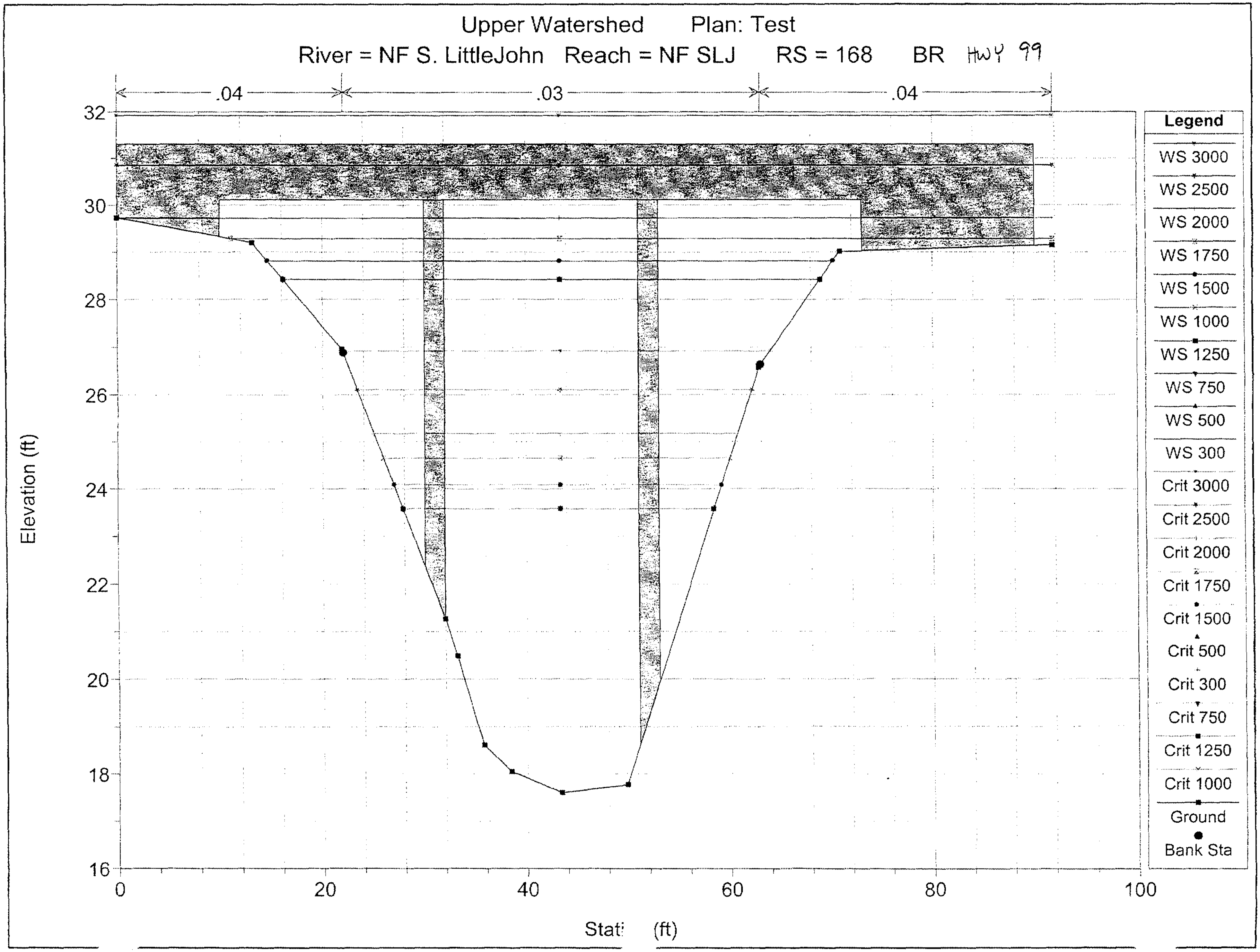
River = NF S. LittleJohn Reach = NF SLJ RS = 421.5 BR JACK TONE RD



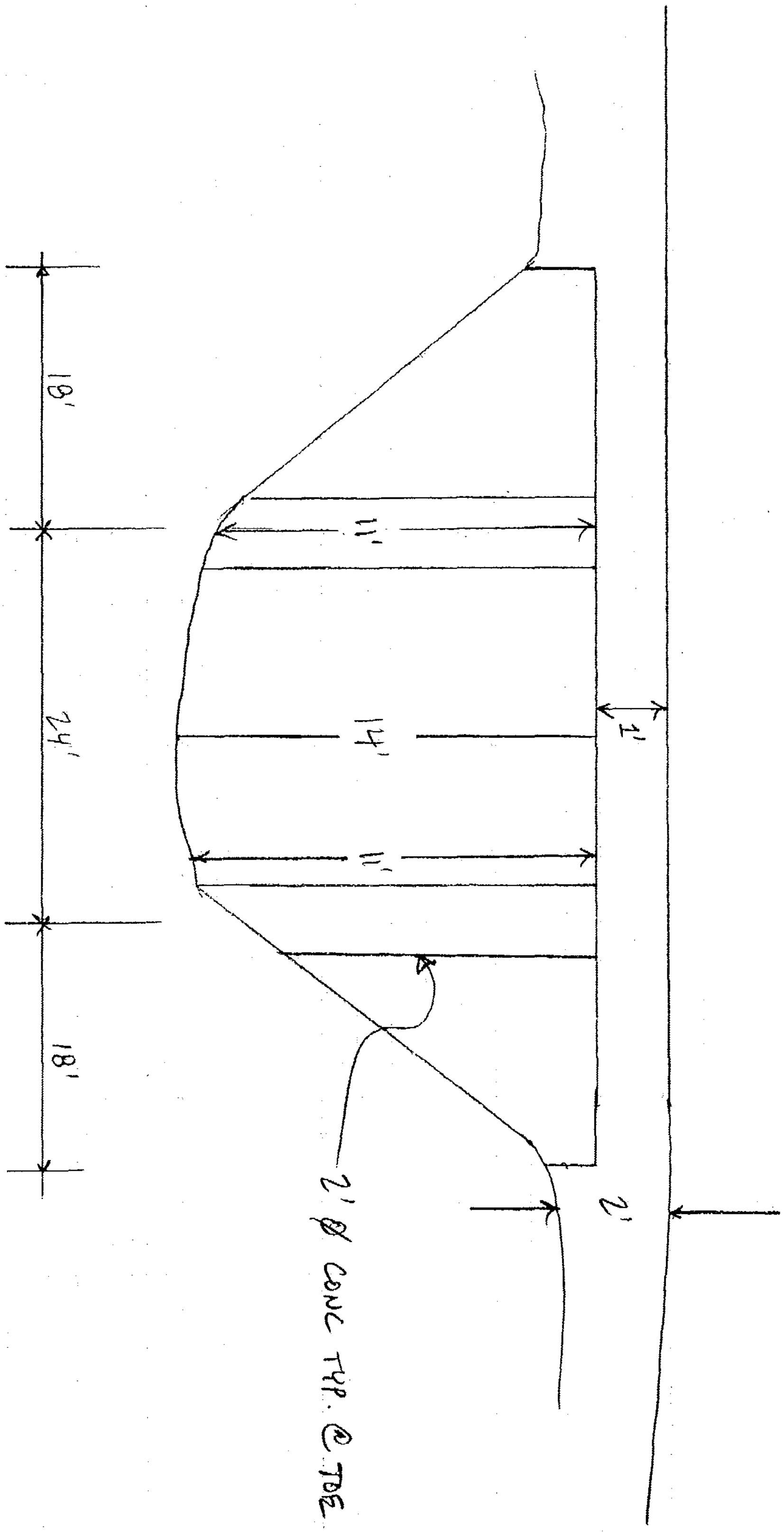
Upper Watershed Plan: Test

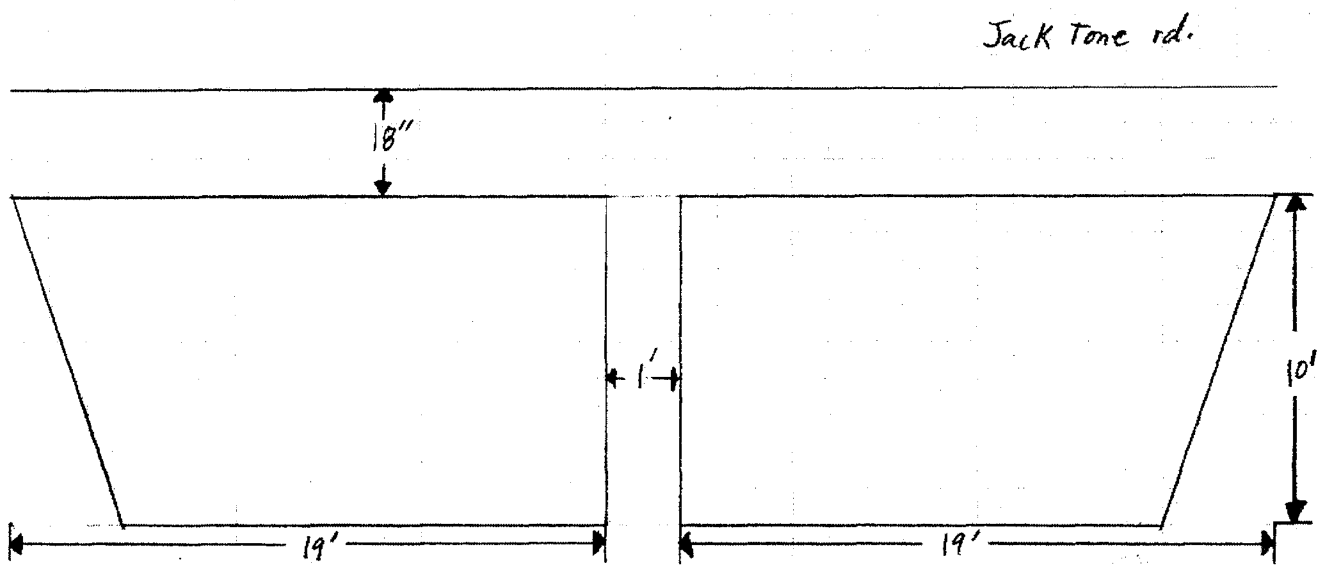
River = NF S. LittleJohn Reach = NF SLJ RS = 295.5 BR AUSTIN RD



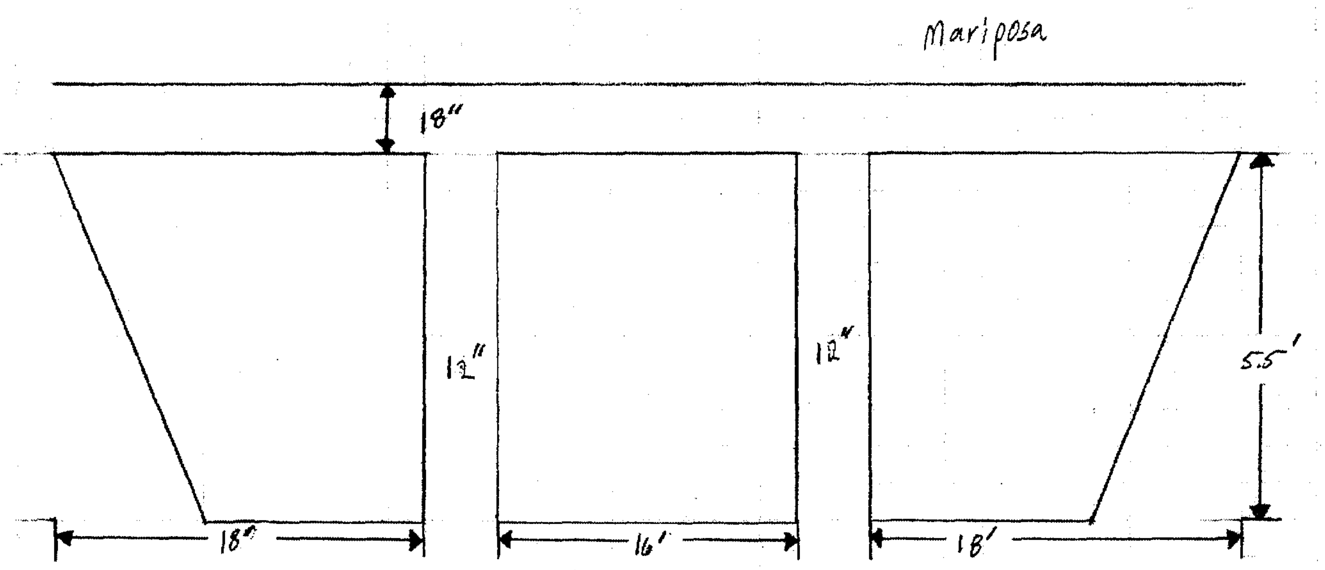

Upper Watershed Plan: Test
River = NF S. LittleJohn Reach = NF SLJ RS = 168 BR HWY 99
.04
.03
.04
32
30
28
26
24
22
20
18
16
0
20
40
60
80
100
Elevation (ft)
Stati (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1000
WS 1250
WS 750
WS 500
WS 300
Crit 3000
Crit 2500
Crit 2000
Crit 1750
Crit 1500
Crit 500
Crit 300
Crit 750
Crit 1250
Crit 1000
Ground
Bank Sta

S. FORK S. LITTLEJOHNS
@ AUSTIN ROAD

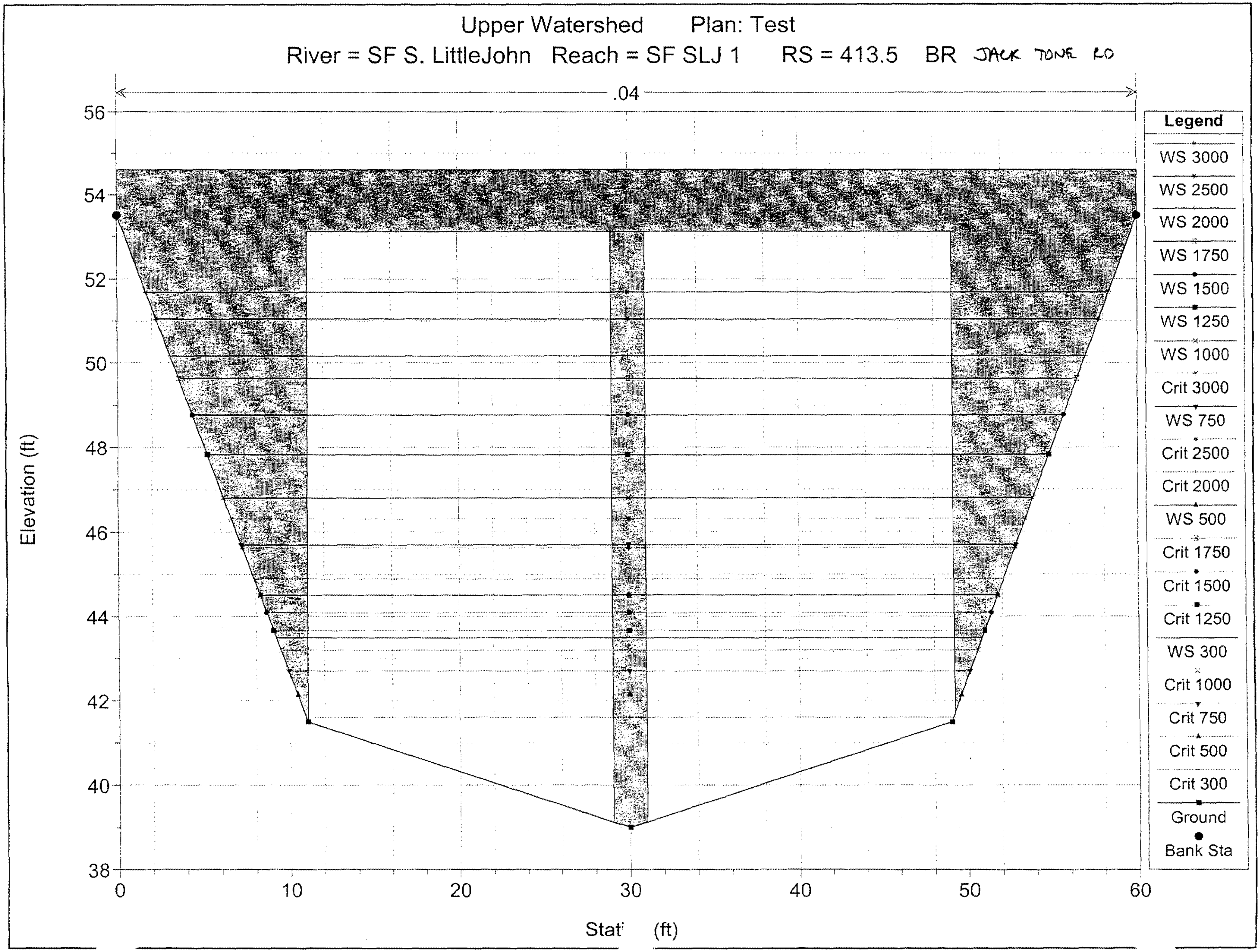




South Fork South Littlejohns CreeK and Jack Tone rd

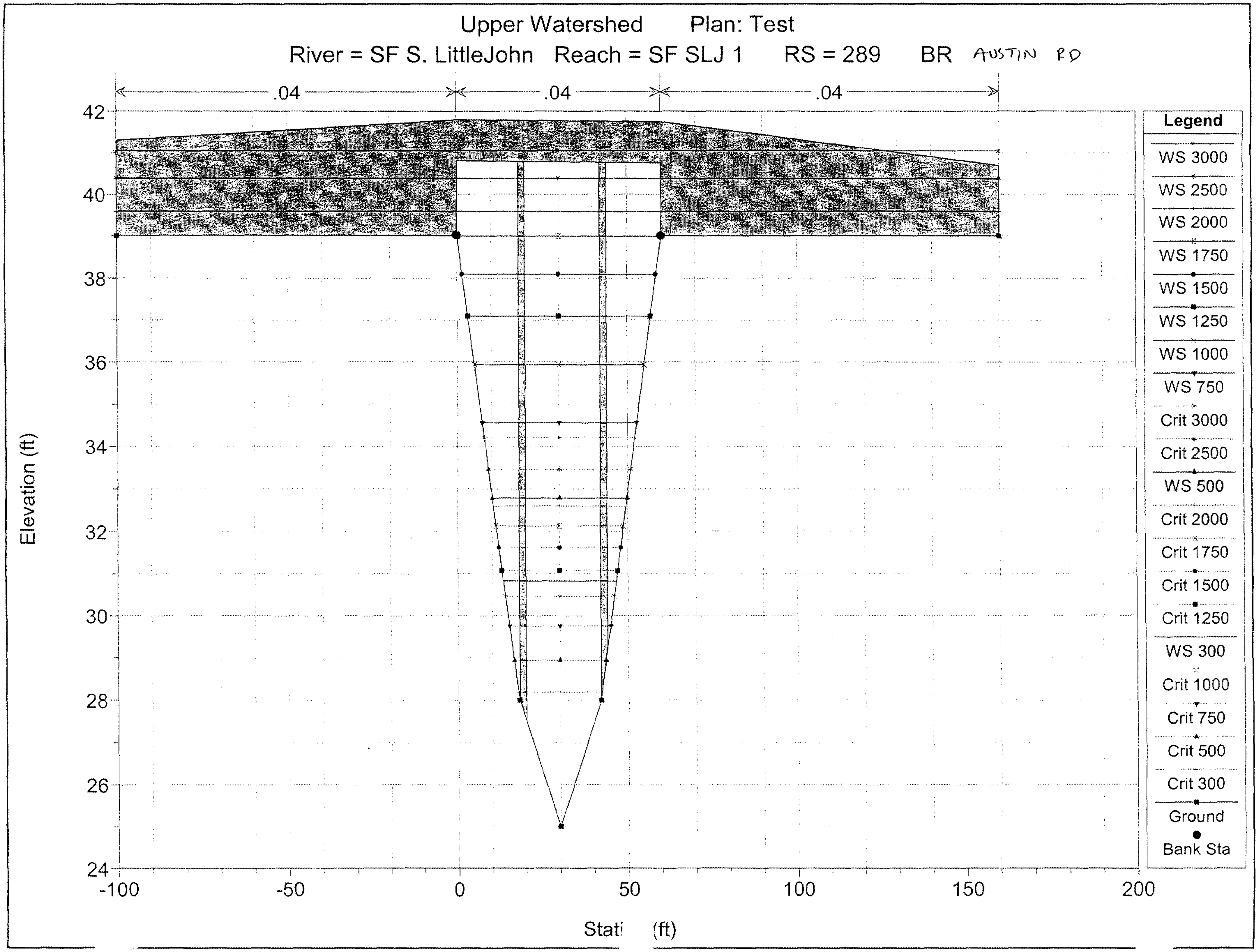


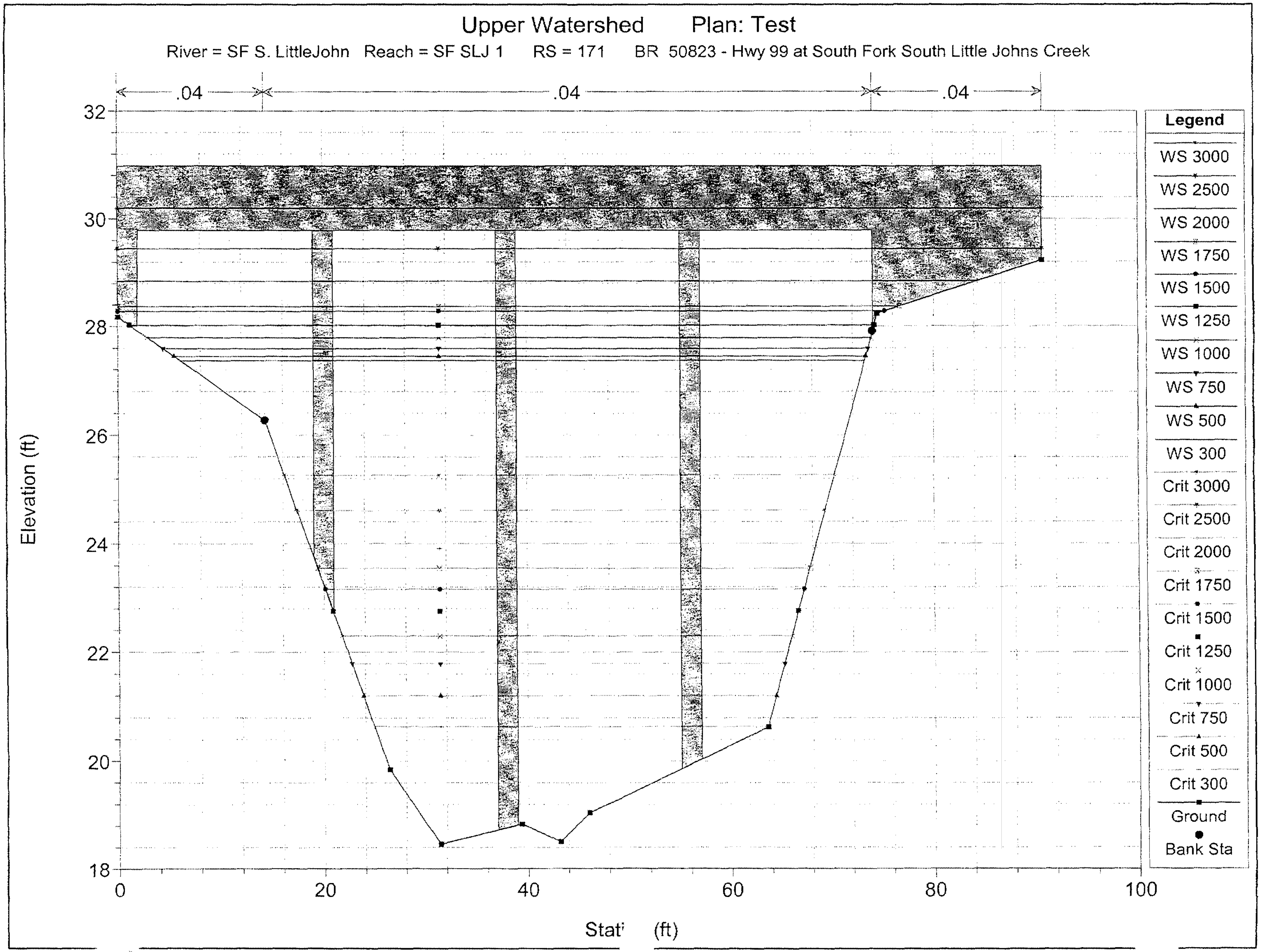
S. Littlejohn Creek and Mariposa



Upper Watershed Plan: Test

River = SF S. LittleJohn Reach = SF SLJ 1 RS = 413.5 BR JACK TONE RD

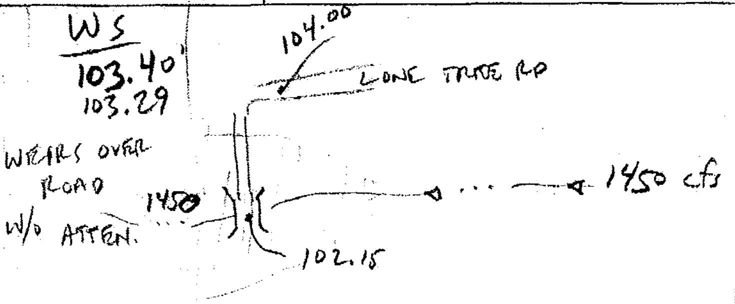

Upper Watershed Plan: Test
River = SF S. LittleJohn Reach = SF SLJ 1 RS = 289 BR AUSTIN RD
.04
.04
.04
Elevation (ft)
42
40
38
36
34
32
30
28
26
24
-100
-50
0
50
100
150
200
Stati (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
Crit 3000
Crit 2500
WS 500
Crit 2000
Crit 1750
Crit 1500
Crit 1250
WS 300
Crit 1000
Crit 750
Crit 500
Crit 300
Ground
Bank Sta


Upper Watershed Plan: Test
River = SF S. LittleJohn Reach = SF SLJ 1 RS = 171 BR 50823 - Hwy 99 at South Fork South Little Johns Creek
.04
.04
.04
Elevation (ft)
32
30
28
26
24
22
20
18
0
20
40
60
80
100
Stat (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
WS 500
WS 300
Crit 3000
Crit 2500
Crit 2000
Crit 1750
Crit 1500
Crit 1250
Crit 1000
Crit 750
Crit 500
Crit 300
Ground
Bank Sta

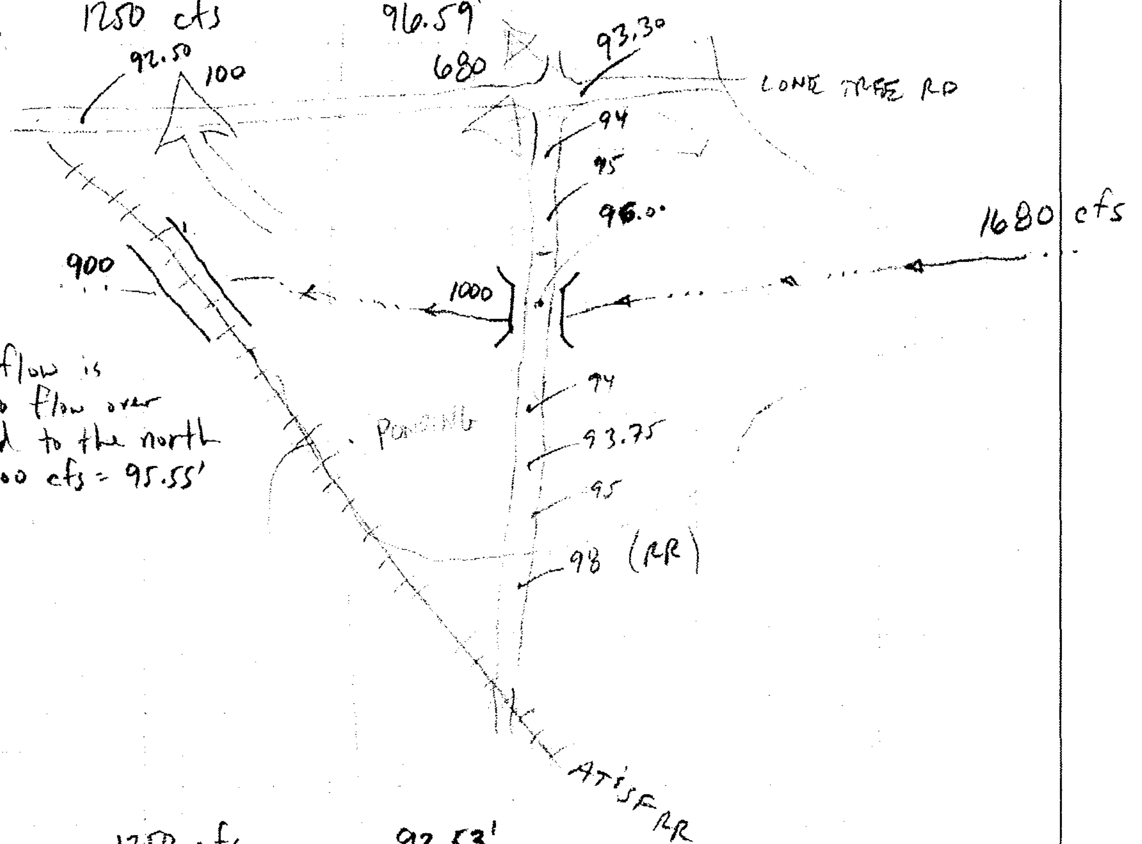
LONE TREE CREEK | CROSSING RATING TABLE

WS

RS 877 BRENNAN RD: 1750 cfs 103.40'
1450 cfs 103.29



RS 812 SEXTON RD: 1250 cfs 96.59'



AT 1250 cfs, flow is deep enough to flow over Lone Tree road to the north 3' Deep. 1000 cfs = 95.55'

RS 808 AT&SF RR 1250 cfs 93.53'
1000 cfs 93.27'

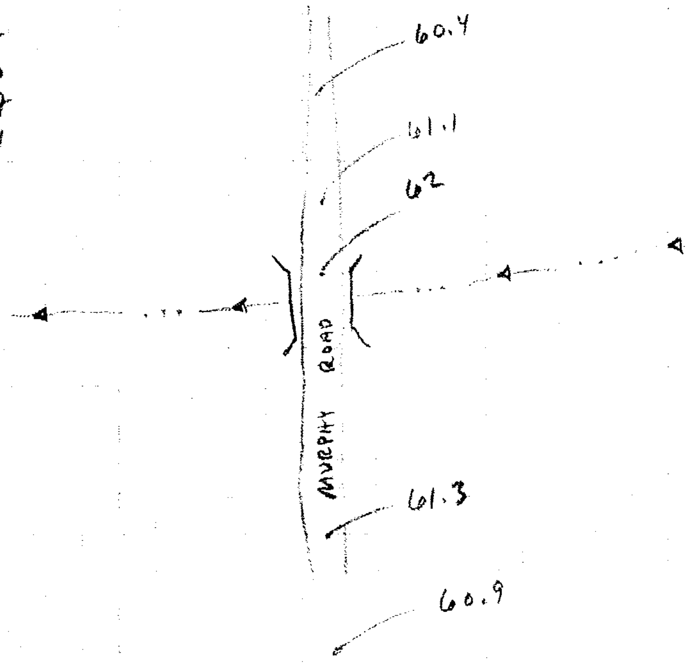
NEGLIGIBLE STORAGE; EXCESS FLOW IS DIVERTED TO NORTH BEHIND RR
SEXTON + AT&SFRR COMBINED INFLOW-DIV TABLE

IN	DIV
0	0
800	0
1000	150
1200	330
1500	600
2000	1050

LONE TREE CREEK | CROSSINGS RATING TABLES

RS 560

MURPHY RD:	WS
1500	62.82
1250	62.70
1000	62.47
750	61.94



ELEV	AREA	STORAGE	OUTFLOW
60.05	0	0	500
61.94	580	500	750
62.47	800	650	1000
62.7	850	675	1250
62.8	900	700	1500

RESULTS IN HYDROGRAPH:

USE DIVERSION TO METERED RES TO SIMULATE OVERBANK STORAGE

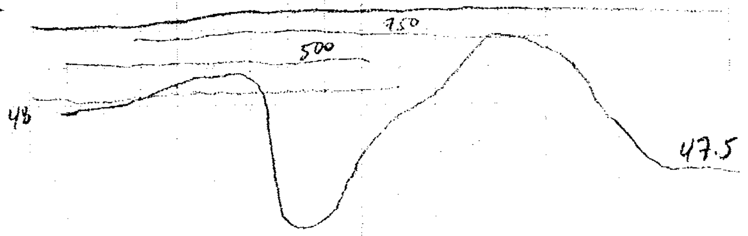
INFLOW	DIV
0	0
500	0
750	150
1000	350
1250	550
1500	700

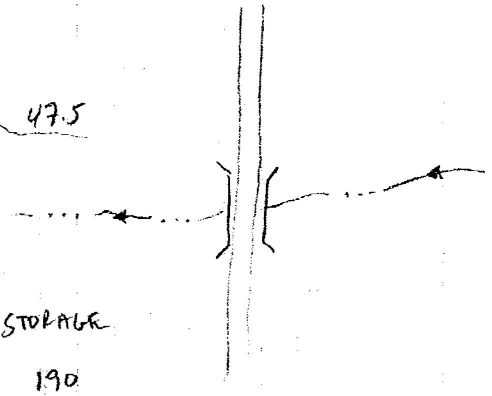
→

STORAGE	DISCHARGE
0	0
100	10
300	50
600	100
700	120

RS 450 JACK TONE RD.

RD= 50.95





Q	ELEV	AREA	AVG. DEPTH	STORAGE
500	49.2	190	1	190
750	49.8	280	1.5	420
1000	50.9	645	2	1300
1250	51.3	790	2.2	1740
1500	51.6	940	2.4	2300

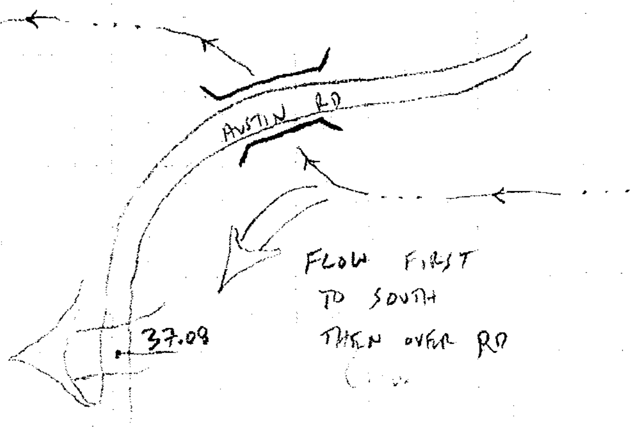
AGAIN, USE DIV TO RES TO AVOID THIS HYDROGRAPH:

IN	DIV	STOR.	OUTFLOW
0	0	0	0
500	50	190	10
750	200	420	20
1000	350	1300	40
1250	475	1740	60
1500	600	2300	70

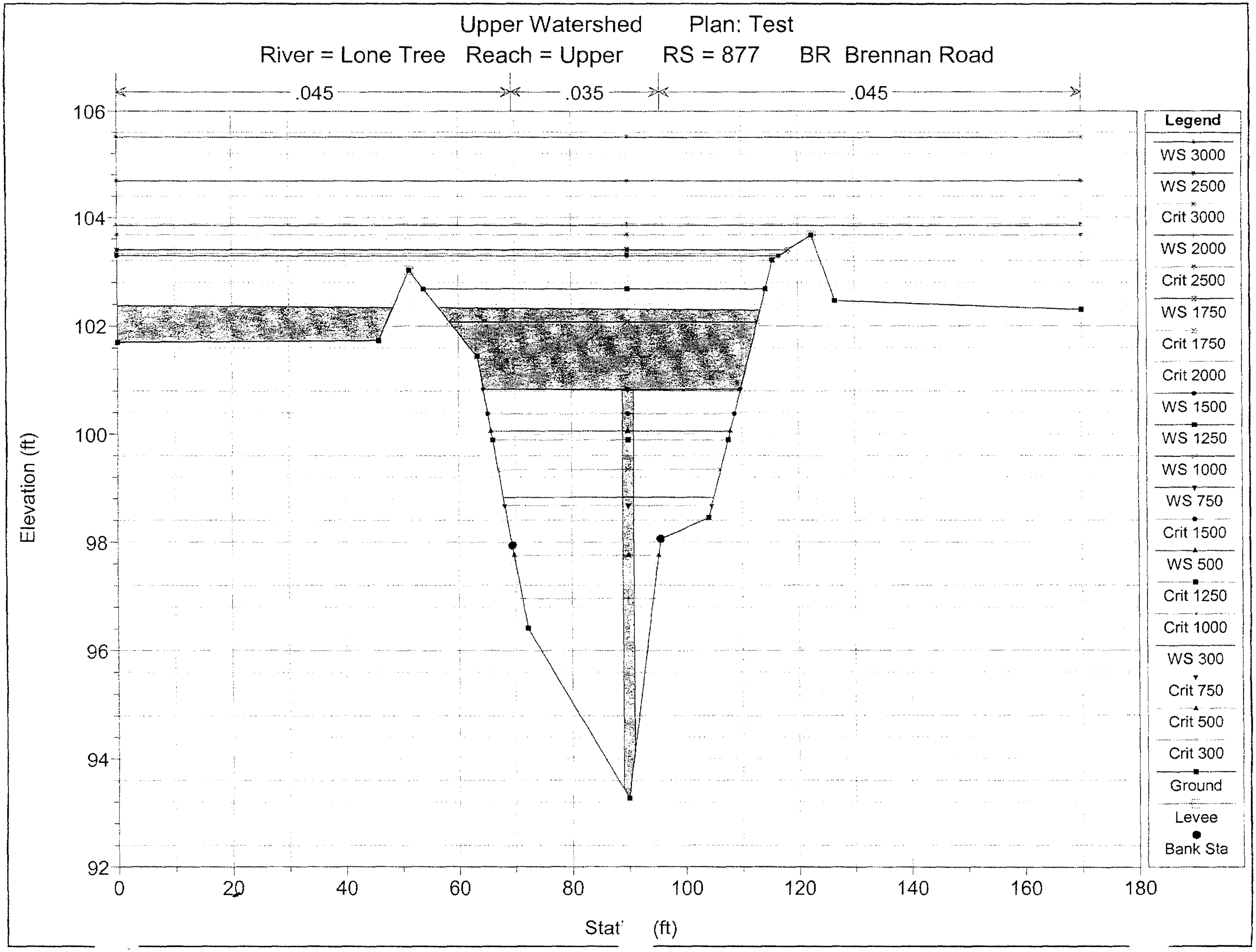
RS 302.5 AUSTIN RD

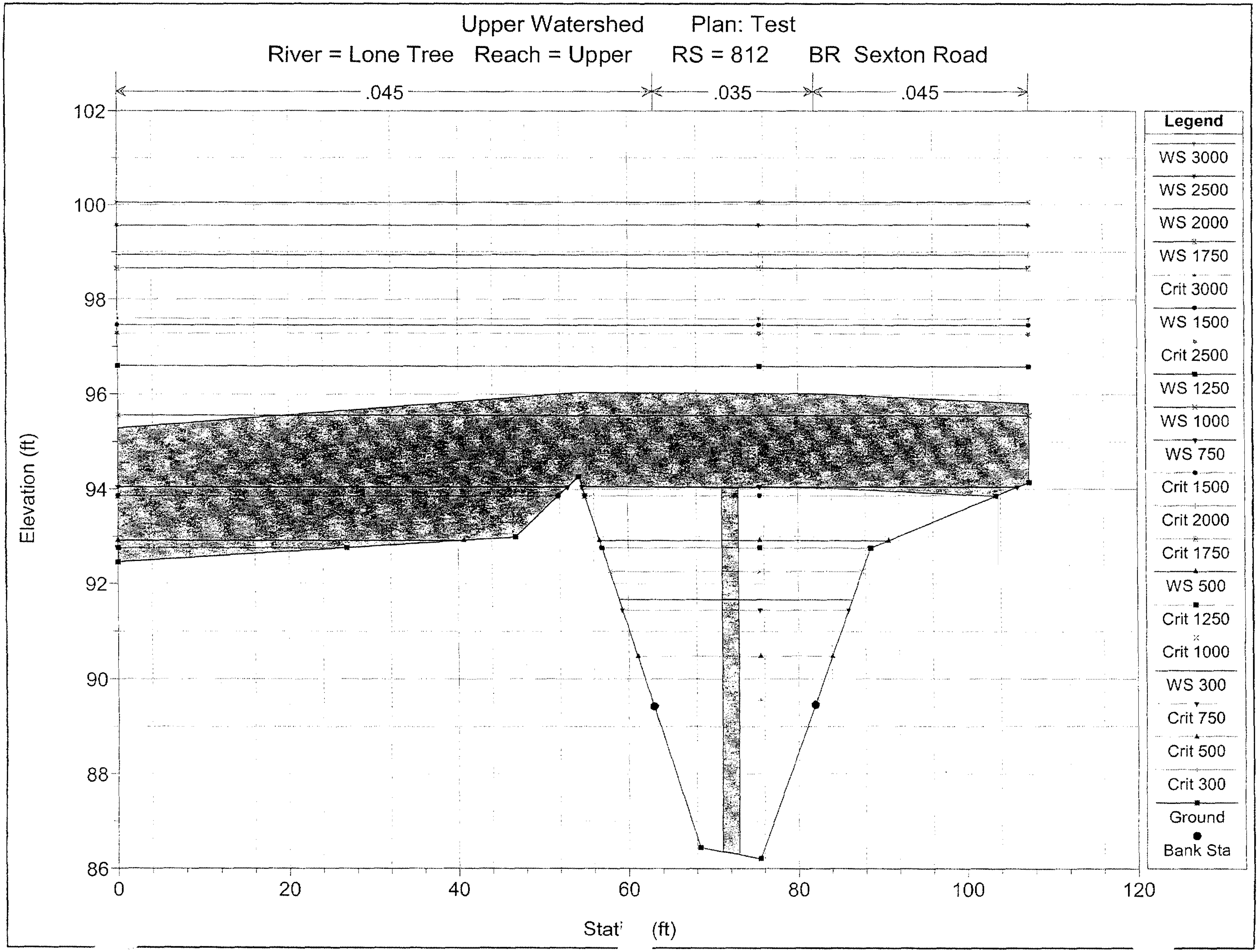
DECK ELEV= 39.4

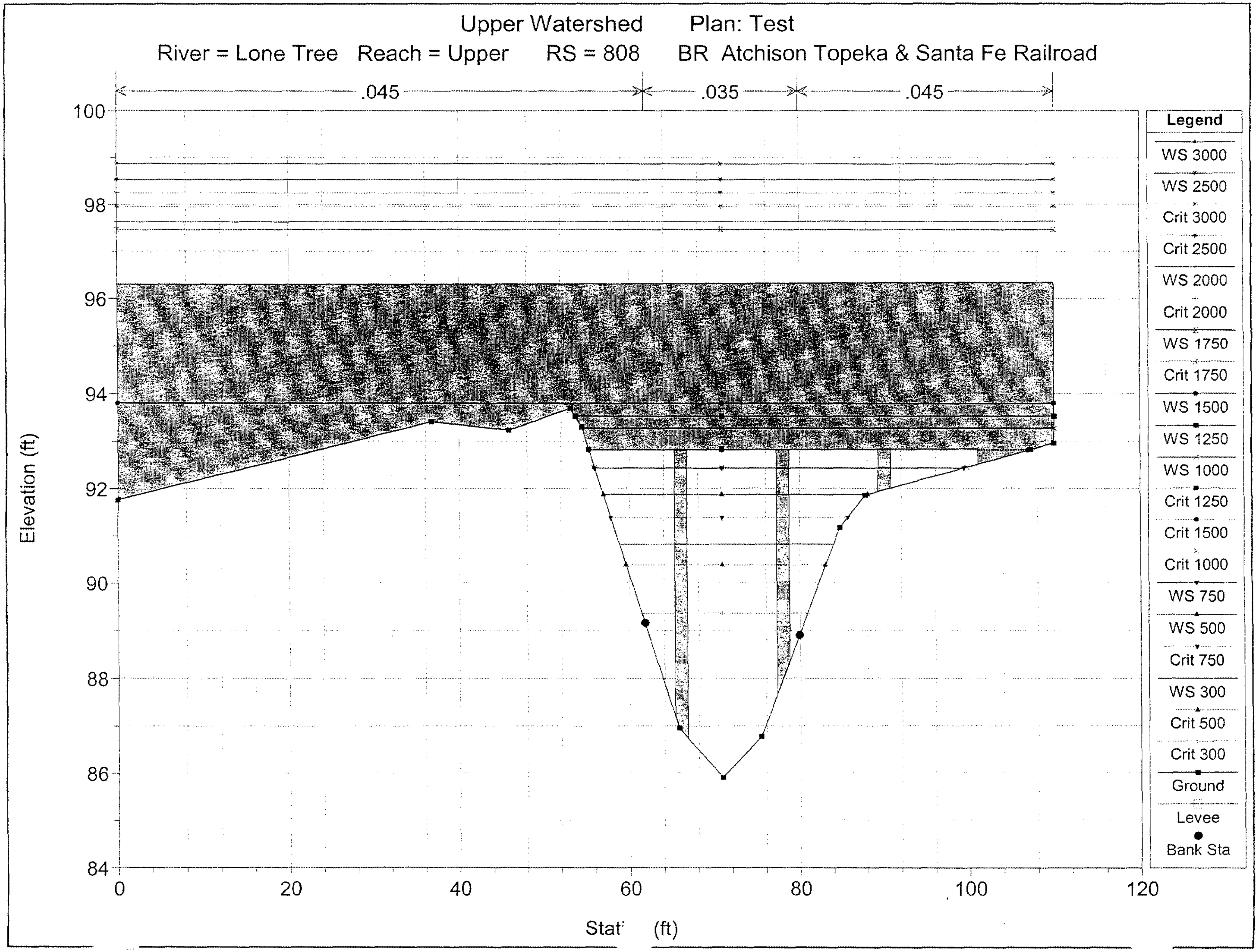
Q	ELEV	AREA	DEPTH	STORAGE
1250	37	12.5	0.1	1.3
1500	38.6	32	1	32
1750	38.9	49	1.1	55
2000	39.1	65	1.3	85
2500	39.5	83	1.5	125
3000	40.0	145	1.7	250

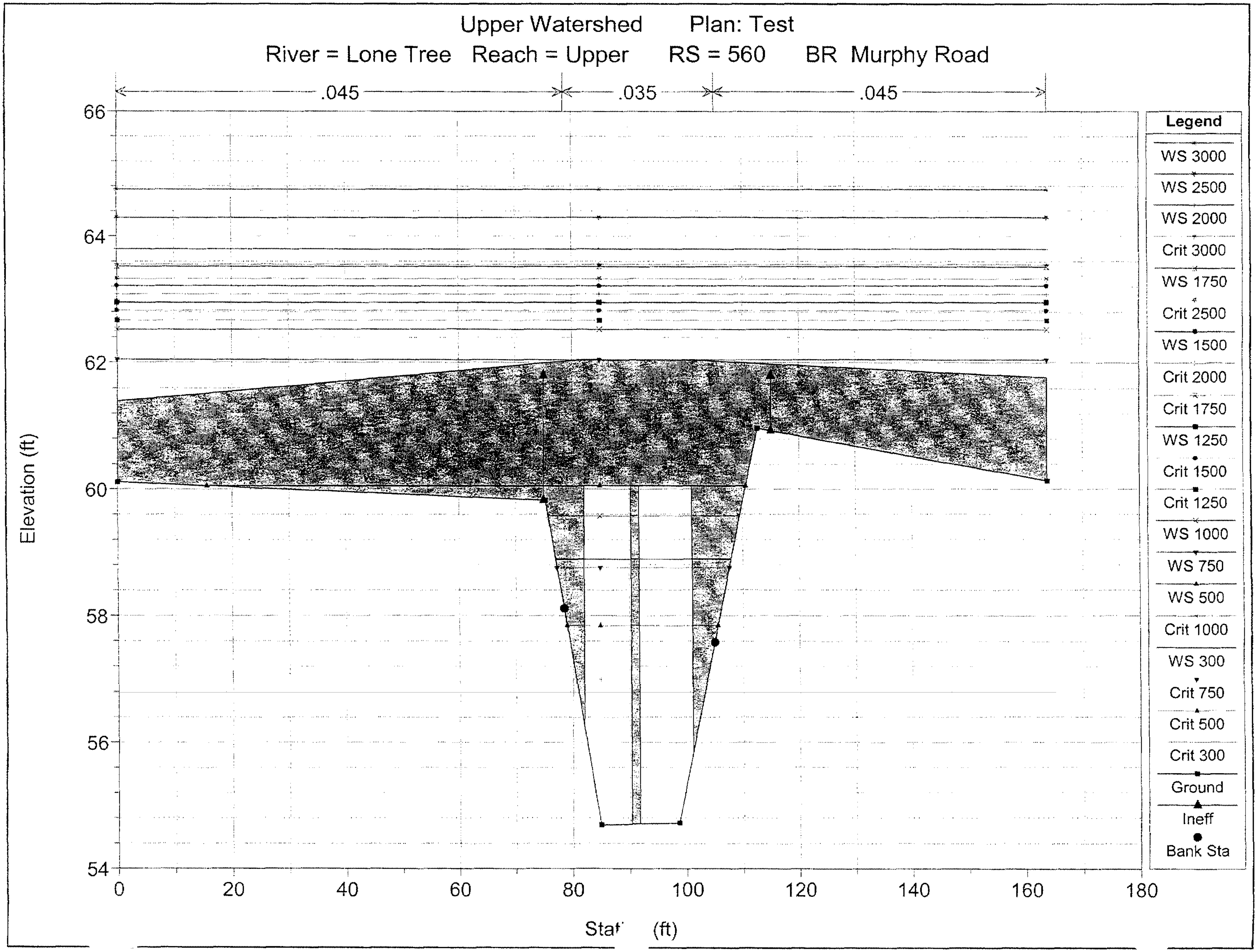


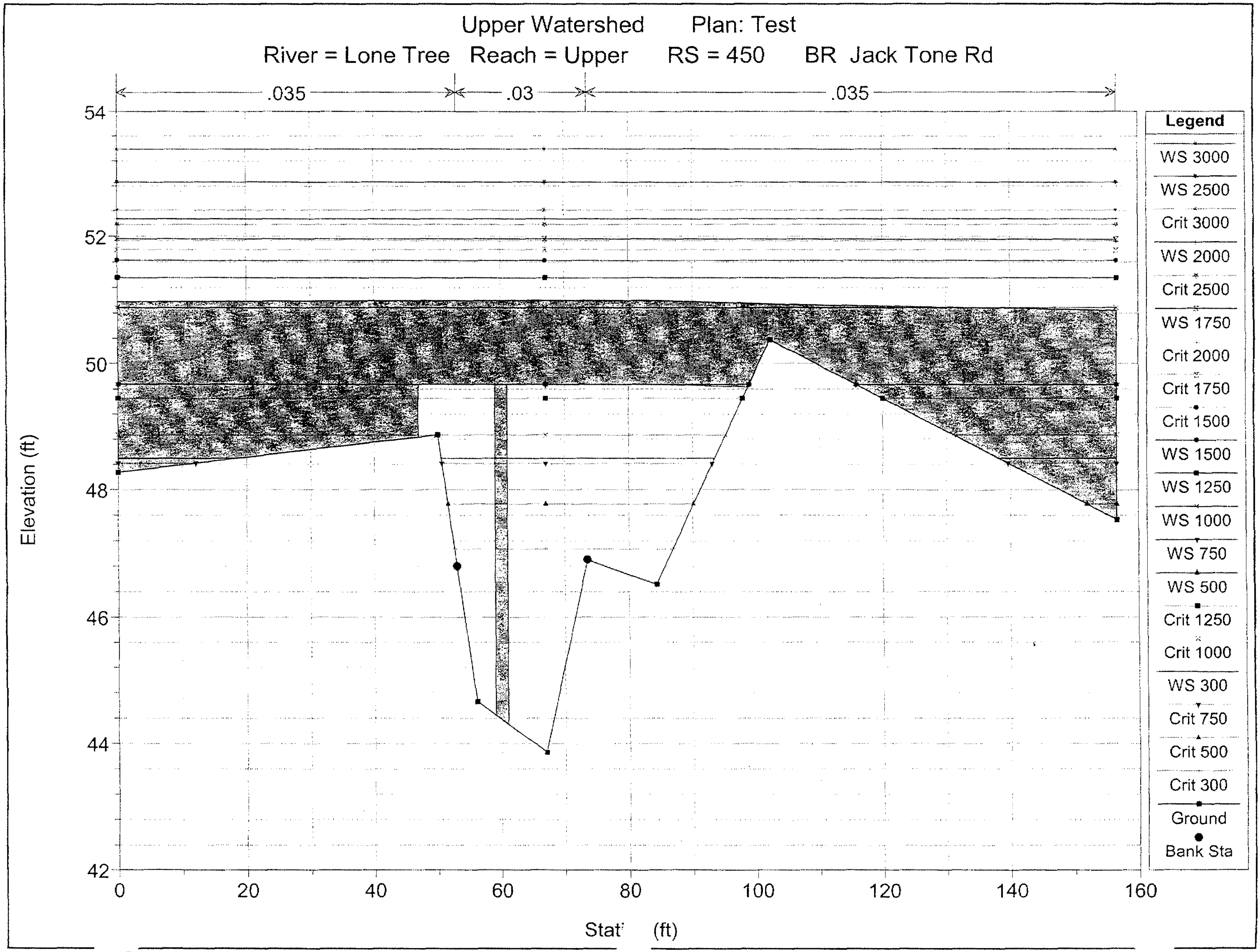
FLOW FIRST TO SOUTH THEN OVER RD

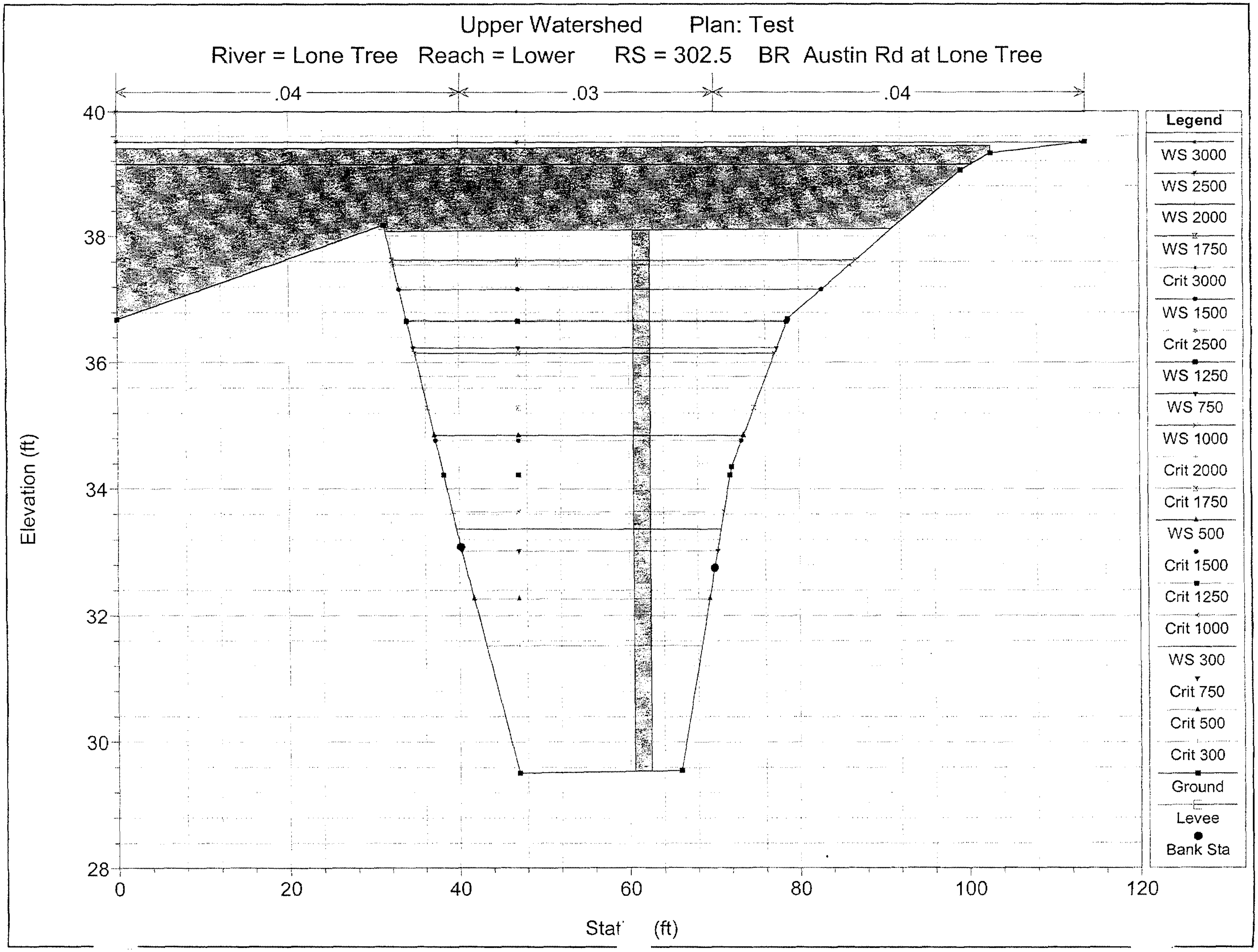

Upper Watershed Plan: Test
River = Lone Tree Reach = Upper RS = 877 BR Brennan Road
.045
.035
.045
Elevation (ft)
106
104
102
100
98
96
94
92
0
20
40
60
80
100
120
140
160
180
Stat (ft)
Legend
WS 3000
WS 2500
Crit 3000
WS 2000
Crit 2500
WS 1750
Crit 1750
Crit 2000
WS 1500
WS 1250
WS 1000
WS 750
Crit 1500
WS 500
Crit 1250
Crit 1000
WS 300
Crit 750
Crit 500
Crit 300
Ground
Levee
Bank Sta

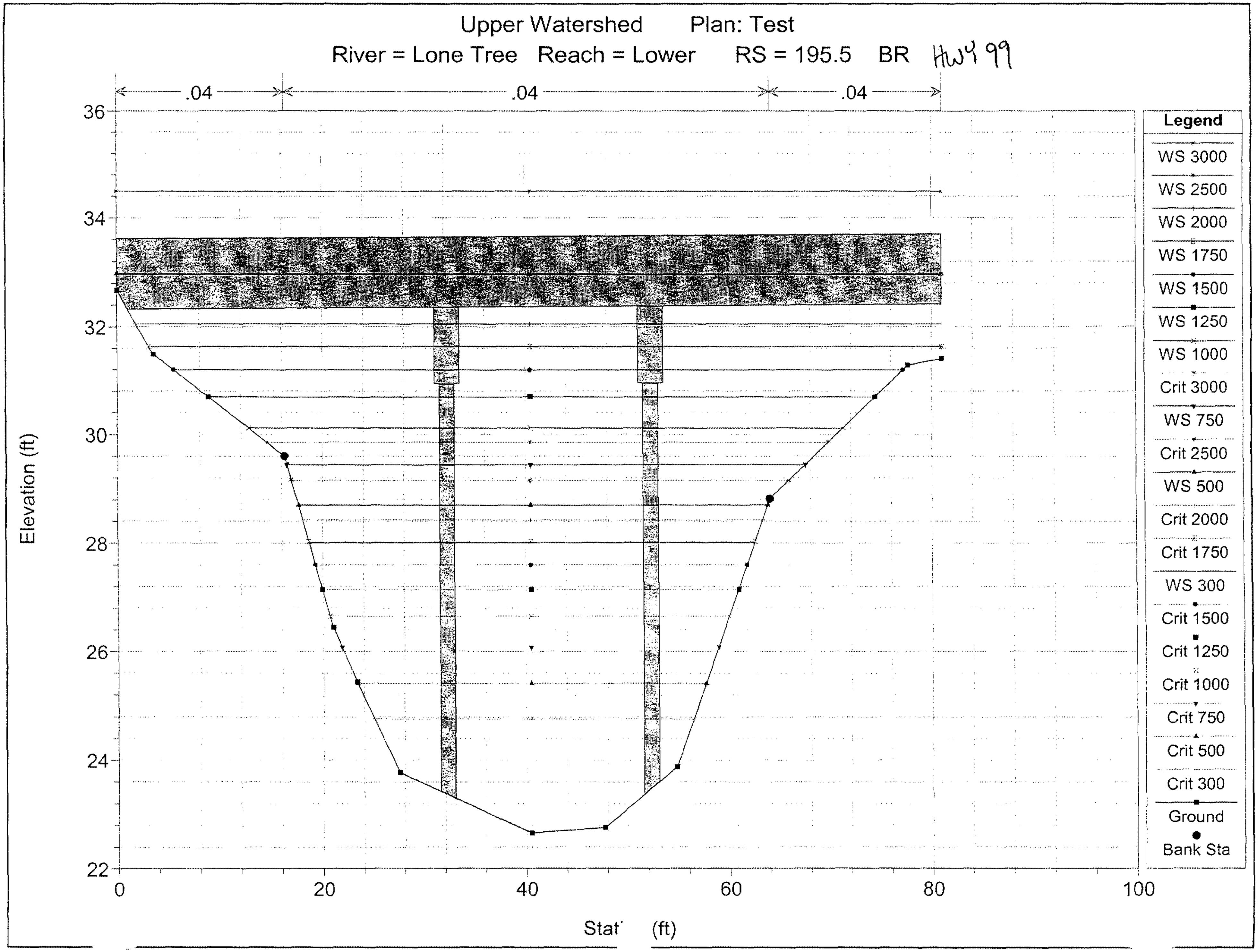

Upper Watershed Plan: Test
River = Lone Tree Reach = Upper RS = 812 BR Sexton Road
.045
.035
.045
102
100
98
96
94
92
90
88
86
Elevation (ft)
0
20
40
60
80
100
120
Stat' (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
Crit 3000
WS 1500
Crit 2500
WS 1250
WS 1000
WS 750
Crit 1500
Crit 2000
Crit 1750
WS 500
Crit 1250
Crit 1000
WS 300
Crit 750
Crit 500
Crit 300
Ground
Bank Sta

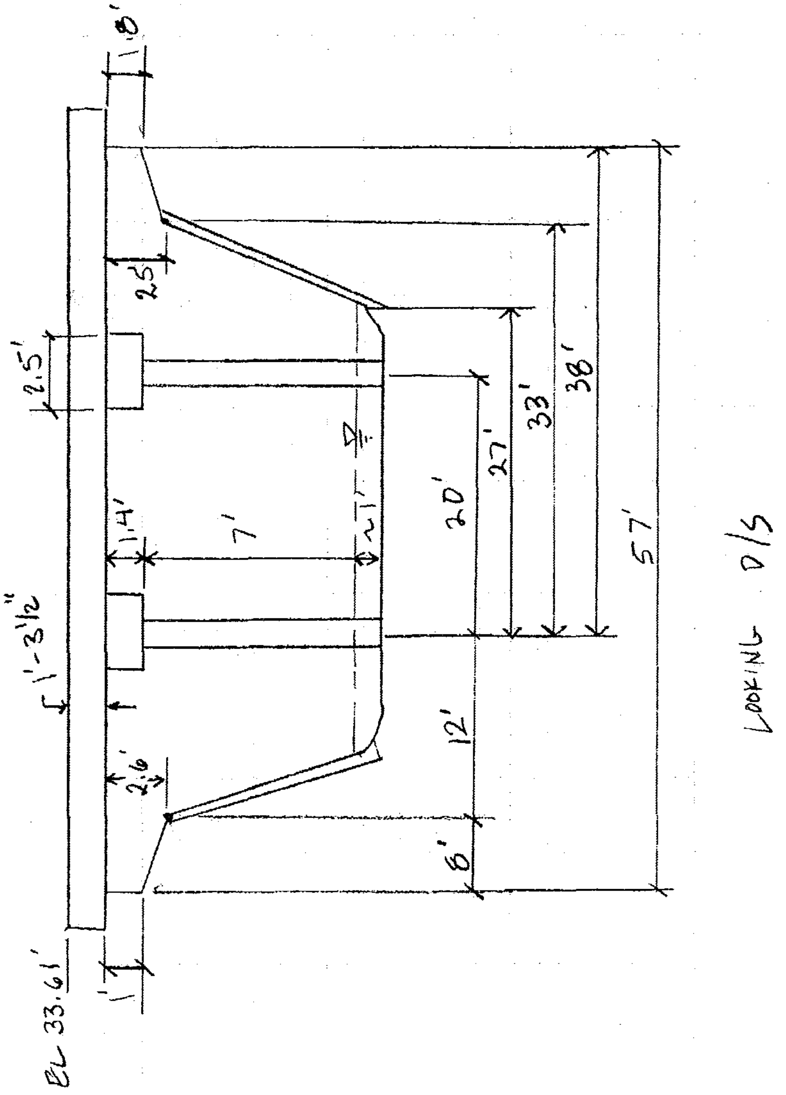

Upper Watershed Plan: Test
River = Lone Tree Reach = Upper RS = 808 BR Atchison Topeka & Santa Fe Railroad
.045
.035
.045
Elevation (ft)
100
98
96
94
92
90
88
86
84
0
20
40
60
80
100
120
Stat (ft)
Legend
WS 3000
WS 2500
Crit 3000
Crit 2500
WS 2000
Crit 2000
WS 1750
Crit 1750
WS 1500
WS 1250
WS 1000
Crit 1250
Crit 1500
Crit 1000
WS 750
WS 500
Crit 750
WS 300
Crit 500
Crit 300
Ground
Levee
Bank Sta


Upper Watershed Plan: Test
River = Lone Tree Reach = Upper RS = 560 BR Murphy Road
.045
.035
.045
Elevation (ft)
66
64
62
60
58
56
54
0
20
40
60
80
100
120
140
160
180
Stat (ft)
Legend
WS 3000
WS 2500
WS 2000
Crit 3000
WS 1750
Crit 2500
WS 1500
Crit 2000
Crit 1750
WS 1250
Crit 1500
Crit 1250
WS 1000
WS 750
WS 500
Crit 1000
WS 300
Crit 750
Crit 500
Crit 300
Ground
Ineff
Bank Sta


Upper Watershed Plan: Test
River = Lone Tree Reach = Upper RS = 450 BR Jack Tone Rd
.035
.03
.035
Legend
WS 3000
WS 2500
Crit 3000
WS 2000
Crit 2500
WS 1750
Crit 2000
Crit 1750
Crit 1500
WS 1500
WS 1250
WS 1000
WS 750
WS 500
Crit 1250
Crit 1000
WS 300
Crit 750
Crit 500
Crit 300
Ground
Bank Sta
Elevation (ft)
54
52
50
48
46
44
42
0
20
40
60
80
100
120
140
160
Stat' (ft)

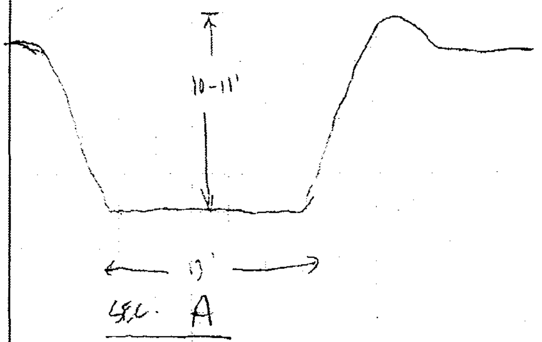

Upper Watershed Plan: Test
River = Lone Tree Reach = Lower RS = 302.5 BR Austin Rd at Lone Tree
.04
.03
.04
Elevation (ft)
40
38
36
34
32
30
28
0
20
40
60
80
100
120
Staf (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
Crit 3000
WS 1500
Crit 2500
WS 1250
WS 750
WS 1000
Crit 2000
Crit 1750
WS 500
Crit 1500
Crit 1250
Crit 1000
WS 300
Crit 750
Crit 500
Crit 300
Ground
Levee
Bank Sta

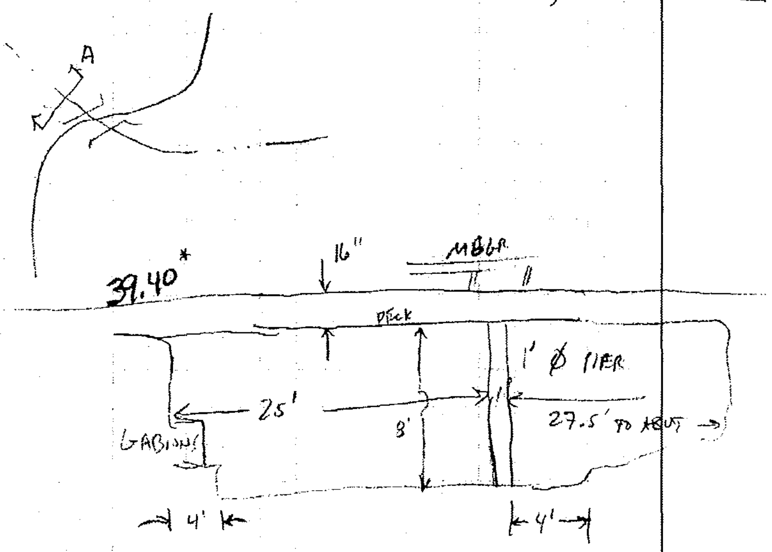

Upper Watershed Plan: Test
River = Lone Tree Reach = Lower RS = 195.5 BR Hwy 99
.04
.04
.04
Elevation (ft)
36
34
32
30
28
26
24
22
0
20
40
60
80
100
Stat' (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
Crit 3000
WS 750
Crit 2500
WS 500
Crit 2000
Crit 1750
WS 300
Crit 1500
Crit 1250
Crit 1000
Crit 750
Crit 500
Crit 300
Ground
Bank Sta


1.8'
2.5'
2.5'
1.4'
7'
~1'
20'
27'
33'
38'
57'
12'
8'
2.6'
1'-3½"
EL 33.61'
1'
LOOKING D/S

TELL KSN WHEN SHOOTING X-SEC, CONTINUE INTO FIELDS (OVER LEVEES, BERMS) 1/10/06

▨ LONE TREE @ AUSTIN RD
PICS 1070 - 1075



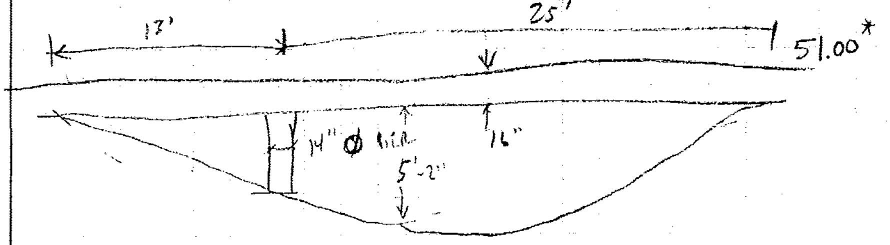


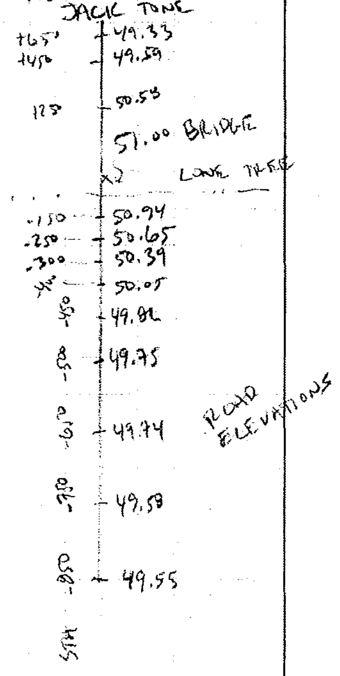
• VISIBLE HIGH WATER 24" BELOW BOTTOM OF DECK (JAN 1, 2006 STORM)

• SURVEY NEEDS: BRIDGE, 1 SEC 100' UPSTREAM
2 SEC 100', 200' DOWNSTREAM

▨ LONE TREE @ JACK TONE RD — HIGH WATER 10" BELOW BOTTOM OF DECK

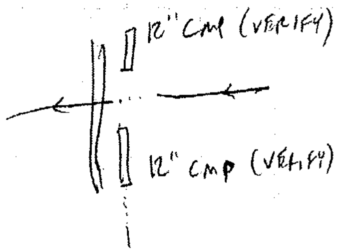
PICS 1076 - 1082





SURVEY: 3 SECTIONS: BRIDGE, 100' UP & D/S.

▨ TEMPLE @ JACK TONE RD.
NO PICS



SURVEY
BRIDGE GEOM.
U/S, D/S 100'
ROAD
SIDE CULVERTS

▨ LONE TREE @ MURPHY RD DOUBLE BOX
18' × 6' 30° SKEW

SURVEY
BRIDGE GEOM.
U/S, D/S 100'
ROAD
SIDE CULVERT

* FROM SIEGFRIED
ROAD SURVEY

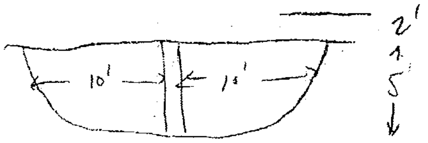
1/10/06

▨ LONE TREE @ LONE TREE RD - HIGH WATER 10" BELOW BOTTOM OF DECK

PICS 1083-1085

▨ LONE TREE @ CARROLTON

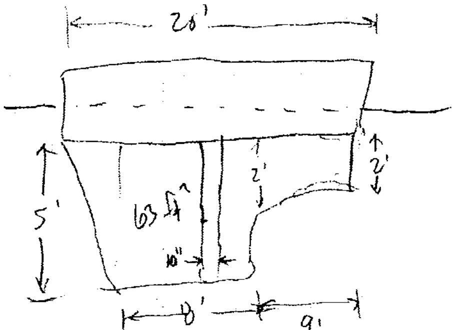
PICS 1086-88

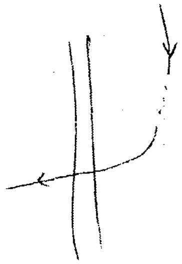


▨ LONE TREE @ VAN ALLEN

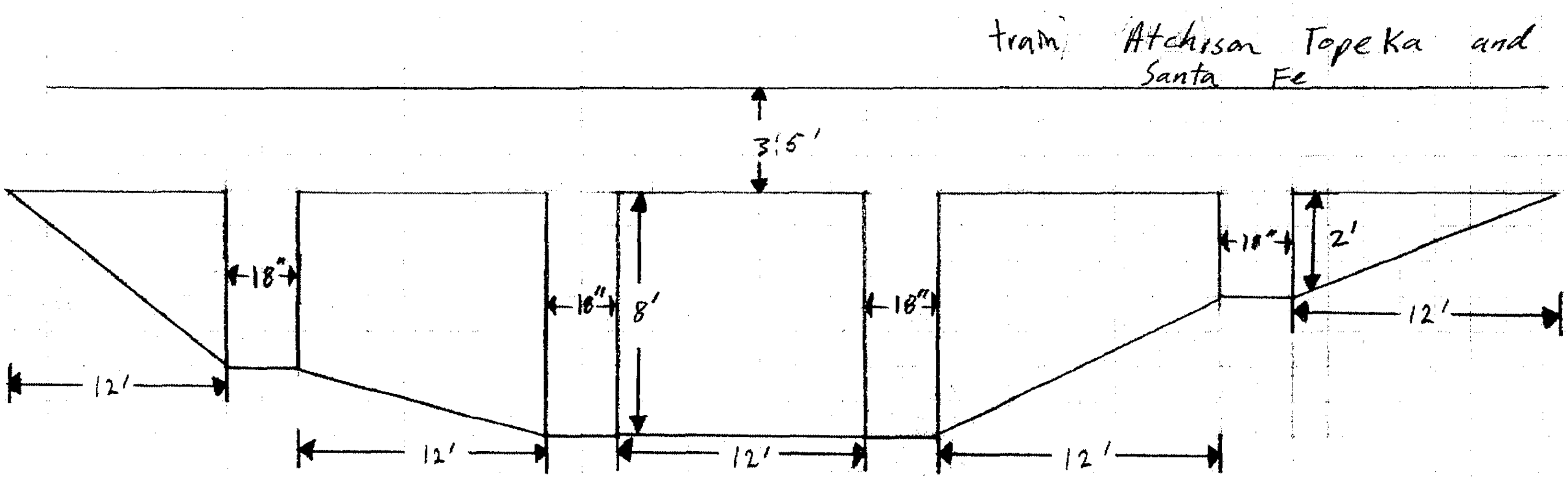
PICS 89-95

63 sq. ft @ 6 ft/s = 380 cfs





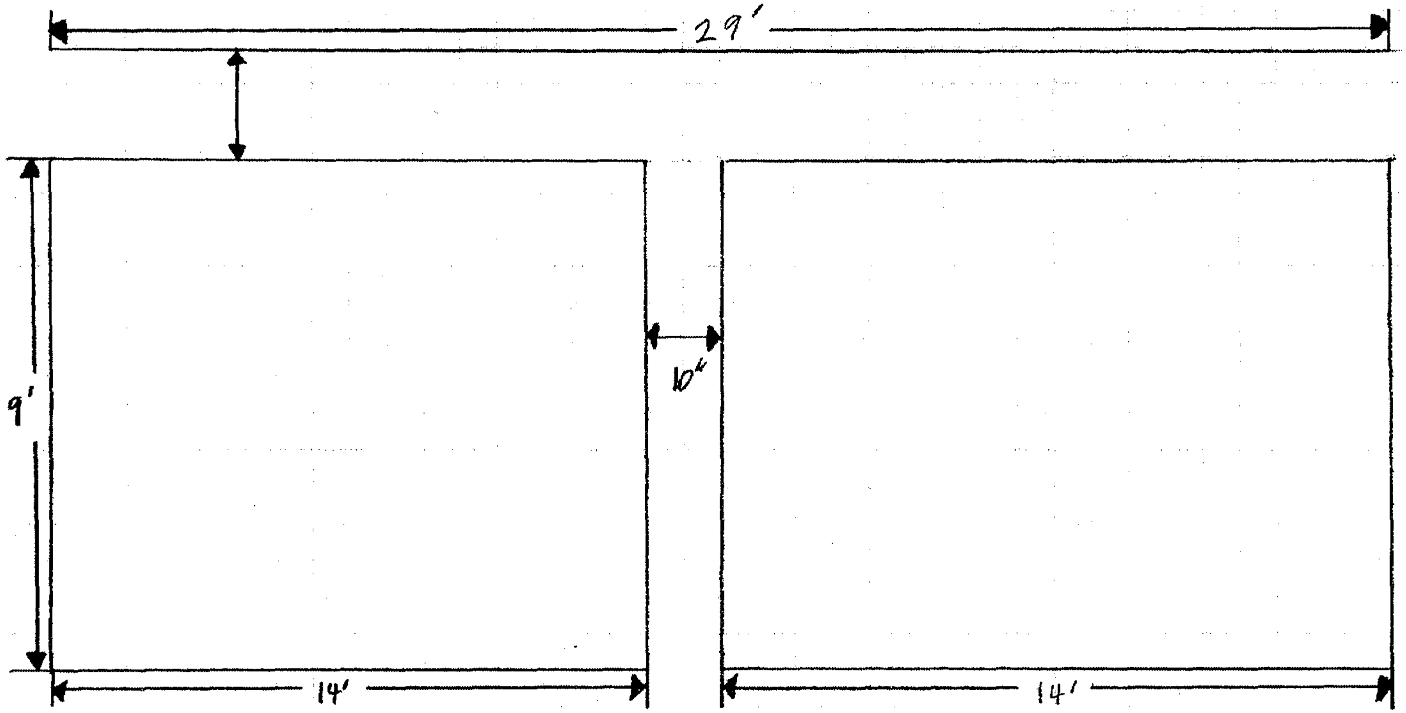
A



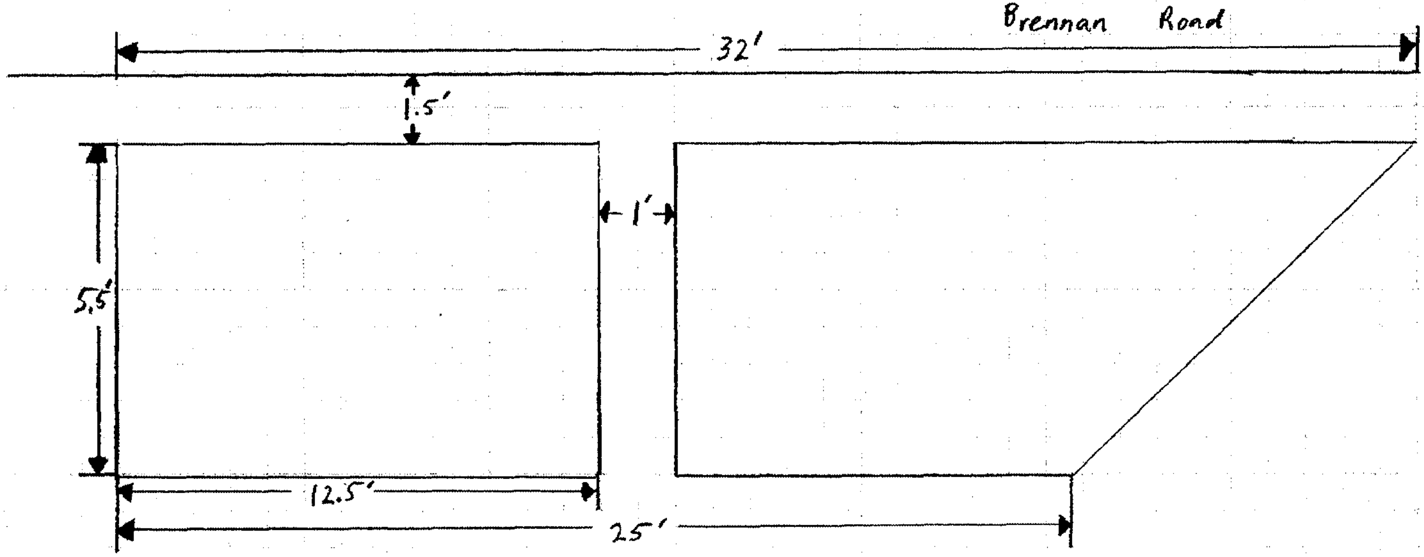
note 2 ft/sec
Adv. 1.25'

Lone Tree Creek and Atchison Topeka / Santa Fe

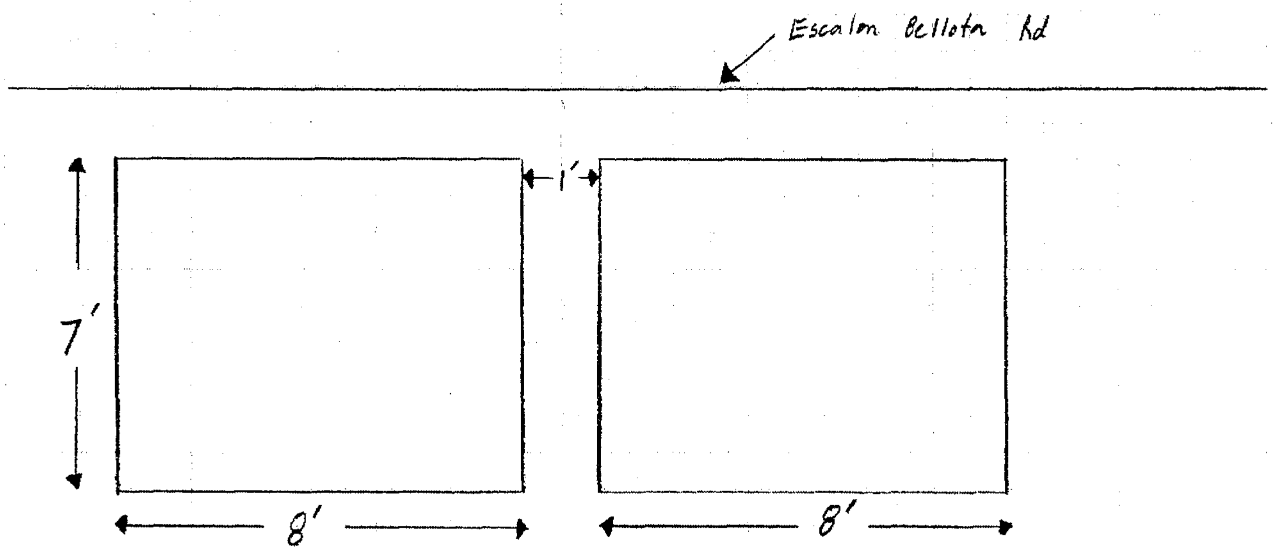
B



Lone Tree Creek and Sexton Rd.

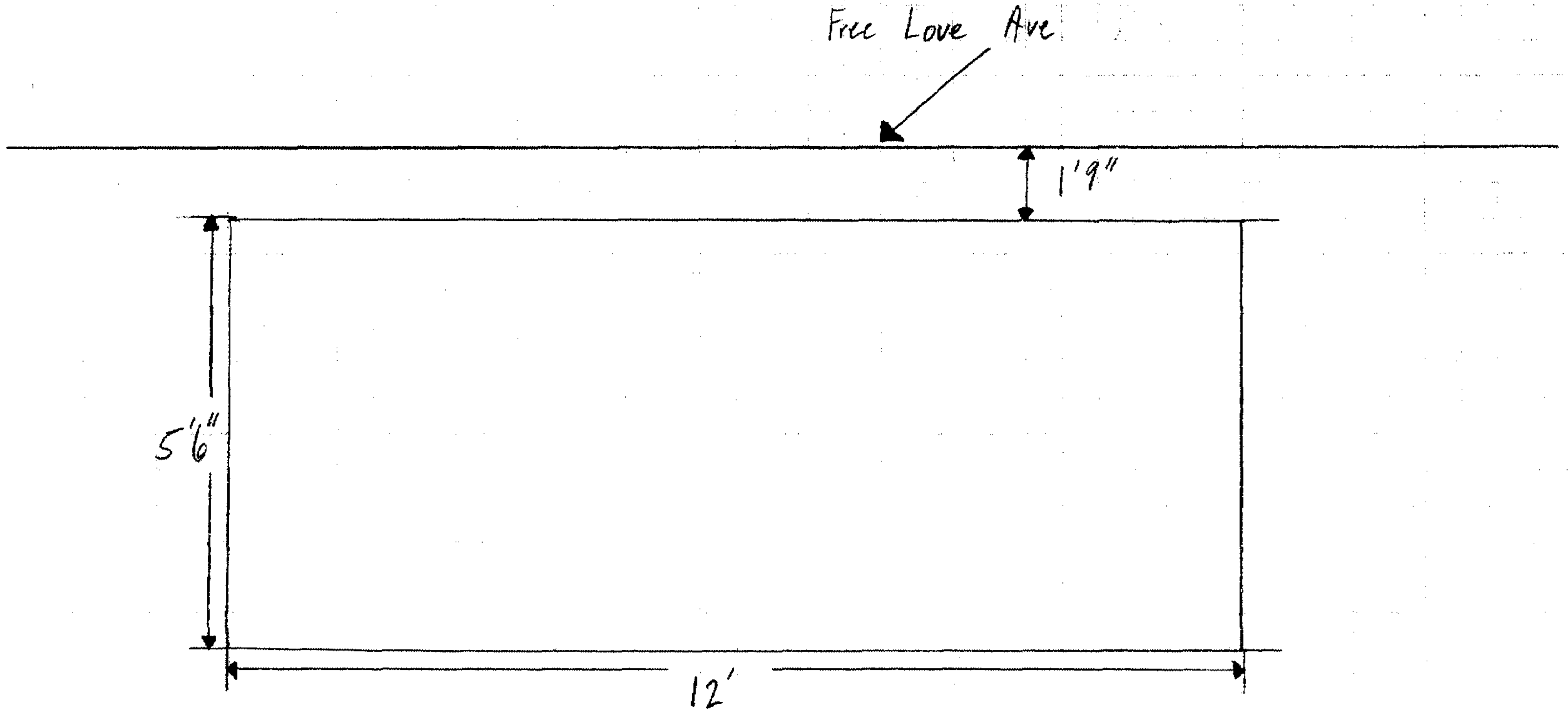


Lone Creek and Brennan Rd.



Lone Creek and Escalon Bellota rd.

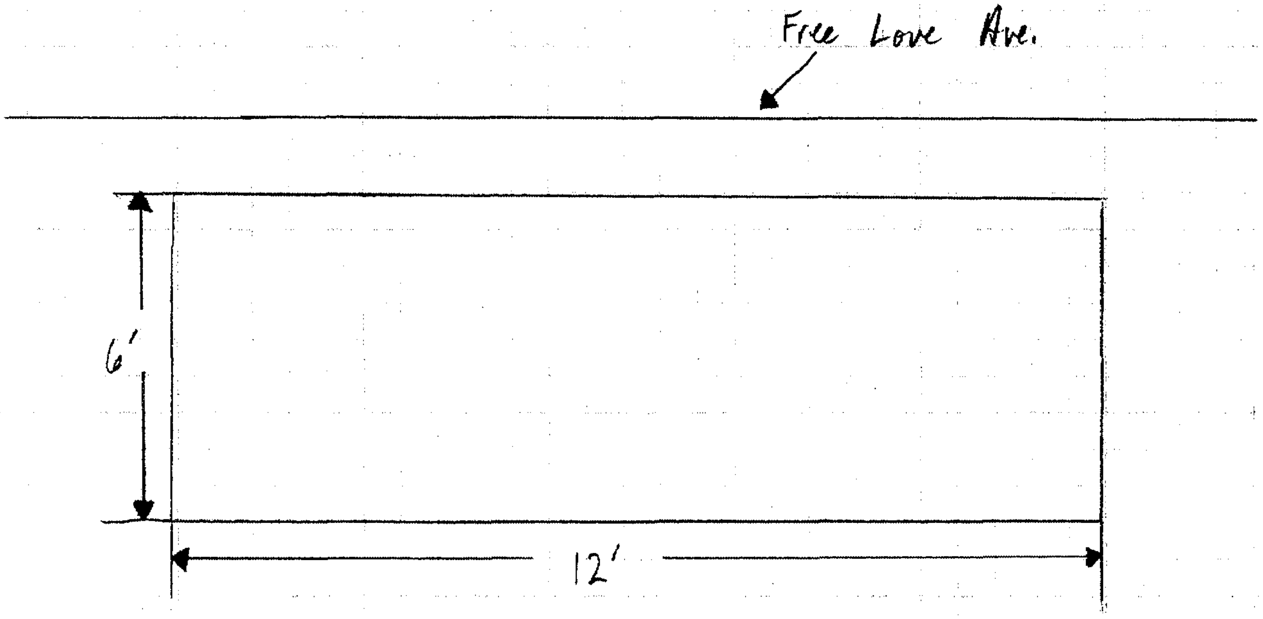
8



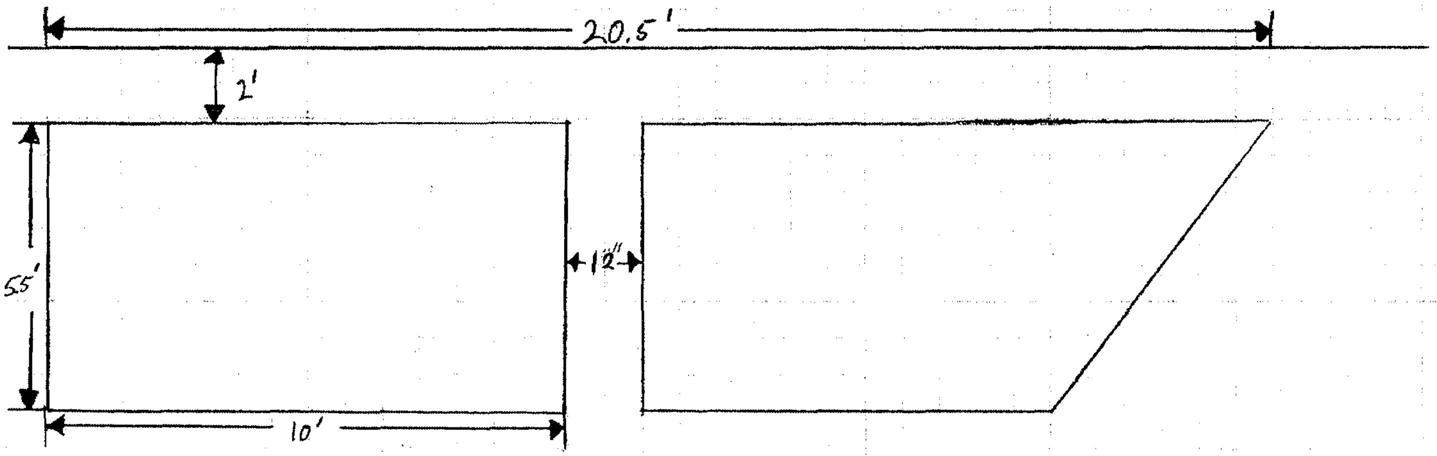
note: Sandy soil

Lone Creek and Free Love Ave

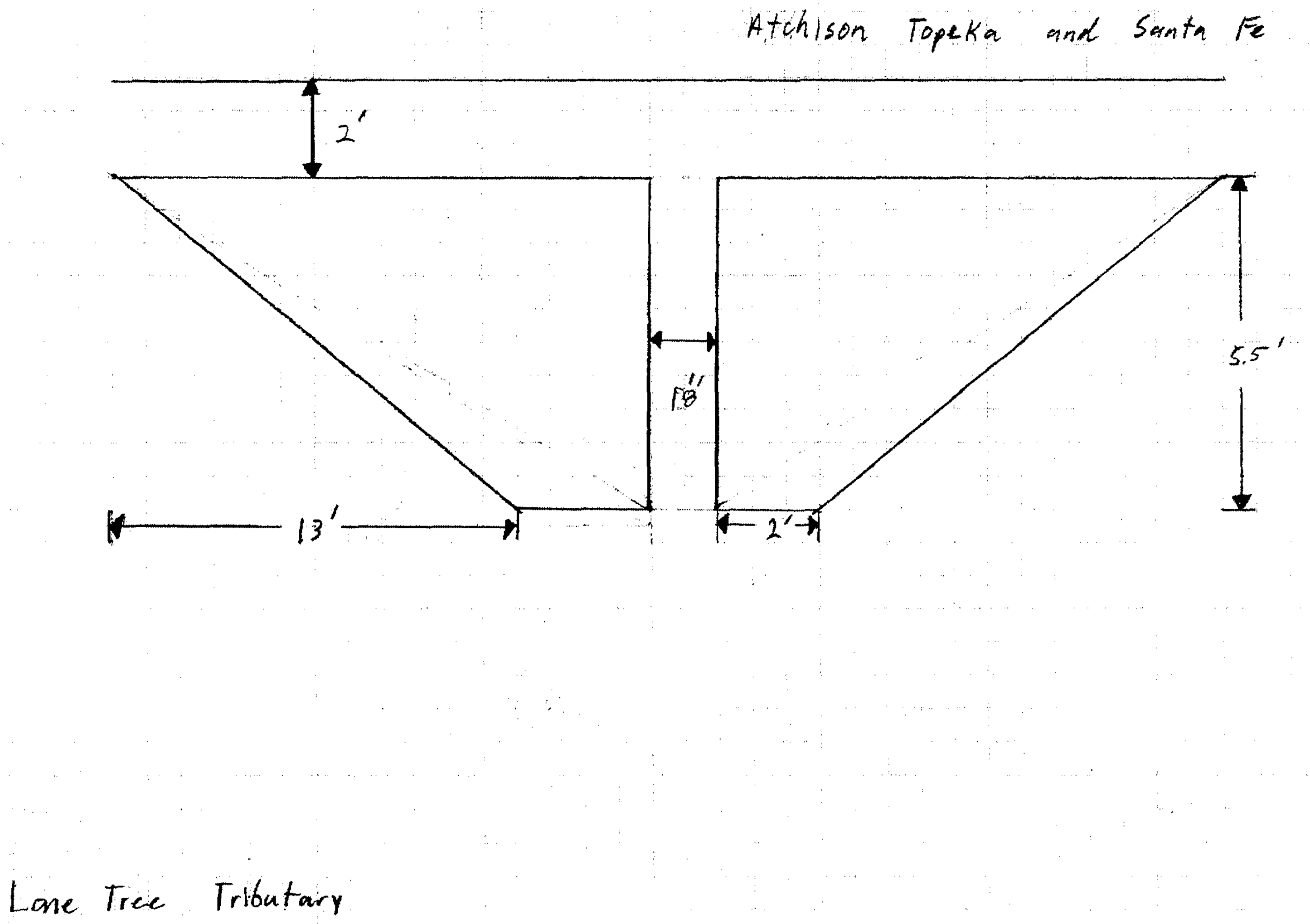
A



Lone Creek and Free Love Ave.



Lone tree Creek and Carrolton rd.


Atchison Topeka and Santa Fe
2'
18"
5.5'
13'
2'
Lone Tree Tributary

TEMPLE CREEK | CROSSING RATINGS

RS 430 JACK TONE RD. DECK = 51.57

Q	ELEV	AREA	AVG DEPTH	STORAGE
300	49.9			
500	50.1			
750	52.1	280	1	280
1000	52.7	355	1.4	500
1250	53.3	450	1.7	770

50.1 — WEIR

52.1–53.3: Lower – model is constrained

AT&SF RR

Q	ELEV	AREA	AVG DEPTH	STORAGE
750	63.2			
1000	64.0			
1250	64.3			
1500	65.9			
1750	67.5			

1000–1750 → EXCESS FLOWS TO SFSLJ

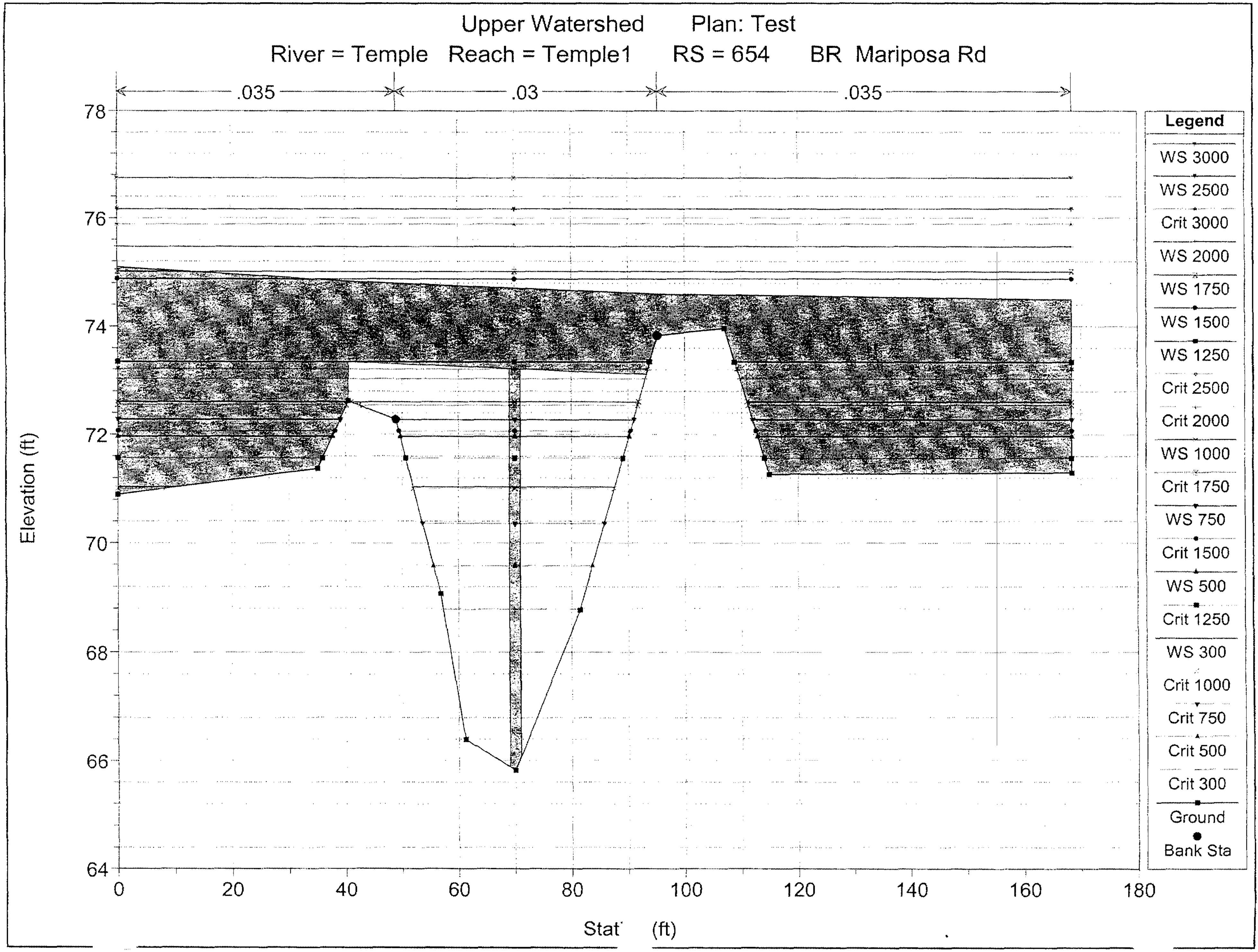
MURPHY RD

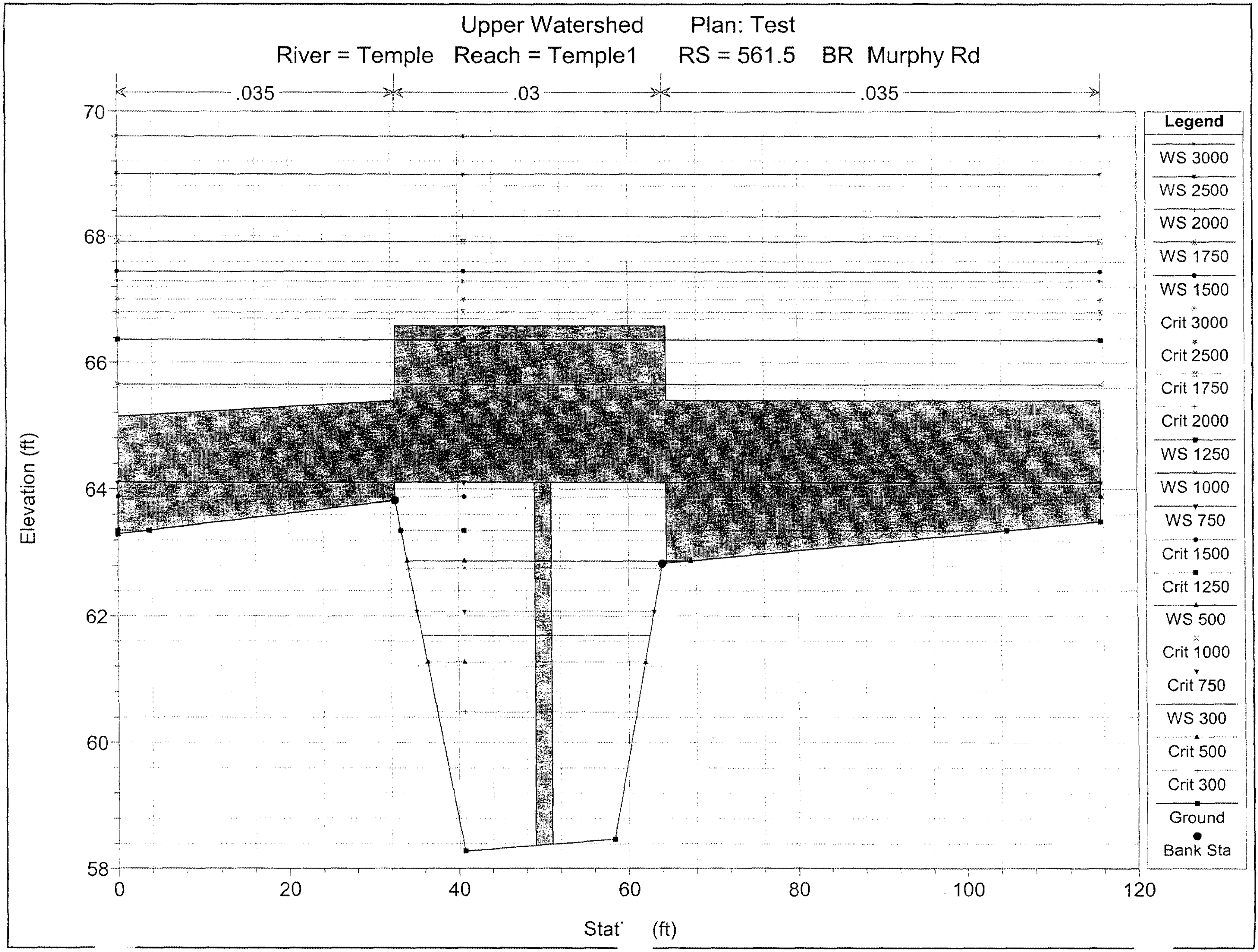
Q	ELEV	AREA	AVG DEPTH	STORAGE
500	62.9			
750	64.1	40	1	40
1000	65.45	205	1.5	310
1250	66.4	410	2	820

MARIPOSA RD

Q	EL	AREA	D	STOR.
1250	72.8			
1500	74.8			
1750	75			
2000	75.3			
2500	75.7			

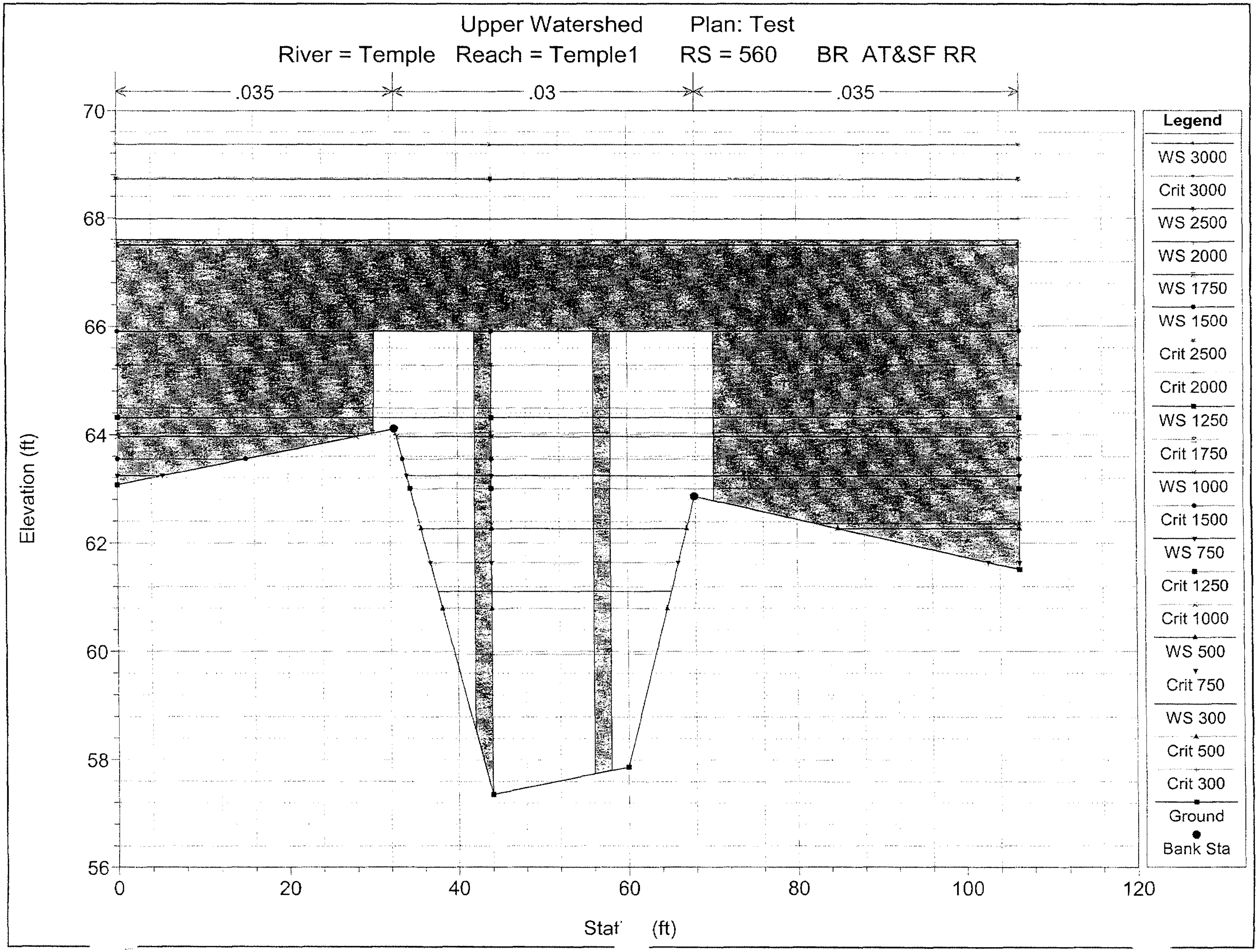
1500–2500 → FLOWS TO SFSLJ

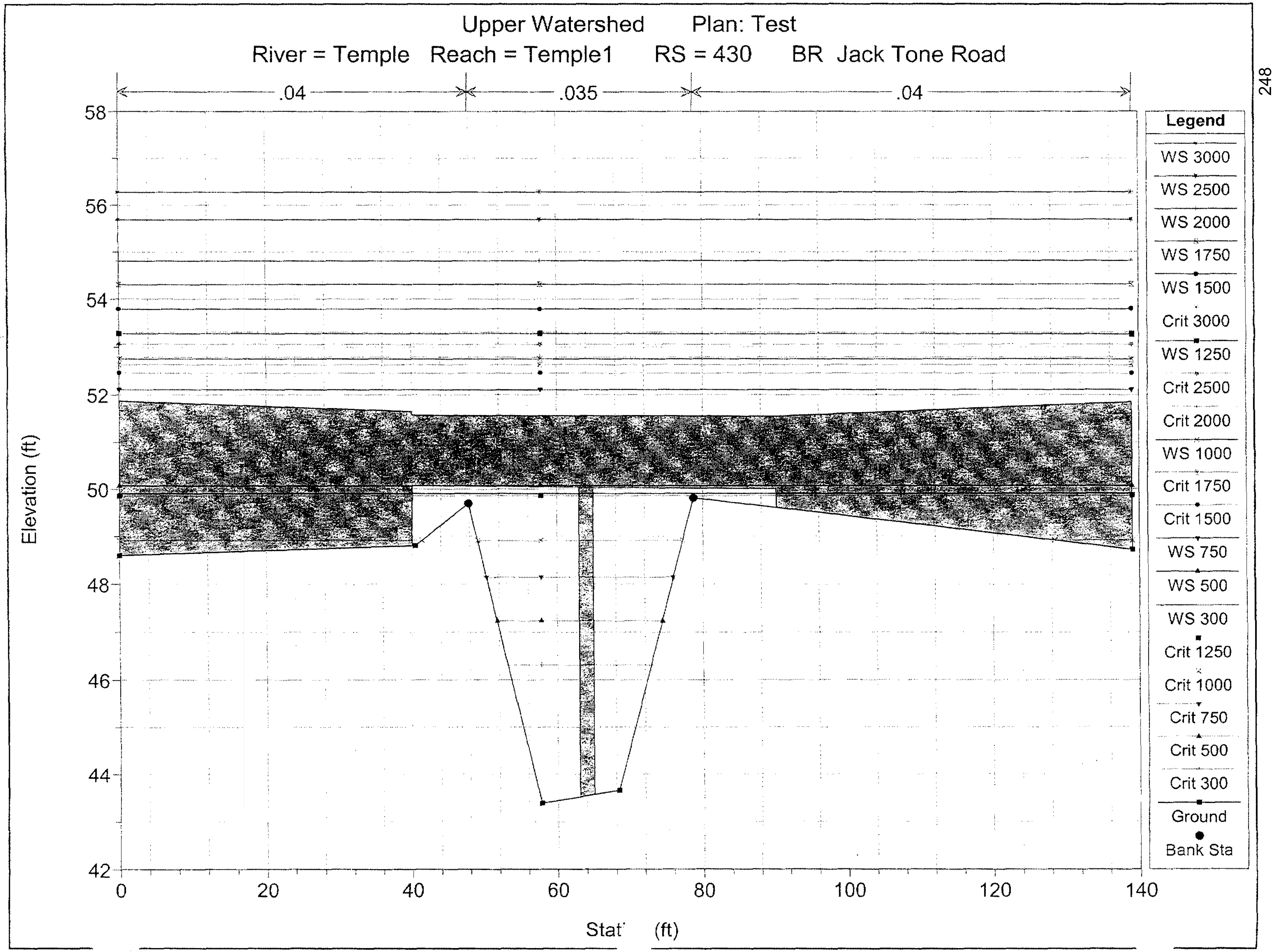

Upper Watershed Plan: Test
River = Temple Reach = Temple1 RS = 654 BR Mariposa Rd
.035
.03
.035
Elevation (ft)
78
76
74
72
70
68
66
64
0
20
40
60
80
100
120
140
160
180
Stat (ft)
Legend
WS 3000
WS 2500
Crit 3000
WS 2000
WS 1750
WS 1500
WS 1250
Crit 2500
Crit 2000
WS 1000
Crit 1750
WS 750
Crit 1500
WS 500
Crit 1250
WS 300
Crit 1000
Crit 750
Crit 500
Crit 300
Ground
Bank Sta

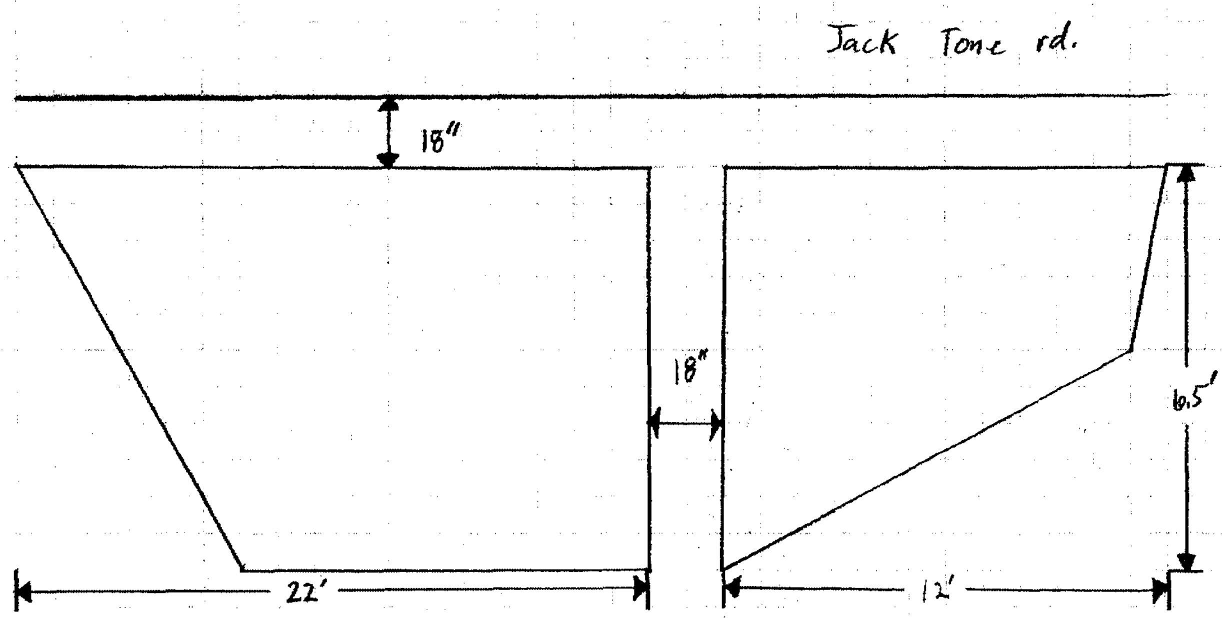


Upper Watershed Plan: Test

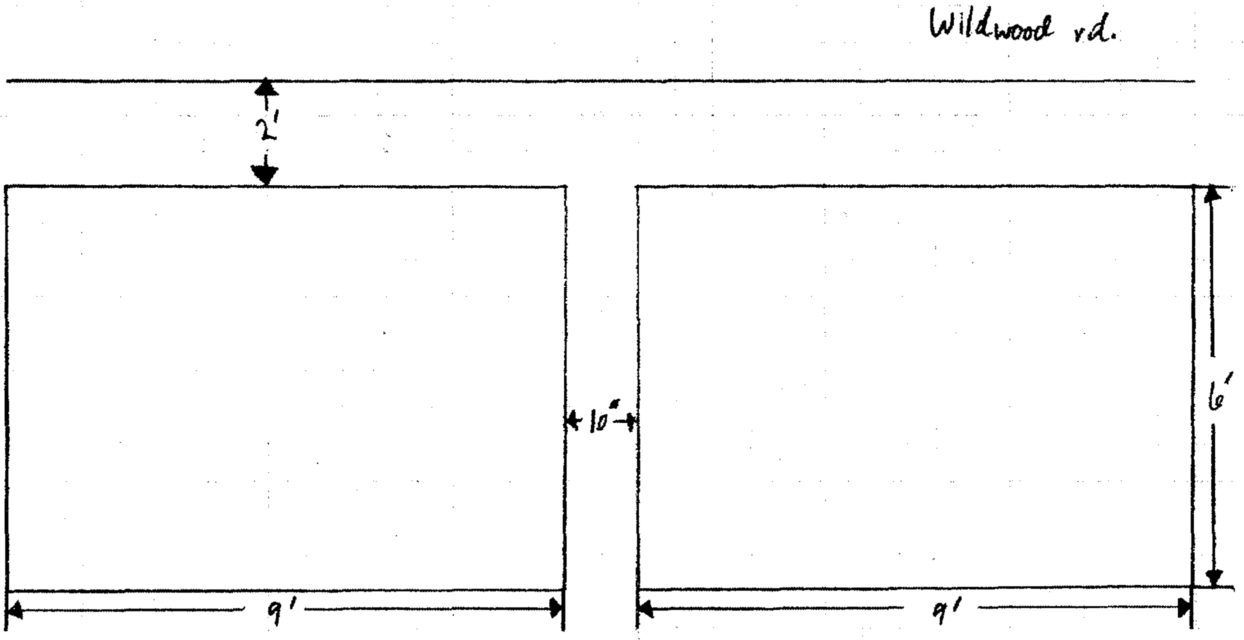
River = Temple Reach = Temple1 RS = 561.5 BR Murphy Rd


Upper Watershed Plan: Test
River = Temple Reach = Temple1 RS = 560 BR AT&SF RR
.035
.03
.035
Elevation (ft)
70
68
66
64
62
60
58
56
0
20
40
60
80
100
120
Staf (ft)
Legend
WS 3000
Crit 3000
WS 2500
WS 2000
WS 1750
WS 1500
Crit 2500
Crit 2000
WS 1250
Crit 1750
WS 1000
Crit 1500
WS 750
Crit 1250
Crit 1000
WS 500
Crit 750
WS 300
Crit 500
Crit 300
Ground
Bank Sta


Upper Watershed Plan: Test
River = Temple Reach = Temple1 RS = 430 BR Jack Tone Road
.04
.035
.04
Elevation (ft)
58
56
54
52
50
48
46
44
42
0
20
40
60
80
100
120
140
Stat' (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
Crit 3000
WS 1250
Crit 2500
Crit 2000
WS 1000
Crit 1750
Crit 1500
WS 750
WS 500
WS 300
Crit 1250
Crit 1000
Crit 750
Crit 500
Crit 300
Ground
Bank Sta

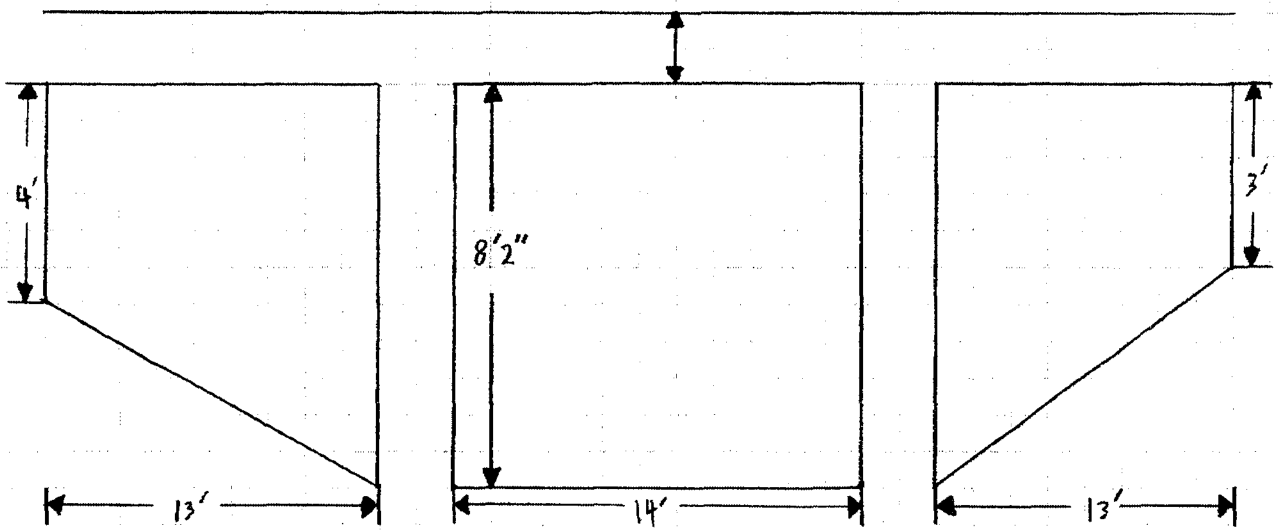


Temple and Jack Tone rd.



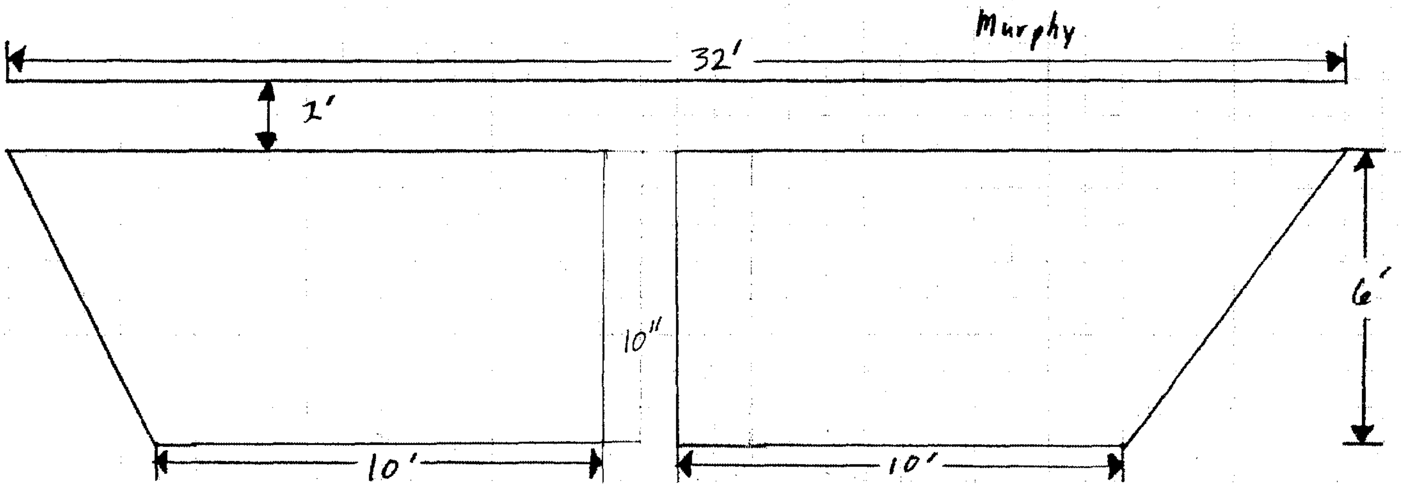
Temple and Wildwood rd.

B

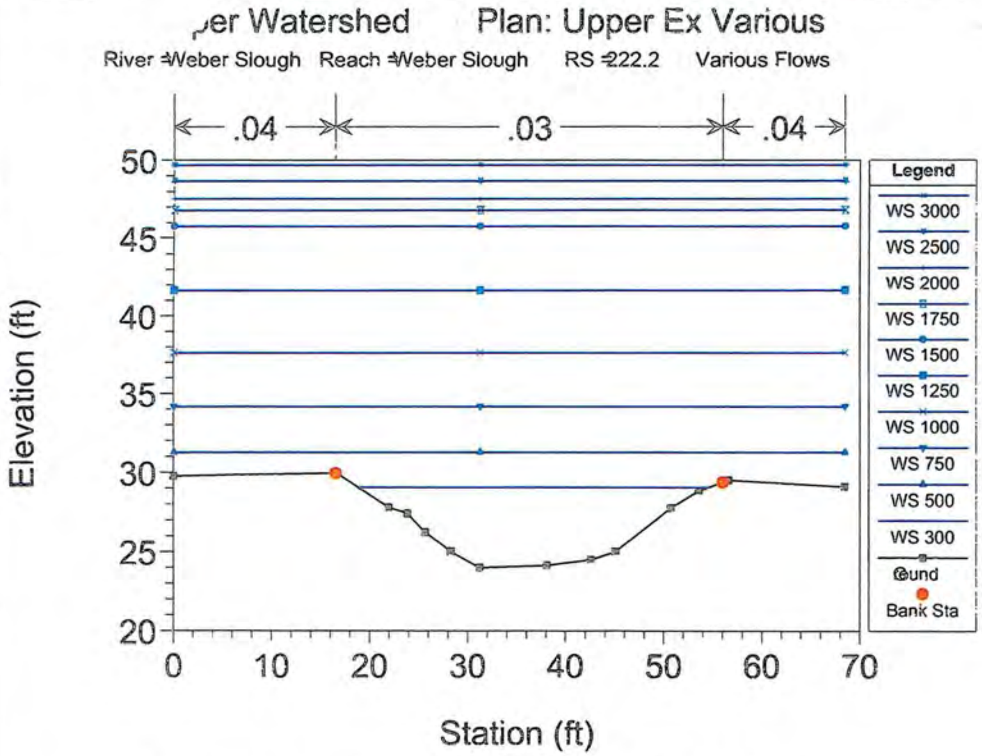


Temple and Atchison Topeka and Santa Fe RR

A

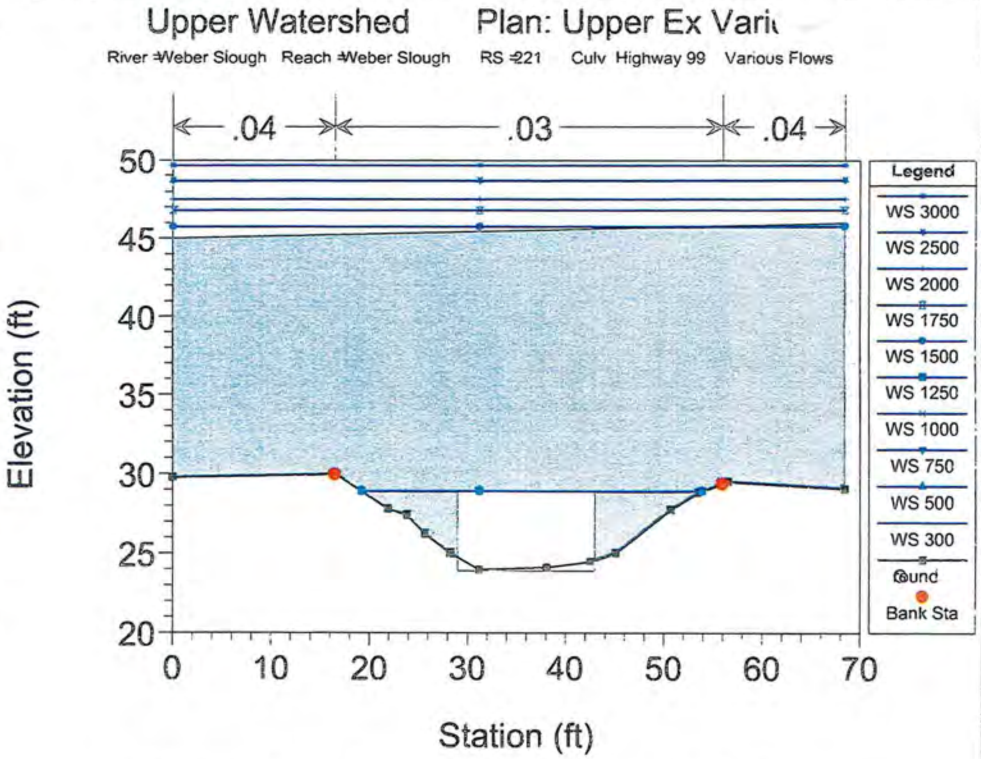


Temple and Murphy rd.



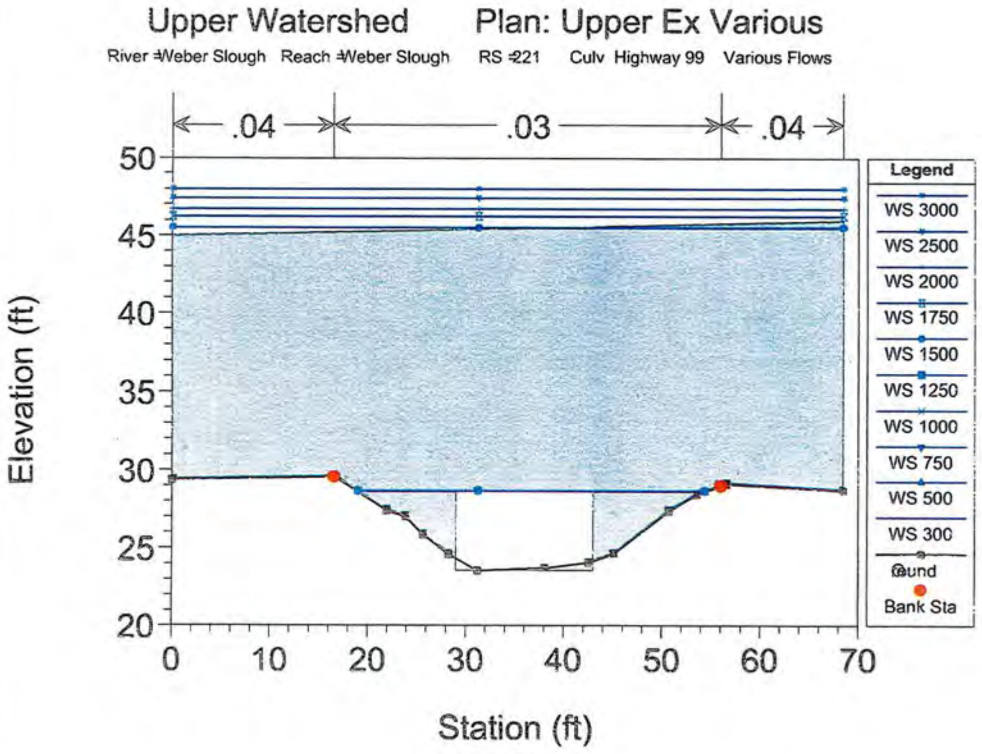
per Watershed Plan: Upper Ex Various

River =Weber Slough Reach =Weber Slough RS =222.2 Various Flows



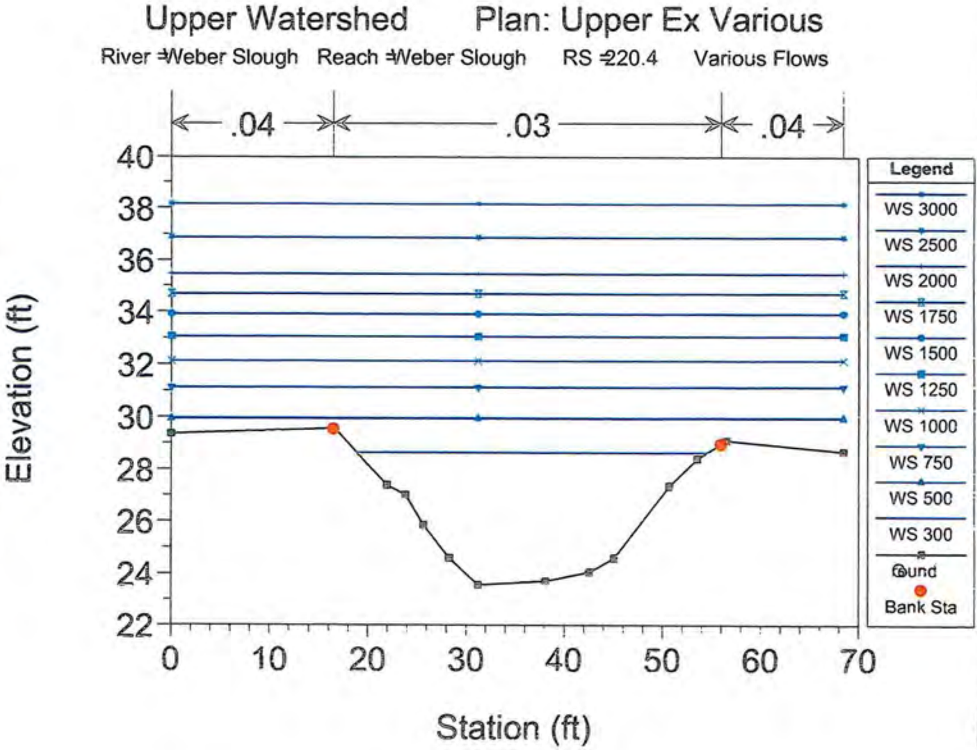
Upper Watershed Plan: Upper Ex Vari

River =Weber Slough Reach =Weber Slough RS =221 Culv Highway 99 Various Flows



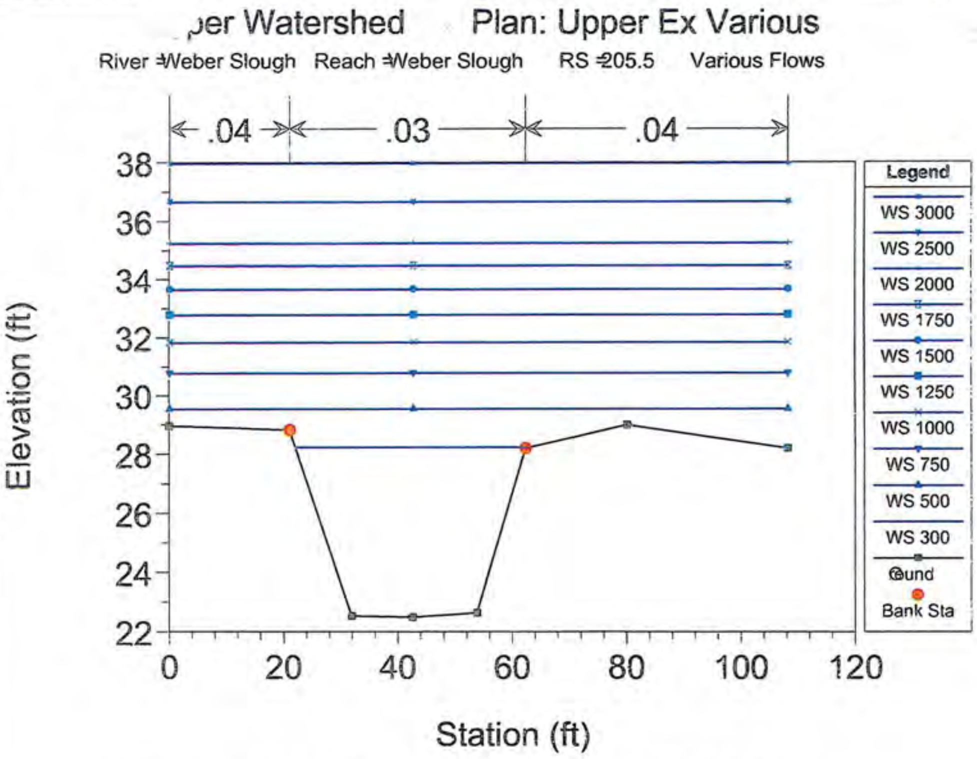
Upper Watershed Plan: Upper Ex Various

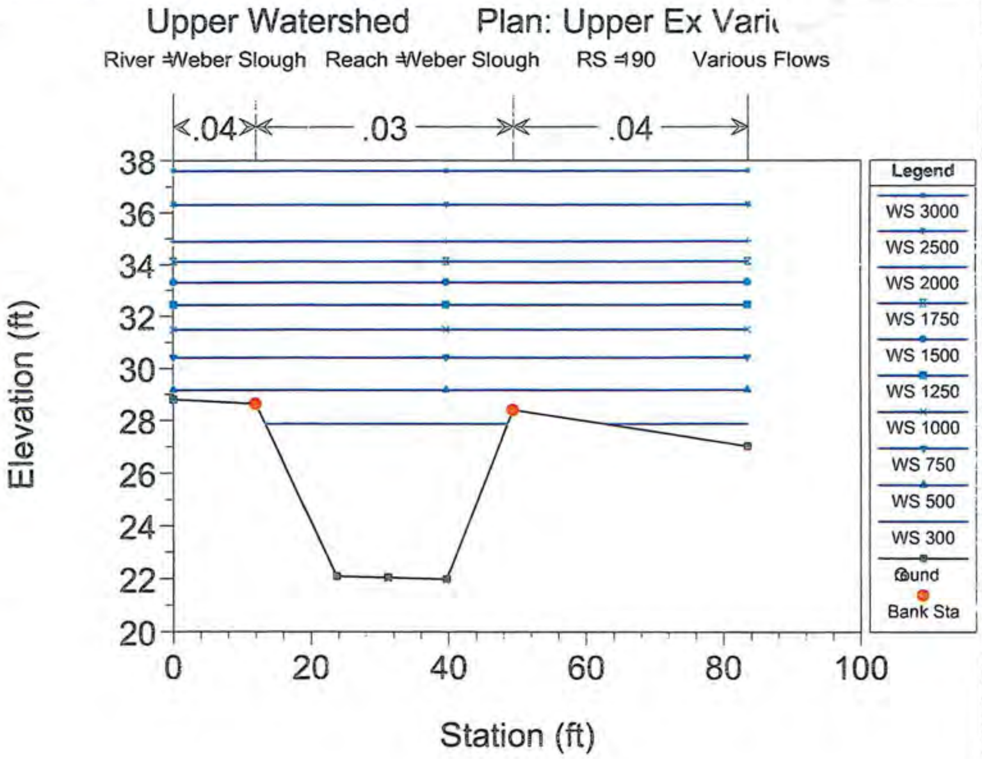
River =Weber Slough Reach =Weber Slough RS =221 Culv Highway 99 Various Flows

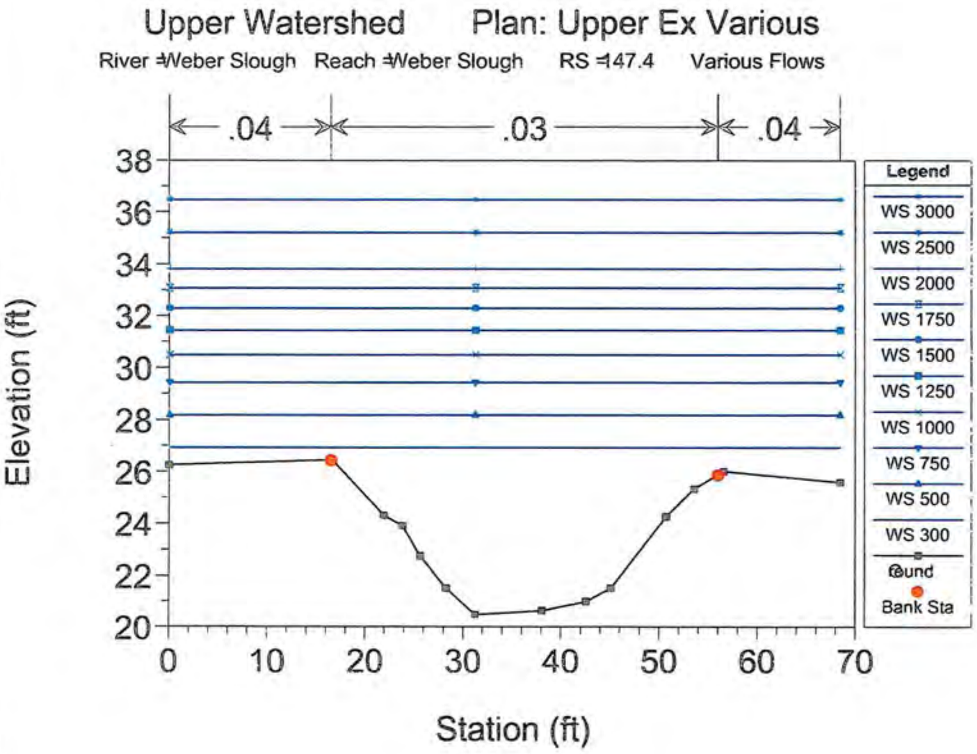


Upper Watershed Plan: Upper Ex Various

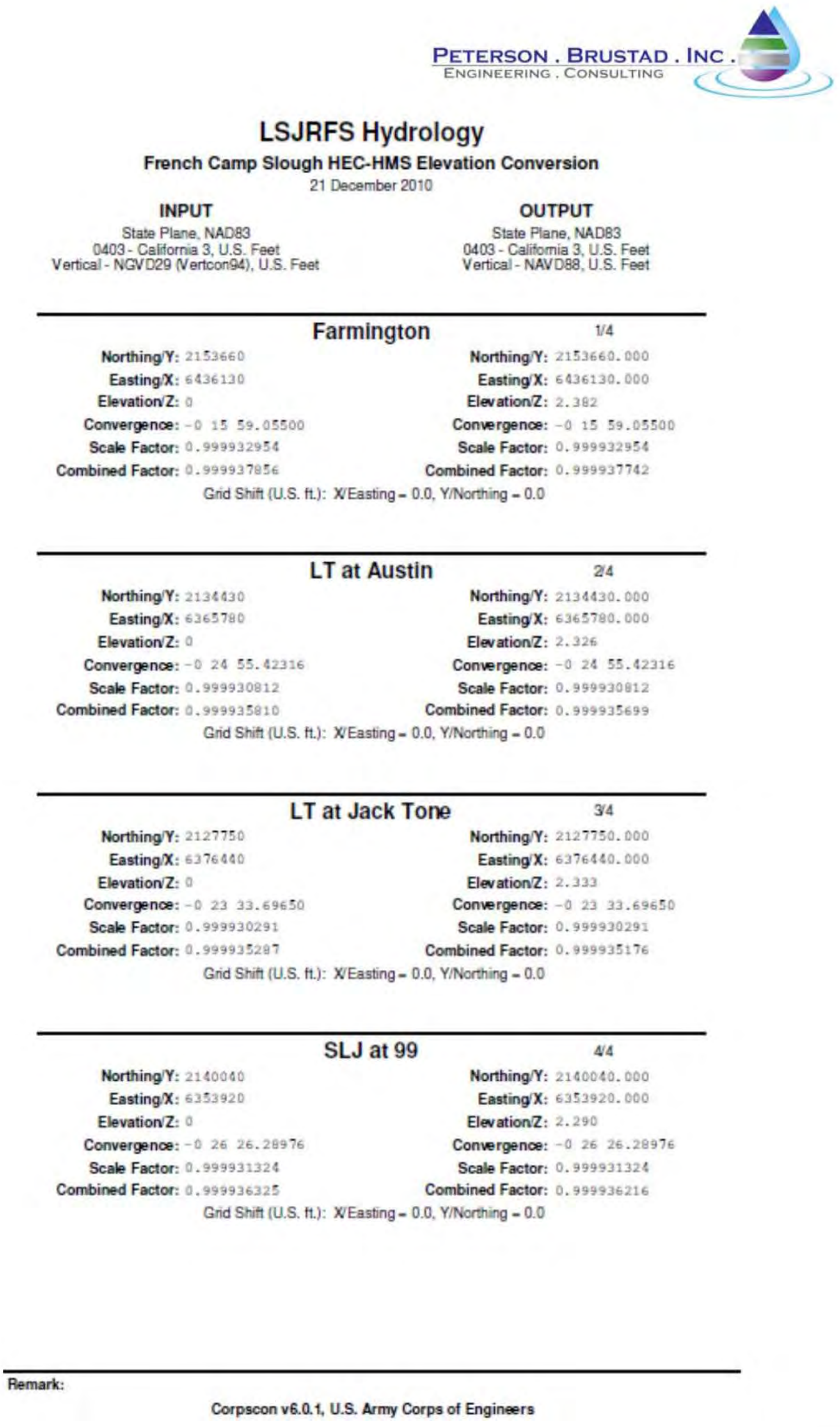
River =Weber Slough Reach =Weber Slough RS =220.4 Various Flows


per Watershed
Plan: Upper Ex Various
River = Weber Slough
Reach = Weber Slough
RS = 205.5
Various Flows
.04
.03
.04
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
WS 500
WS 300
Ground
Bank Sta
Elevation (ft)
38
36
34
32
30
28
26
24
22
0
20
40
60
80
100
120
Station (ft)


Upper Watershed
Plan: Upper Ex Vari
River = Weber Slough
Reach = Weber Slough
RS = 190
Various Flows
.04
.03
.04
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
WS 500
WS 300
Ground
Bank Sta
Elevation (ft)
38
36
34
32
30
28
26
24
22
20
0
20
40
60
80
100
Station (ft)


Upper Watershed
Plan: Upper Ex Various
River = Weber Slough
Reach = Weber Slough
RS = 147.4
Various Flows
.04
.03
.04
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
WS 500
WS 300
Ground
Bank Sta
Elevation (ft)
38
36
34
32
30
28
26
24
22
20
0
10
20
30
40
50
60
70
Station (ft)

Attachment 6- C. Corpscon Vertical Datum Conversion for French Camp Slough Model Elements



PETERSON . BRUSTAD . INC .
ENGINEERING . CONSULTING

LSJRFS Hydrology

French Camp Slough HEC-HMS Elevation Conversion

21 December 2010

INPUT	OUTPUT
State Plane, NAD83 0403 - California 3, U.S. Feet Vertical - NGVD29 (Vertcon94), U.S. Feet	State Plane, NAD83 0403 - California 3, U.S. Feet Vertical - NAVD88, U.S. Feet

Farmington 1/4

Northing/Y:	2153660	**Northing/Y:**	2153660.000
Easting/X:	6436130	**Easting/X:**	6436130.000
Elevation/Z:	0	**Elevation/Z:**	2.382
Convergence:	-0 15 59.05500	**Convergence:**	-0 15 59.05500
Scale Factor:	0.999932954	**Scale Factor:**	0.999932954
Combined Factor:	0.999937856	**Combined Factor:**	0.999937742

Grid Shift (U.S. ft.): X/Easting = 0.0, Y/Northing = 0.0

LT at Austin 2/4

Northing/Y:	2134430	**Northing/Y:**	2134430.000
Easting/X:	6365780	**Easting/X:**	6365780.000
Elevation/Z:	0	**Elevation/Z:**	2.326
Convergence:	-0 24 55.42316	**Convergence:**	-0 24 55.42316
Scale Factor:	0.999930812	**Scale Factor:**	0.999930812
Combined Factor:	0.999935810	**Combined Factor:**	0.999935699

Grid Shift (U.S. ft.): X/Easting = 0.0, Y/Northing = 0.0

LT at Jack Tone 3/4

Northing/Y:	2127750	**Northing/Y:**	2127750.000
Easting/X:	6376440	**Easting/X:**	6376440.000
Elevation/Z:	0	**Elevation/Z:**	2.333
Convergence:	-0 23 33.69650	**Convergence:**	-0 23 33.69650
Scale Factor:	0.999930291	**Scale Factor:**	0.999930291
Combined Factor:	0.999935287	**Combined Factor:**	0.999935176

Grid Shift (U.S. ft.): X/Easting = 0.0, Y/Northing = 0.0

SLJ at 99 4/4

Northing/Y:	2140040	**Northing/Y:**	2140040.000
Easting/X:	6353920	**Easting/X:**	6353920.000
Elevation/Z:	0	**Elevation/Z:**	2.290
Convergence:	-0 26 26.28976	**Convergence:**	-0 26 26.28976
Scale Factor:	0.999931324	**Scale Factor:**	0.999931324
Combined Factor:	0.999936325	**Combined Factor:**	0.999936216

Grid Shift (U.S. ft.): X/Easting = 0.0, Y/Northing = 0.0

Remark:

Corpscon v6.0.1, U.S. Army Corps of Engineers

Attachment 6- D. French Camp Slough Subbasin Characteristics – Existing Conditions

Subbasin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
ARCH	0.74	0.02	1.60	40	30	0.68	6.25	0.35	VD	1.5	0.021	60	10.8
BACH	43.07	0.04	14.79	1090	150	9.66	63.55	2.87	FH	1.5	0.030	0	
CLAY	0.10	0.02	0.59	12	10	0.34	3.40	0.21	VD	1.5	0.021	40	9.8
DC1	5.60	0.2	6.74	169	80	3.40	13.21	9.66	VU	1.5	0.051	2	
DC10	0.04	0.02	0.47	15	10	0.47	10.62	0.17	VD	1.5	0.021	50	
DC11	0.23	0.02	1.21	15	10	0.55	4.14	0.31	VD	1.5	0.021	40	
DC2	5.91	0.2	7.81	90	46	4.46	5.63	13.32	VU	1.5	0.023	2	
DC3	4.04	0.2	4.40	75	52	2.11	5.23	8.17	VU	1.5	0.024	2	
DC4	3.80	0.2	3.17	53	32	1.31	6.63	5.75	VU	1.5	0.022	2	
DC5	1.68	0.2	2.80	52	40	1.40	4.28	6.12	VU	1.5	0.021	2	
DC6	0.92	0.025	2.34	40	27	0.97	5.55	0.59	VD	1.5	0.021	40	
DC7	0.32	0.02	1.04	30	23	0.49	6.74	0.26	VD	1.5	0.021	50	
DC8	0.62	0.1	1.45	30	20	0.83	6.88	1.78	VU	1.5	0.021	10	
DC9	0.17	0.02	0.62	10	9	0.32	1.60	0.24	VD	1.5	0.021	40	
DCAP	1.79	0.015	2.86	30	10	1.28	6.99	0.41	VD	1.5	0.021	60	114.8
DUCK	28.28	0.04	16.44	315	96	10.04	13.32	4.09	FH	1.5	0.031	0	
FARM	32.48	0.03	12.32	260	150	5.68	8.93	2.39	FH	1.5	0.036	0	
FCS1	1.70	0.1	2.29	27	18	1.04	3.93	2.58	VU	1.5	0.047	5	
FCS2	0.46	0.15	1.27	22	15	0.59	5.50	2.33	VU	1.5	0.086	5	
FCS3	0.26	0.15	1.38	15	11	0.68	2.90	2.87	VU	1.5	0.059	5	
FCS4	0.20	0.025	1.04	15	11	0.52	3.86	0.37	VD	1.5	0.021	40	
FCS5	0.30	0.025	1.15	20	15	0.56	4.36	0.38	VD	1.5	0.021	40	
FCS6	0.38	0.15	1.14	15	10	0.74	4.38	2.55	VU	1.5	0.021	5	
FCS7	0.12	0.15	0.71	11	5	0.41	8.45	1.50	VU	1.5	0.021	5	
GRUPE	0.20	0.015	1.17	15	10	0.73	4.28	0.26	VD	1.5	0.032	60	120.7
GTWY	0.77	0.02	1.42	21	15	0.73	4.22	0.37	VD	1.5	0.021	50	22.3
LJ1	6.30	0.2	4.81	239	96	2.29	29.73	6.27	VU	1.5	0.067	2	
LJ2	1.40	0.2	3.31	100	78	1.80	6.64	6.60	VU	1.5	0.095	2	
LT A1	2.32	0.2	3.40	208	150	1.70	17.06	5.46	VU	1.5	0.021	2	

Subbasin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
LT A2	3.80	0.2	4.40	214	150	2.21	14.53	6.85	VU	1.5	0.021	2	
LT A3	0.15	0.1	0.70	150	147	0.36	4.28	1.08	VU	1.5	0.111	5	
LT B1	3.20	0.2	4.06	229	157	2.04	17.72	6.20	VU	1.5	0.046	2	
LT B2	3.26	0.2	2.66	155	130	1.42	9.40	5.20	VU	1.5	0.052	2	
LT B3	4.07	0.2	4.19	152	115	1.82	8.84	6.86	VU	1.5	0.035	2	
LT B4	2.75	0.2	4.30	156	115	1.99	9.54	7.07	VU	1.5	0.144	2	
LT B5	2.05	0.2	2.77	115	90	1.36	9.04	5.23	VU	1.5	0.046	2	
LT C1	2.47	0.2	3.48	106	75	1.44	8.90	5.85	VU	1.5	0.080	2	
LT C2	2.66	0.2	2.99	85	60	1.44	8.35	5.59	VU	1.5	0.052	2	
LT C3	0.97	0.2	1.14	50	43	0.83	6.16	3.33	VU	1.5	0.045	2	
LT C4a	1.68	0.2	1.69	48	39	0.76	5.04	3.87	VU	1.5	0.042	2	
LT C4b	1.19	0.2	2.01	40	30	0.97	4.98	4.55	VU	1.5	0.063	2	
LT D1	3.51	0.2	3.71	115	79	1.74	9.70	6.34	VU	1.5	0.027	2	
LT D2	2.25	0.2	4.73	76	50	2.12	5.49	8.34	VU	1.5	0.035	2	
LT E1	8.62	0.2	8.66	115	50	3.48	7.51	11.94	VU	1.5	0.142	2	
LT F1	1.26	0.2	2.24	90	69	1.06	9.36	4.36	VU	1.5	0.021	2	
LT F2	3.28	0.2	4.32	69	44	2.31	5.79	8.24	VU	1.5	0.021	2	
LT G1	0.45	0.2	1.02	31	24	0.44	6.84	2.45	VU	1.5	0.060	2	
NFSLJ1	1.07	0.1	1.99	65	45	1.00	10.06	2.01	VU	1.5	0.021	2	
NFSLJ2	6.78	0.1	5.00	56	18	2.67	7.60	4.37	VU	1.5	0.021	2	
NLJ1	3.33	0.2	4.58	97	76	2.13	4.59	8.55	VU	1.5	0.046	2	
NLJ2	3.22	0.2	4.53	77	50	2.18	5.96	8.17	VU	1.5	0.028	2	
NLJ3	0.75	0.2	1.58	51	40	0.69	6.95	3.44	VU	1.5	0.021	2	
NLJ4	1.19	0.15	2.43	45	30	1.02	6.18	3.60	VU	1.5	0.021	5	
ROCK1	6.19	0.04	4.46	1250	560	2.19	154.63	0.88	FH	1.5	0.025	0	
Rock2	16.01	0.04	7.09	1400	210	3.41	167.78	1.22	FH	1.5	0.023	0	
Rock3	11.88	0.04	10.29	210	150	3.03	5.83	2.54	FH	1.5	0.037	0	
SABC	1.80	0.02	2.40	32	25	0.97	2.92	0.54	VD	1.5	0.021	50	66.8

Subbasin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
SALT1	2.23	0.04	2.75	2000	1175	1.52	300.41	0.56	FH	1.5	0.022	0	
SALT2	2.39	0.04	2.08	2140	1200	0.85	451.20	0.37	FH	1.5	0.021	0	
SALT3	15.06	0.03	3.84	1240	1075	1.52	42.92	0.69	FH	1.5	0.045	0	
SFSLJ1	0.75	0.2	1.55	58	44	0.76	9.01	3.36	VU	1.5	0.021	2	
SFSLJ2	3.30	0.2	4.70	52	19	2.35	7.03	8.25	VU	1.5	0.021	2	
SFSLJ A1	5.93	0.2	8.66	110	44	4.92	7.63	13.58	VU	1.5	0.044	2	
SLJ	4.45	0.1	6.95	78	49	3.66	4.17	6.25	VU	1.5	0.028	5	
STAGE	0.50	0.015	0.75	30	28	0.29	2.68	0.17	VD	1.5	0.021	60	155.4
TE A1	3.49	0.1	2.25	166	128	1.04	16.86	1.94	VU	1.5	0.021	2	
TE B1	2.72	0.1	4.51	162	105	2.58	12.65	3.76	VU	1.5	0.021	2	
TE B2	2.27	0.1	3.83	128	105	2.09	6.01	3.76	VU	1.5	0.021	2	
TE B3	2.09	0.1	2.31	84	67	1.29	7.36	2.49	VU	1.5	0.021	2	
TE B4	2.83	0.2	4.89	101	59	1.59	8.60	6.95	VU	1.5	0.023	2	
TE C1	2.66	0.2	4.39	71	37	1.80	7.74	7.14	VU	1.5	0.022	2	
TE D1	4.53	0.1	3.18	200	147	1.99	16.66	2.84	VU	1.5	0.027	2	
TE D2	5.34	0.15	8.90	147	99	4.64	5.39	10.75	VU	1.5	0.033	2	
TE D3	2.90	0.15	4.68	107	69	2.50	8.12	6.16	VU	1.5	0.025	2	
TE E1	4.18	0.1	4.77	185	107	2.48	16.34	3.61	VU	1.5	0.021	2	
TE F1	5.13	0.15	6.69	165	85	3.45	11.97	7.40	VU	1.5	0.044	2	
TE F2	3.47	0.15	4.98	128	78	2.65	10.04	6.19	VU	1.5	0.021	2	
TURN	2.33	0.015	2.15	15	8	0.93	3.25	0.37	VD	1.5	0.022	60	116.9
UPLJ1	35.10	0.04	19.98	1860	550	10.61	65.57	3.32	FH	1.5	0.023	0	
UPLJ2	51.62	0.04	20.00	1400	150	11.36	62.49	3.44	FH	1.5	0.030	0	
Web1a	3.72	0.1	4.55	75	45	2.08	6.60	3.94	VU	1.5	0.021	5	
Web1b	1.11	0.1	3.28	45	25	1.70	6.10	3.27	VU	1.5	0.021	5	
Web2a	1.42	0.1	2.27	25	20	1.00	2.20	2.83	VU	1.5	0.021	5	
Web2b	0.89	0.04	1.89	22	12	0.95	5.28	0.87	VD	1.5	0.021	50	50.0
WPIP	0.93	0.02	1.38	19	10	0.40	6.50	0.27	VD	1.5	0.021	50	60.7

Attachment 6- E. French Camp Slough Subbasin Characteristics – Future Conditions

Subbasin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
ARCH	0.74	0.02	1.60	40	30	0.68	6.25	0.35	VD	1.5	0.021	60	10.8
BACH	43.07	0.04	14.79	1090	150	9.66	63.55	2.87	FH	1.5	0.030	0	
CLAY	0.10	0.02	0.59	12	10	0.34	3.40	0.21	VD	1.5	0.021	40	9.8
DC1	5.60	0.2	6.74	169	80	3.40	13.21	9.66	VU	1.5	0.051	2	
DC10	0.04	0.02	0.47	15	10	0.47	10.62	0.17	VD	1.5	0.021	50	9.5
DC11	0.23	0.02	1.21	15	10	0.55	4.14	0.31	VD	1.5	0.021	40	54.5
DC2	5.91	0.2	7.81	90	46	4.46	5.63	13.32	VU	1.5	0.023	2	
DC3	4.04	0.2	4.40	75	52	2.11	5.23	8.17	VU	1.5	0.024	2	
DC4	3.80	0.015	3.17	53	32	1.31	6.63	0.43	VD	1.5	0.022	60	899.8
DC5	1.68	0.015	2.80	52	40	1.40	4.28	0.46	VD	1.5	0.021	60	397.8
DC6	0.92	0.025	2.34	40	27	0.97	5.55	0.59	VD	1.5	0.021	40	217.9
DC7	0.32	0.02	1.04	30	23	0.49	6.74	0.26	VD	1.5	0.021	50	75.8
DC8	0.62	0.015	1.45	30	20	0.83	6.88	0.27	VD	1.5	0.021	60	146.8
DC9	0.17	0.02	0.62	10	9	0.32	1.60	0.24	VD	1.5	0.021	40	40.3
DCAP	1.79	0.015	2.86	30	10	1.28	6.99	0.41	VD	1.5	0.021	60	114.8
DUCK	28.28	0.04	16.44	315	96	10.04	13.32	4.09	FH	1.5	0.031	0	
FARM	32.48	0.03	12.32	260	150	5.68	8.93	2.39	FH	1.5	0.036	0	
FCS1	1.70	0.015	2.29	27	18	1.04	3.93	0.39	VD	1.5	0.047	60	402.6
FCS2	0.46	0.015	1.27	22	15	0.59	5.50	0.23	VD	1.5	0.086	60	108.9
FCS3	0.26	0.015	1.38	15	11	0.68	2.90	0.29	VD	1.5	0.059	60	61.6
FCS4	0.20	0.025	1.04	15	11	0.52	3.86	0.37	VD	1.5	0.021	40	47.4
FCS5	0.30	0.025	1.15	20	15	0.56	4.36	0.38	VD	1.5	0.021	40	71.0
FCS6	0.38	0.015	1.14	15	10	0.74	4.38	0.26	VD	1.5	0.021	60	90.0
FCS7	0.12	0.015	0.71	11	5	0.41	8.45	0.15	VD	1.5	0.021	60	28.4
GRUPE	0.20	0.015	1.17	15	10	0.73	4.28	0.26	VD	1.5	0.032	60	120.7
GTWY	0.77	0.02	1.42	21	15	0.73	4.22	0.37	VD	1.5	0.021	50	22.3
LJ1	6.30	0.2	4.81	239	96	2.29	29.73	6.27	VU	1.5	0.067	2	
LJ2	1.40	0.2	3.31	100	78	1.80	6.64	6.60	VU	1.5	0.095	2	
LT A1	2.32	0.2	3.40	208	150	1.70	17.06	5.46	VU	1.5	0.021	2	

Subbasin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
LT A2	3.80	0.2	4.40	214	150	2.21	14.53	6.85	VU	1.5	0.021	2	
LT A3	0.15	0.1	0.70	150	147	0.36	4.28	1.08	VU	1.5	0.111	5	
LT B1	3.20	0.2	4.06	229	157	2.04	17.72	6.20	VU	1.5	0.046	2	
LT B2	3.26	0.2	2.66	155	130	1.42	9.40	5.20	VU	1.5	0.052	2	
LT B3	4.07	0.2	4.19	152	115	1.82	8.84	6.86	VU	1.5	0.035	2	
LT B4	2.75	0.2	4.30	156	115	1.99	9.54	7.07	VU	1.5	0.144	2	
LT B5	2.05	0.2	2.77	115	90	1.36	9.04	5.23	VU	1.5	0.046	2	
LT C1	2.47	0.2	3.48	106	75	1.44	8.90	5.85	VU	1.5	0.080	2	
LT C2	2.66	0.2	2.99	85	60	1.44	8.35	5.59	VU	1.5	0.052	2	
LT C3	0.97	0.2	1.14	50	43	0.83	6.16	3.33	VU	1.5	0.045	2	
LT C4a	1.68	0.2	1.69	48	39	0.76	5.04	3.87	VU	1.5	0.042	2	
LT C4b	1.19	0.015	2.01	40	30	0.97	4.98	0.34	VD	1.5	0.063	60	281.8
LT D1	3.51	0.2	3.71	115	79	1.74	9.70	6.34	VU	1.5	0.027	2	
LT D2	2.25	0.2	4.73	76	50	2.12	5.49	8.34	VU	1.5	0.035	2	
LT E1	8.62	0.2	8.66	115	50	3.48	7.51	11.94	VU	1.5	0.142	2	
LT F1	1.26	0.2	2.24	90	69	1.06	9.36	4.36	VU	1.5	0.021	2	
LT F2	3.28	0.2	4.32	69	44	2.31	5.79	8.24	VU	1.5	0.021	2	
LT G1	0.45	0.015	1.02	31	24	0.44	6.84	0.18	VD	1.5	0.060	60	106.6
NFSLJ1	1.07	0.1	1.99	65	45	1.00	10.06	2.01	VU	1.5	0.021	2	
NFSLJ2	6.78	0.015	5.00	56	18	2.67	7.60	0.66	VD	1.5	0.021	60	1605.5
NLJ1	3.33	0.2	4.58	97	76	2.13	4.59	8.55	VU	1.5	0.046	2	
NLJ2	3.22	0.2	4.53	77	50	2.18	5.96	8.17	VU	1.5	0.028	2	
NLJ3	0.75	0.015	1.58	51	40	0.69	6.95	0.26	VD	1.5	0.021	60	177.6
NLJ4	1.19	0.015	2.43	45	30	1.02	6.18	0.36	VD	1.5	0.021	60	281.8
ROCK1	6.19	0.04	4.46	1250	560	2.19	154.63	0.88	FH	1.5	0.025	0	
Rock2	16.01	0.04	7.09	1400	210	3.41	167.78	1.22	FH	1.5	0.023	0	
Rock3	11.88	0.04	10.29	210	150	3.03	5.83	2.54	FH	1.5	0.037	0	
SABC	1.80	0.02	2.40	32	25	0.97	2.92	0.54	VD	1.5	0.021	50	66.8
SALT1	2.23	0.04	2.75	2000	1175	1.52	300.41	0.56	FH	1.5	0.022	0	

Subbasin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
SALT2	2.39	0.04	2.08	2140	1200	0.85	451.20	0.37	FH	1.5	0.021	0	
SALT3	15.06	0.03	3.84	1240	1075	1.52	42.92	0.69	FH	1.5	0.045	0	
SFSLJ1	0.75	0.2	1.55	58	44	0.76	9.01	3.36	VU	1.5	0.021	2	
SFSLJ2	3.30	0.015	4.70	52	19	2.35	7.03	0.62	VD	1.5	0.021	60	781.4
SFSLJ A1	5.93	0.2	8.66	110	44	4.92	7.63	13.58	VU	1.5	0.044	2	
SLJ	4.45	0.1	6.95	78	49	3.66	4.17	6.25	VU	1.5	0.028	5	
STAGE	0.50	0.015	0.75	30	28	0.29	2.68	0.17	VD	1.5	0.021	60	155.4
TE A1	3.49	0.1	2.25	166	128	1.04	16.86	1.94	VU	1.5	0.021	2	
TE B1	2.72	0.1	4.51	162	105	2.58	12.65	3.76	VU	1.5	0.021	2	
TE B2	2.27	0.1	3.83	128	105	2.09	6.01	3.76	VU	1.5	0.021	2	
TE B3	2.09	0.1	2.31	84	67	1.29	7.36	2.49	VU	1.5	0.021	2	
TE B4	2.83	0.2	4.89	101	59	1.59	8.60	6.95	VU	1.5	0.023	2	
TE C1	2.66	0.2	4.39	71	37	1.80	7.74	7.14	VU	1.5	0.022	2	
TE D1	4.53	0.1	3.18	200	147	1.99	16.66	2.84	VU	1.5	0.027	2	
TE D2	5.34	0.15	8.90	147	99	4.64	5.39	10.75	VU	1.5	0.033	2	
TE D3	2.90	0.15	4.68	107	69	2.50	8.12	6.16	VU	1.5	0.025	2	
TE E1	4.18	0.1	4.77	185	107	2.48	16.34	3.61	VU	1.5	0.021	2	
TE F1	5.13	0.15	6.69	165	85	3.45	11.97	7.40	VU	1.5	0.044	2	
TE F2	3.47	0.15	4.98	128	78	2.65	10.04	6.19	VU	1.5	0.021	2	
TURN	2.33	0.015	2.15	15	8	0.93	3.25	0.37	VD	1.5	0.022	60	116.9
UPLJ1	35.10	0.04	19.98	1860	550	10.61	65.57	3.32	FH	1.5	0.023	0	
UPLJ2	51.62	0.04	20.00	1400	150	11.36	62.49	3.44	FH	1.5	0.030	0	
Web1a	3.72	0.1	4.55	75	45	2.08	6.60	3.94	VU	1.5	0.021	5	
Web1b	1.11	0.015	3.28	45	25	1.70	6.10	0.49	VD	1.5	0.021	60	262.8
Web2a	1.42	0.015	2.27	25	20	1.00	2.20	0.42	VD	1.5	0.021	60	336.3
Web2b	0.89	0.04	1.89	22	12	0.95	5.28	0.87	VD	1.5	0.021	50	50.0
WPIP	0.93	0.02	1.38	19	10	0.40	6.50	0.27	VD	1.5	0.021	50	60.7

Attachment 6- F. French Camp Slough Subbasin Soil Groups and Loss Rates

Soil Group:	A (0.35 in/hr)	B (0.2 in/hr)	C (0.1 in/hr)	D (0.025 in/hr)	Composite Loss Rate	Adjusted Loss Rate (Adjustment Factor = 0.85)
Subbasin	[sq. mi.]	[sq. mi.]	[sq. mi.]	[sq. mi.]	[in/hr]	[in/hr]
ARCH	0.00	0.00	0.00	0.74	0.025	0.021
BACH	0.00	1.03	3.54	38.69	0.035	0.030
CLAY	0.00	0.00	0.00	0.10	0.025	0.021
DC1	0.00	0.02	2.56	3.01	0.060	0.051
DC10	0.00	0.00	0.00	0.04	0.025	0.021
DC11	0.00	0.00	0.00	0.23	0.025	0.021
DC2	0.00	0.00	0.19	5.71	0.027	0.023
DC3	0.00	0.06	0.07	3.92	0.029	0.024
DC4	0.00	0.00	0.02	3.78	0.025	0.022
DC5	0.00	0.00	0.01	1.67	0.025	0.021
DC6	0.00	0.00	0.00	0.92	0.025	0.021
DC7	0.00	0.00	0.00	0.32	0.025	0.021
DC8	0.00	0.00	0.00	0.62	0.025	0.021
DC9	0.00	0.00	0.00	0.16	0.025	0.021
DCAP	0.00	0.00	0.00	1.79	0.025	0.021
DUCK	0.00	1.58	0.50	26.15	0.036	0.031
FARM	0.00	1.57	3.30	24.54	0.043	0.036
FCS1	0.00	0.29	0.01	1.39	0.056	0.047
FCS2	0.00	0.20	0.00	0.26	0.101	0.086
FCS3	0.00	0.06	0.00	0.19	0.070	0.059
FCS4	0.00	0.00	0.00	0.18	0.025	0.021
FCS5	0.00	0.00	0.00	0.30	0.025	0.021
FCS6	0.00	0.00	0.00	0.34	0.025	0.021
FCS7	0.00	0.00	0.00	0.01	0.025	0.021
GRUPE	0.00	0.01	0.00	0.17	0.037	0.032
GTWY	0.00	0.00	0.00	0.77	0.025	0.021
LJ1	0.00	0.88	2.47	2.91	0.079	0.067
LJ2	0.00	0.16	1.24	0.00	0.111	0.095
LT A1	0.00	0.00	0.00	2.44	0.025	0.021
LT A2	0.00	0.00	0.00	4.00	0.025	0.021
LT A3	0.00	0.10	0.00	0.06	0.131	0.111
LT B1	0.00	0.00	1.36	2.20	0.054	0.046
LT B2	0.00	0.51	0.48	2.44	0.062	0.052
LT B3	0.05	0.11	0.38	3.53	0.041	0.035
LT B4	0.53	0.99	0.65	0.57	0.169	0.144
LT B5	0.04	0.32	0.03	2.03	0.054	0.046
LT C1	0.00	1.12	0.06	1.71	0.095	0.080
LT C2	0.02	0.31	0.67	2.13	0.061	0.052
LT C3	0.00	0.04	0.29	0.69	0.053	0.045
LT C4a	0.09	0.01	0.19	1.58	0.049	0.042
LT C4b	0.03	0.23	0.38	0.95	0.074	0.063

Soil Group:	A (0.35 in/hr)	B (0.2 in/hr)	C (0.1 in/hr)	D (0.025 in/hr)	Composite Loss Rate	Adjusted Loss Rate (Adjustment Factor = 0.85)
Subbasin	[sq. mi.]	[sq. mi.]	[sq. mi.]	[sq. mi.]	[in/hr]	[in/hr]
LT D1	0.00	0.00	0.35	3.55	0.032	0.027
LT D2	0.00	0.00	0.58	2.07	0.041	0.035
LT E1	0.84	3.83	3.62	0.24	0.167	0.142
LT F1	0.00	0.00	0.00	1.26	0.025	0.021
LT F2	0.00	0.00	0.00	3.26	0.025	0.021
LT G1	0.00	0.09	0.06	0.30	0.070	0.060
NFSLJ1	0.00	0.00	0.00	1.07	0.025	0.021
NFSLJ2	0.00	0.00	0.00	6.55	0.025	0.021
NLJ1	0.00	0.01	1.28	2.05	0.054	0.046
NLJ2	0.00	0.15	0.00	3.06	0.033	0.028
NLJ3	0.00	0.00	0.00	0.75	0.025	0.021
NLJ4	0.00	0.00	0.00	1.19	0.025	0.021
ROCK1	0.00	0.00	0.39	5.78	0.030	0.025
Rock2	0.00	0.07	0.39	15.62	0.028	0.023
Rock3	0.00	0.89	0.79	10.20	0.043	0.037
SABC	0.00	0.00	0.00	1.80	0.025	0.021
SALT1	0.00	0.00	0.03	2.20	0.026	0.022
SALT2	0.00	0.00	0.00	2.39	0.025	0.021
SALT3	0.00	0.23	4.46	8.74	0.053	0.045
SFSLJ1	0.00	0.00	0.00	0.75	0.025	0.021
SFSLJ2	0.00	0.00	0.00	3.23	0.025	0.021
SFSLJ A1	0.00	0.20	1.67	4.06	0.052	0.044
SLJ	0.00	0.00	0.48	3.97	0.033	0.028
STAGE	0.00	0.00	0.00	0.50	0.025	0.021
TE A1	0.00	0.00	0.00	3.64	0.025	0.021
TE B1	0.00	0.00	0.00	3.62	0.025	0.021
TE B2	0.00	0.00	0.00	3.03	0.025	0.021
TE B3	0.00	0.00	0.00	2.09	0.025	0.021
TE B4	0.00	0.00	0.11	3.67	0.027	0.023
TE C1	0.00	0.00	0.03	3.29	0.026	0.022
TE D1	0.00	0.19	0.00	4.34	0.032	0.027
TE D2	0.00	0.56	0.01	6.55	0.039	0.033
TE D3	0.00	0.00	0.22	3.40	0.030	0.025
TE E1	0.00	0.00	0.00	4.16	0.025	0.021
TE F1	0.00	0.00	2.11	3.93	0.051	0.044
TE F2	0.00	0.00	0.00	4.08	0.025	0.021
TURN	0.00	0.00	0.04	2.28	0.026	0.022
UPLJ1	0.00	0.00	1.07	34.04	0.027	0.023
UPLJ2	0.00	0.73	5.25	46.11	0.035	0.030
Web1a	0.00	0.00	0.00	3.72	0.025	0.021
Web1b	0.00	0.00	0.00	1.11	0.025	0.021
Web2a	0.00	0.00	0.00	1.42	0.025	0.021

Soil Group:	A (0.35 in/hr)	B (0.2 in/hr)	C (0.1 in/hr)	D (0.025 in/hr)	Composite Loss Rate	Adjusted Loss Rate (Adjustment Factor = 0.85)
Subbasin	[sq. mi.]	[sq. mi.]	[sq. mi.]	[sq. mi.]	[in/hr]	[in/hr]
Web2b	0.00	0.00	0.00	0.89	0.025	0.021
WPAC	0.00	0.00	0.00	0.93	0.025	0.021

Attachment 6- G. French Camp Slough Depth-Duration-Frequency Tables

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
10 min	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
15 min	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
30 min	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
60 min	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
3 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
6 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
12 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
24 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
48 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
72 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
96 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.112	0.146	0.174	0.213	0.243	0.272	0.304	0.348
10 min	0.160	0.209	0.249	0.305	0.347	0.391	0.436	0.499
15 min	0.194	0.253	0.301	0.368	0.419	0.472	0.528	0.604
30 min	0.266	0.348	0.415	0.506	0.576	0.649	0.725	0.829
60 min	0.367	0.481	0.573	0.699	0.798	0.898	1.003	1.147
3 hour	0.576	0.715	0.834	1.008	1.149	1.301	1.467	1.708
6 hour	0.763	0.937	1.088	1.306	1.486	1.680	1.892	2.200
12 hour	1.003	1.249	1.454	1.743	1.972	2.211	2.465	2.820
24 hour	1.384	1.751	2.047	2.449	2.758	3.070	3.392	3.828
48 hour	1.720	2.160	2.515	2.992	3.355	3.722	4.097	4.601
72 hour	1.941	2.425	2.814	3.337	3.736	4.139	4.551	5.106
96 hour	2.126	2.648	3.068	3.632	4.060	4.490	4.930	5.521

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Farmington Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.605	0.606	0.602	0.604	0.600	0.603	0.603	0.602
10 min	0.609	0.608	0.603	0.605	0.603	0.602	0.602	0.600
15 min	0.608	0.607	0.603	0.604	0.602	0.602	0.602	0.602
30 min	0.605	0.605	0.604	0.601	0.602	0.601	0.600	0.600
60 min	0.609	0.606	0.606	0.605	0.604	0.604	0.603	0.602
3 hour	0.604	0.601	0.600	0.598	0.598	0.597	0.598	0.598
6 hour	0.601	0.598	0.595	0.595	0.594	0.594	0.594	0.595
12 hour	0.600	0.595	0.593	0.591	0.591	0.590	0.590	0.590
24 hour	0.599	0.596	0.595	0.593	0.592	0.592	0.591	0.590
48 hour	0.598	0.595	0.594	0.593	0.593	0.593	0.593	0.593
72 hour	0.595	0.593	0.592	0.591	0.591	0.591	0.592	0.593
96 hour	0.595	0.593	0.592	0.591	0.592	0.592	0.593	0.594

Farmington Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.079	0.104	0.123	0.150	0.170	0.192	0.215	0.246
10 min	0.114	0.149	0.176	0.216	0.245	0.276	0.308	0.350
15 min	0.138	0.180	0.213	0.260	0.296	0.333	0.372	0.426
30 min	0.188	0.246	0.294	0.356	0.406	0.457	0.509	0.583
60 min	0.262	0.341	0.407	0.495	0.564	0.635	0.708	0.808
3 hour	0.407	0.503	0.586	0.706	0.805	0.910	1.027	1.196
6 hour	0.537	0.656	0.758	0.910	1.034	1.168	1.316	1.533
12 hour	0.705	0.870	1.010	1.206	1.365	1.528	1.703	1.948
24 hour	0.971	1.222	1.426	1.701	1.912	2.128	2.347	2.644
48 hour	1.204	1.505	1.749	2.077	2.330	2.584	2.845	3.195
72 hour	1.352	1.684	1.951	2.310	2.586	2.865	3.155	3.546
96 hour	1.482	1.839	2.127	2.514	2.814	3.113	3.423	3.840

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Upper Watershed Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.615	0.616	0.613	0.614	0.611	0.613	0.613	0.612
10 min	0.618	0.618	0.614	0.615	0.613	0.612	0.612	0.610
15 min	0.618	0.616	0.614	0.614	0.612	0.612	0.612	0.612
30 min	0.615	0.615	0.614	0.612	0.612	0.611	0.611	0.610
60 min	0.618	0.616	0.616	0.615	0.614	0.614	0.613	0.612
3 hour	0.614	0.612	0.610	0.609	0.609	0.608	0.609	0.609
6 hour	0.611	0.608	0.607	0.606	0.605	0.606	0.606	0.607
12 hour	0.609	0.606	0.604	0.603	0.603	0.602	0.602	0.602
24 hour	0.609	0.607	0.606	0.605	0.604	0.604	0.603	0.603
48 hour	0.607	0.606	0.604	0.604	0.604	0.604	0.604	0.604
72 hour	0.605	0.603	0.602	0.602	0.602	0.603	0.603	0.604
96 hour	0.605	0.603	0.602	0.602	0.602	0.603	0.603	0.605

Upper Watershed Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.081	0.105	0.125	0.153	0.174	0.196	0.218	0.250
10 min	0.116	0.151	0.179	0.220	0.249	0.280	0.313	0.356
15 min	0.140	0.182	0.217	0.265	0.300	0.338	0.378	0.433
30 min	0.191	0.250	0.298	0.363	0.413	0.464	0.519	0.592
60 min	0.266	0.347	0.413	0.504	0.573	0.645	0.720	0.822
3 hour	0.414	0.512	0.596	0.719	0.820	0.927	1.046	1.218
6 hour	0.546	0.667	0.773	0.927	1.053	1.192	1.342	1.564
12 hour	0.716	0.887	1.029	1.231	1.392	1.559	1.737	1.988
24 hour	0.987	1.244	1.453	1.735	1.950	2.171	2.395	2.703
48 hour	1.222	1.533	1.779	2.116	2.373	2.632	2.897	3.254
72 hour	1.375	1.712	1.984	2.353	2.634	2.923	3.213	3.611
96 hour	1.506	1.870	2.163	2.560	2.862	3.171	3.481	3.911

FRENCH CAMP SLOUGH

NOAA14 Precipitation Frequency Depths

Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.610	0.611	0.608	0.609	0.606	0.608	0.608	0.607
10 min	0.614	0.613	0.609	0.610	0.608	0.607	0.607	0.605
15 min	0.613	0.612	0.609	0.609	0.607	0.607	0.607	0.607
30 min	0.610	0.610	0.609	0.607	0.607	0.606	0.606	0.605
60 min	0.614	0.611	0.611	0.610	0.609	0.609	0.608	0.607
3 hour	0.609	0.607	0.605	0.604	0.604	0.603	0.604	0.604
6 hour	0.606	0.603	0.601	0.601	0.600	0.600	0.600	0.601
12 hour	0.605	0.601	0.599	0.597	0.597	0.596	0.596	0.596
24 hour	0.604	0.602	0.601	0.599	0.598	0.598	0.597	0.597
48 hour	0.603	0.601	0.599	0.599	0.599	0.599	0.599	0.599
72 hour	0.600	0.598	0.597	0.597	0.597	0.597	0.598	0.599
96 hour	0.600	0.598	0.597	0.597	0.597	0.598	0.598	0.600

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.080	0.104	0.124	0.152	0.172	0.194	0.216	0.248
10 min	0.115	0.150	0.178	0.218	0.247	0.278	0.310	0.353
15 min	0.139	0.181	0.215	0.262	0.298	0.336	0.375	0.429
30 min	0.190	0.248	0.296	0.360	0.410	0.461	0.514	0.587
60 min	0.264	0.344	0.410	0.500	0.569	0.640	0.714	0.815
3 hour	0.410	0.508	0.591	0.713	0.813	0.919	1.038	1.208
6 hour	0.541	0.661	0.766	0.919	1.044	1.180	1.329	1.548
12 hour	0.711	0.879	1.020	1.218	1.378	1.543	1.720	1.968
24 hour	0.979	1.234	1.441	1.718	1.931	2.150	2.371	2.676
48 hour	1.214	1.520	1.764	2.098	2.353	2.610	2.873	3.227
72 hour	1.364	1.698	1.967	2.333	2.612	2.894	3.187	3.581
96 hour	1.494	1.854	2.145	2.539	2.838	3.144	3.452	3.879

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FLW

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.137	0.182	0.220	0.273	0.316	0.360	0.407	0.474
10 min	0.196	0.261	0.316	0.392	0.453	0.516	0.584	0.680
15 min	0.237	0.316	0.382	0.474	0.547	0.624	0.706	0.822
30 min	0.328	0.437	0.528	0.656	0.758	0.864	0.978	1.138
60 min	0.444	0.592	0.715	0.888	1.026	1.170	1.324	1.541
3 hour	0.740	0.949	1.125	1.373	1.572	1.782	2.005	2.320
6 hour	1.021	1.296	1.528	1.854	2.115	2.389	2.681	3.092
12 hour	1.393	1.781	2.106	2.561	2.921	3.298	3.696	4.254
24 hour	1.936	2.494	2.955	3.590	4.085	4.597	5.132	5.871
48 hour	2.491	3.180	3.736	4.483	5.053	5.629	6.218	7.013
72 hour	2.910	3.694	4.318	5.147	5.769	6.390	7.018	7.852
96 hour	3.220	4.072	4.746	5.631	6.291	6.943	7.598	8.459

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.658	0.657	0.657	0.656	0.659	0.660	0.661	0.662
10 min	0.659	0.655	0.658	0.657	0.657	0.660	0.661	0.662
15 min	0.658	0.657	0.656	0.657	0.658	0.659	0.661	0.662
30 min	0.657	0.657	0.656	0.658	0.658	0.659	0.661	0.663
60 min	0.656	0.655	0.656	0.657	0.658	0.659	0.660	0.662
3 hour	0.661	0.660	0.659	0.660	0.660	0.660	0.660	0.660
6 hour	0.663	0.663	0.663	0.662	0.662	0.662	0.661	0.661
12 hour	0.665	0.664	0.664	0.663	0.663	0.663	0.663	0.664
24 hour	0.666	0.664	0.664	0.664	0.664	0.664	0.665	0.666
48 hour	0.669	0.668	0.667	0.667	0.667	0.667	0.667	0.667
72 hour	0.671	0.670	0.669	0.669	0.669	0.669	0.669	0.669
96 hour	0.672	0.671	0.671	0.671	0.670	0.670	0.670	0.670

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.090	0.120	0.145	0.179	0.208	0.238	0.269	0.314
10 min	0.129	0.171	0.208	0.258	0.298	0.341	0.386	0.450
15 min	0.156	0.208	0.251	0.311	0.360	0.411	0.467	0.544
30 min	0.215	0.287	0.346	0.432	0.499	0.569	0.646	0.754
60 min	0.291	0.388	0.469	0.583	0.675	0.771	0.874	1.020
3 hour	0.489	0.626	0.741	0.906	1.038	1.176	1.323	1.531
6 hour	0.677	0.859	1.013	1.227	1.400	1.582	1.772	2.044
12 hour	0.926	1.183	1.398	1.698	1.937	2.187	2.450	2.825
24 hour	1.289	1.656	1.962	2.384	2.712	3.052	3.413	3.910
48 hour	1.666	2.124	2.492	2.990	3.370	3.755	4.147	4.678
72 hour	1.953	2.475	2.889	3.443	3.859	4.275	4.695	5.253
96 hour	2.164	2.732	3.185	3.778	4.215	4.652	5.091	5.668

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FLW

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.137	0.182	0.220	0.273	0.316	0.360	0.407	0.474
10 min	0.196	0.261	0.316	0.392	0.453	0.516	0.584	0.680
15 min	0.237	0.316	0.382	0.474	0.547	0.624	0.706	0.822
30 min	0.328	0.437	0.528	0.656	0.758	0.864	0.978	1.138
60 min	0.444	0.592	0.715	0.888	1.026	1.170	1.324	1.541
3 hour	0.740	0.949	1.125	1.373	1.572	1.782	2.005	2.320
6 hour	1.021	1.296	1.528	1.854	2.115	2.389	2.681	3.092
12 hour	1.393	1.781	2.106	2.561	2.921	3.298	3.696	4.254
24 hour	1.936	2.494	2.955	3.590	4.085	4.597	5.132	5.871
48 hour	2.491	3.180	3.736	4.483	5.053	5.629	6.218	7.013
72 hour	2.910	3.694	4.318	5.147	5.769	6.390	7.018	7.852
96 hour	3.220	4.072	4.746	5.631	6.291	6.943	7.598	8.459

Farmington Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.721	0.720	0.719	0.719	0.720	0.721	0.722	0.723
10 min	0.722	0.719	0.720	0.719	0.719	0.721	0.721	0.723
15 min	0.721	0.720	0.718	0.719	0.720	0.720	0.721	0.723
30 min	0.720	0.719	0.719	0.719	0.719	0.720	0.721	0.723
60 min	0.720	0.718	0.719	0.719	0.720	0.720	0.721	0.723
3 hour	0.723	0.721	0.720	0.720	0.720	0.720	0.720	0.720
6 hour	0.724	0.723	0.722	0.721	0.721	0.720	0.720	0.720
12 hour	0.725	0.723	0.722	0.721	0.721	0.721	0.721	0.721
24 hour	0.726	0.724	0.722	0.722	0.722	0.722	0.722	0.722
48 hour	0.728	0.726	0.725	0.725	0.724	0.724	0.724	0.725
72 hour	0.729	0.728	0.727	0.726	0.726	0.726	0.726	0.726
96 hour	0.730	0.729	0.728	0.728	0.728	0.727	0.727	0.727

Farmington Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.099	0.131	0.158	0.196	0.228	0.260	0.294	0.343
10 min	0.142	0.188	0.228	0.282	0.326	0.372	0.421	0.492
15 min	0.171	0.228	0.274	0.341	0.394	0.449	0.509	0.594
30 min	0.236	0.314	0.380	0.472	0.545	0.622	0.705	0.823
60 min	0.320	0.425	0.514	0.638	0.739	0.842	0.955	1.114
3 hour	0.535	0.684	0.810	0.989	1.132	1.283	1.444	1.670
6 hour	0.739	0.937	1.103	1.337	1.525	1.720	1.930	2.226
12 hour	1.010	1.288	1.521	1.846	2.106	2.378	2.665	3.067
24 hour	1.406	1.806	2.134	2.592	2.949	3.319	3.705	4.239
48 hour	1.813	2.309	2.709	3.250	3.658	4.075	4.502	5.084
72 hour	2.121	2.689	3.139	3.737	4.188	4.639	5.095	5.701
96 hour	2.351	2.968	3.455	4.099	4.580	5.048	5.524	6.150

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FLW

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.137	0.182	0.220	0.273	0.316	0.360	0.407	0.474
10 min	0.196	0.261	0.316	0.392	0.453	0.516	0.584	0.680
15 min	0.237	0.316	0.382	0.474	0.547	0.624	0.706	0.822
30 min	0.328	0.437	0.528	0.656	0.758	0.864	0.978	1.138
60 min	0.444	0.592	0.715	0.888	1.026	1.170	1.324	1.541
3 hour	0.740	0.949	1.125	1.373	1.572	1.782	2.005	2.320
6 hour	1.021	1.296	1.528	1.854	2.115	2.389	2.681	3.092
12 hour	1.393	1.781	2.106	2.561	2.921	3.298	3.696	4.254
24 hour	1.936	2.494	2.955	3.590	4.085	4.597	5.132	5.871
48 hour	2.491	3.180	3.736	4.483	5.053	5.629	6.218	7.013
72 hour	2.910	3.694	4.318	5.147	5.769	6.390	7.018	7.852
96 hour	3.220	4.072	4.746	5.631	6.291	6.943	7.598	8.459

Upper Watershed Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.779	0.779	0.780	0.780	0.781	0.780	0.781	0.780
10 min	0.780	0.780	0.781	0.780	0.780	0.781	0.780	0.780
15 min	0.779	0.780	0.780	0.780	0.780	0.780	0.780	0.780
30 min	0.778	0.779	0.780	0.780	0.780	0.780	0.780	0.780
60 min	0.779	0.780	0.781	0.781	0.781	0.781	0.781	0.780
3 hour	0.779	0.780	0.780	0.780	0.781	0.780	0.780	0.780
6 hour	0.778	0.779	0.780	0.780	0.780	0.780	0.780	0.780
12 hour	0.778	0.779	0.780	0.781	0.781	0.782	0.782	0.782
24 hour	0.777	0.779	0.780	0.781	0.781	0.782	0.783	0.783
48 hour	0.777	0.778	0.779	0.780	0.780	0.781	0.782	0.782
72 hour	0.777	0.778	0.778	0.779	0.780	0.780	0.781	0.781
96 hour	0.776	0.777	0.778	0.778	0.779	0.779	0.780	0.780

Upper Watershed Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.107	0.142	0.172	0.213	0.247	0.281	0.318	0.370
10 min	0.153	0.204	0.247	0.306	0.353	0.403	0.456	0.530
15 min	0.185	0.246	0.298	0.370	0.427	0.487	0.551	0.641
30 min	0.255	0.340	0.412	0.512	0.591	0.674	0.763	0.888
60 min	0.346	0.462	0.558	0.694	0.801	0.914	1.034	1.202
3 hour	0.576	0.740	0.878	1.071	1.228	1.390	1.564	1.810
6 hour	0.794	1.010	1.192	1.446	1.650	1.863	2.091	2.412
12 hour	1.084	1.387	1.643	2.000	2.281	2.579	2.890	3.327
24 hour	1.504	1.943	2.305	2.804	3.190	3.595	4.018	4.597
48 hour	1.936	2.474	2.910	3.497	3.941	4.396	4.862	5.484
72 hour	2.261	2.874	3.359	4.010	4.500	4.984	5.481	6.132
96 hour	2.499	3.164	3.692	4.381	4.901	5.409	5.926	6.598

FRENCH CAMP SLOUGH

NOAA14 Precipitation Frequency Depths

Rainfall Zone: FLW

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.137	0.182	0.220	0.273	0.316	0.360	0.407	0.474
10 min	0.196	0.261	0.316	0.392	0.453	0.516	0.584	0.680
15 min	0.237	0.316	0.382	0.474	0.547	0.624	0.706	0.822
30 min	0.328	0.437	0.528	0.656	0.758	0.864	0.978	1.138
60 min	0.444	0.592	0.715	0.888	1.026	1.170	1.324	1.541
3 hour	0.740	0.949	1.125	1.373	1.572	1.782	2.005	2.320
6 hour	1.021	1.296	1.528	1.854	2.115	2.389	2.681	3.092
12 hour	1.393	1.781	2.106	2.561	2.921	3.298	3.696	4.254
24 hour	1.936	2.494	2.955	3.590	4.085	4.597	5.132	5.871
48 hour	2.491	3.180	3.736	4.483	5.053	5.629	6.218	7.013
72 hour	2.910	3.694	4.318	5.147	5.769	6.390	7.018	7.852
96 hour	3.220	4.072	4.746	5.631	6.291	6.943	7.598	8.459

Average Storm Centering Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.750	0.750	0.750	0.750	0.751	0.751	0.752	0.752
10 min	0.751	0.750	0.751	0.750	0.750	0.751	0.751	0.752
15 min	0.750	0.750	0.749	0.750	0.750	0.750	0.751	0.752
30 min	0.749	0.749	0.750	0.750	0.750	0.750	0.751	0.752
60 min	0.750	0.749	0.750	0.750	0.751	0.751	0.751	0.752
3 hour	0.751	0.751	0.750	0.750	0.751	0.750	0.750	0.750
6 hour	0.751	0.751	0.751	0.751	0.751	0.750	0.750	0.750
12 hour	0.752	0.751	0.751	0.751	0.751	0.752	0.752	0.752
24 hour	0.752	0.752	0.751	0.752	0.752	0.752	0.753	0.753
48 hour	0.753	0.752	0.752	0.753	0.752	0.753	0.753	0.754
72 hour	0.753	0.753	0.753	0.753	0.753	0.753	0.754	0.754
96 hour	0.753	0.753	0.753	0.753	0.754	0.753	0.754	0.754

Average Storm Centering Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.103	0.137	0.165	0.205	0.237	0.270	0.306	0.356
10 min	0.147	0.196	0.237	0.294	0.340	0.388	0.439	0.511
15 min	0.178	0.237	0.286	0.356	0.410	0.468	0.530	0.618
30 min	0.246	0.327	0.396	0.492	0.569	0.648	0.734	0.856
60 min	0.333	0.443	0.536	0.666	0.771	0.879	0.994	1.159
3 hour	0.556	0.713	0.844	1.030	1.181	1.337	1.504	1.740
6 hour	0.767	0.973	1.148	1.392	1.588	1.792	2.011	2.319
12 hour	1.048	1.338	1.582	1.923	2.194	2.480	2.779	3.199
24 hour	1.456	1.875	2.219	2.700	3.072	3.457	3.864	4.421
48 hour	1.876	2.391	2.809	3.376	3.800	4.239	4.682	5.288
72 hour	2.191	2.782	3.251	3.876	4.344	4.812	5.292	5.920
96 hour	2.425	3.066	3.574	4.240	4.743	5.228	5.729	6.378

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.637	0.637	0.635	0.635	0.635	0.638	0.637	0.638
10 min	0.637	0.635	0.634	0.635	0.635	0.637	0.638	0.639
15 min	0.636	0.636	0.635	0.636	0.636	0.637	0.638	0.639
30 min	0.638	0.636	0.635	0.636	0.636	0.637	0.638	0.639
60 min	0.637	0.635	0.634	0.635	0.635	0.636	0.637	0.638
3 hour	0.638	0.637	0.636	0.636	0.636	0.637	0.637	0.638
6 hour	0.641	0.639	0.639	0.638	0.638	0.638	0.638	0.638
12 hour	0.643	0.640	0.639	0.638	0.638	0.637	0.637	0.637
24 hour	0.644	0.642	0.640	0.639	0.638	0.637	0.637	0.636
48 hour	0.646	0.644	0.643	0.642	0.641	0.640	0.640	0.639
72 hour	0.647	0.646	0.645	0.643	0.643	0.642	0.641	0.641
96 hour	0.648	0.647	0.646	0.645	0.644	0.644	0.643	0.642

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.107	0.139	0.166	0.205	0.236	0.271	0.307	0.360
10 min	0.154	0.199	0.238	0.293	0.339	0.388	0.441	0.518
15 min	0.186	0.241	0.288	0.356	0.410	0.469	0.533	0.626
30 min	0.259	0.335	0.401	0.495	0.571	0.654	0.743	0.872
60 min	0.345	0.446	0.533	0.659	0.761	0.870	0.989	1.159
3 hour	0.574	0.717	0.841	1.023	1.173	1.338	1.517	1.782
6 hour	0.812	1.001	1.166	1.400	1.592	1.798	2.021	2.343
12 hour	1.126	1.386	1.605	1.910	2.153	2.403	2.671	3.047
24 hour	1.589	1.966	2.268	2.681	2.995	3.312	3.644	4.088
48 hour	2.060	2.546	2.931	3.437	3.810	4.179	4.557	5.049
72 hour	2.420	2.995	3.440	4.011	4.438	4.848	5.255	5.796
96 hour	2.714	3.359	3.853	4.485	4.940	5.388	5.820	6.380

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Farmington Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.637	0.637	0.635	0.635	0.635	0.638	0.637	0.638
10 min	0.637	0.635	0.634	0.635	0.635	0.637	0.638	0.639
15 min	0.636	0.636	0.635	0.636	0.636	0.637	0.638	0.639
30 min	0.638	0.636	0.635	0.636	0.636	0.637	0.638	0.639
60 min	0.637	0.635	0.634	0.635	0.635	0.636	0.637	0.638
3 hour	0.638	0.637	0.636	0.636	0.636	0.637	0.637	0.638
6 hour	0.641	0.639	0.639	0.638	0.638	0.638	0.638	0.638
12 hour	0.643	0.640	0.639	0.638	0.638	0.637	0.637	0.637
24 hour	0.644	0.642	0.640	0.639	0.638	0.637	0.637	0.636
48 hour	0.646	0.644	0.643	0.642	0.641	0.640	0.640	0.639
72 hour	0.647	0.646	0.645	0.643	0.643	0.642	0.641	0.641
96 hour	0.648	0.647	0.646	0.645	0.644	0.644	0.643	0.642

Farmington Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.107	0.139	0.166	0.205	0.236	0.271	0.307	0.360
10 min	0.154	0.199	0.238	0.293	0.339	0.388	0.441	0.518
15 min	0.186	0.241	0.288	0.356	0.410	0.469	0.533	0.626
30 min	0.259	0.335	0.401	0.495	0.571	0.654	0.743	0.872
60 min	0.345	0.446	0.533	0.659	0.761	0.870	0.989	1.159
3 hour	0.574	0.717	0.841	1.023	1.173	1.338	1.517	1.782
6 hour	0.812	1.001	1.166	1.400	1.592	1.798	2.021	2.343
12 hour	1.126	1.386	1.605	1.910	2.153	2.403	2.671	3.047
24 hour	1.589	1.966	2.268	2.681	2.995	3.312	3.644	4.088
48 hour	2.060	2.546	2.931	3.437	3.810	4.179	4.557	5.049
72 hour	2.420	2.995	3.440	4.011	4.438	4.848	5.255	5.796
96 hour	2.714	3.359	3.853	4.485	4.940	5.388	5.820	6.380

FRENCH CAMP SLOUGH

NOAA14 Precipitation Frequency Depths

Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Upper Watershed Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
10 min	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
15 min	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
30 min	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
60 min	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
3 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
6 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
12 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
24 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
48 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
72 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
96 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924

Upper Watershed Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.155	0.201	0.241	0.298	0.344	0.393	0.445	0.522
10 min	0.223	0.289	0.347	0.427	0.493	0.563	0.638	0.748
15 min	0.270	0.350	0.419	0.517	0.596	0.681	0.772	0.905
30 min	0.375	0.487	0.583	0.719	0.830	0.948	1.076	1.260
60 min	0.501	0.650	0.777	0.959	1.107	1.264	1.434	1.679
3 hour	0.832	1.040	1.222	1.486	1.704	1.940	2.200	2.581
6 hour	1.171	1.448	1.685	2.027	2.305	2.604	2.927	3.394
12 hour	1.618	2.000	2.320	2.766	3.119	3.485	3.874	4.420
24 hour	2.280	2.829	3.275	3.877	4.337	4.805	5.286	5.939
48 hour	2.947	3.653	4.212	4.946	5.492	6.034	6.579	7.301
72 hour	3.456	4.284	4.929	5.764	6.377	6.978	7.575	8.355
96 hour	3.870	4.796	5.511	6.425	7.088	7.730	8.364	9.182

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.781	0.781	0.780	0.780	0.780	0.781	0.781	0.781
10 min	0.781	0.780	0.779	0.780	0.780	0.781	0.781	0.782
15 min	0.780	0.780	0.780	0.780	0.780	0.781	0.781	0.782
30 min	0.781	0.780	0.780	0.780	0.780	0.781	0.781	0.782
60 min	0.781	0.780	0.779	0.780	0.780	0.780	0.781	0.781
3 hour	0.781	0.781	0.780	0.780	0.780	0.781	0.781	0.781
6 hour	0.783	0.782	0.782	0.781	0.781	0.781	0.781	0.781
12 hour	0.784	0.782	0.782	0.781	0.781	0.781	0.781	0.781
24 hour	0.784	0.783	0.782	0.782	0.781	0.781	0.781	0.780
48 hour	0.785	0.784	0.784	0.783	0.783	0.782	0.782	0.782
72 hour	0.786	0.785	0.785	0.784	0.784	0.783	0.783	0.783
96 hour	0.786	0.786	0.785	0.785	0.784	0.784	0.784	0.783

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.170	0.204	0.252	0.290	0.332	0.376	0.441
10 min	0.188	0.244	0.292	0.360	0.417	0.476	0.540	0.633
15 min	0.228	0.296	0.353	0.436	0.503	0.576	0.653	0.766
30 min	0.317	0.411	0.492	0.607	0.700	0.801	0.909	1.067
60 min	0.423	0.548	0.655	0.810	0.934	1.067	1.212	1.419
3 hour	0.703	0.879	1.031	1.254	1.438	1.640	1.860	2.181
6 hour	0.992	1.225	1.426	1.714	1.949	2.201	2.474	2.869
12 hour	1.373	1.693	1.964	2.338	2.636	2.946	3.275	3.736
24 hour	1.934	2.398	2.771	3.281	3.666	4.061	4.468	5.014
48 hour	2.503	3.100	3.573	4.191	4.654	5.106	5.568	6.179
72 hour	2.940	3.639	4.187	4.891	5.411	5.913	6.419	7.080
96 hour	3.292	4.080	4.682	5.458	6.014	6.559	7.097	7.781

FRENCH CAMP SLOUGH

NOAA14 Precipitation Frequency Depths

Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.731	0.731	0.733	0.733	0.734	0.733	0.732	0.732
10 min	0.728	0.731	0.733	0.732	0.733	0.733	0.733	0.734
15 min	0.729	0.731	0.734	0.733	0.734	0.734	0.733	0.732
30 min	0.731	0.732	0.733	0.734	0.734	0.734	0.734	0.734
60 min	0.729	0.732	0.733	0.733	0.733	0.733	0.733	0.733
3 hour	0.730	0.733	0.734	0.735	0.735	0.736	0.736	0.735
6 hour	0.731	0.734	0.735	0.736	0.737	0.737	0.737	0.737
12 hour	0.732	0.735	0.737	0.738	0.738	0.739	0.739	0.739
24 hour	0.731	0.734	0.736	0.737	0.738	0.738	0.738	0.739
48 hour	0.731	0.734	0.735	0.736	0.736	0.736	0.736	0.736
72 hour	0.732	0.734	0.735	0.736	0.736	0.736	0.736	0.736
96 hour	0.731	0.733	0.734	0.734	0.735	0.735	0.735	0.734

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.093	0.127	0.155	0.191	0.219	0.246	0.273	0.310
10 min	0.132	0.182	0.221	0.273	0.313	0.352	0.391	0.446
15 min	0.160	0.220	0.268	0.331	0.379	0.426	0.474	0.537
30 min	0.224	0.306	0.372	0.460	0.526	0.593	0.660	0.749
60 min	0.303	0.415	0.505	0.623	0.712	0.802	0.893	1.014
3 hour	0.485	0.641	0.770	0.950	1.091	1.239	1.393	1.605
6 hour	0.650	0.854	1.024	1.262	1.450	1.646	1.854	2.143
12 hour	0.870	1.165	1.408	1.739	1.996	2.261	2.533	2.905
24 hour	1.183	1.607	1.949	2.403	2.747	3.088	3.435	3.902
48 hour	1.466	1.956	2.342	2.848	3.222	3.592	3.960	4.444
72 hour	1.673	2.205	2.621	3.163	3.562	3.952	4.339	4.844
96 hour	1.804	2.358	2.791	3.348	3.762	4.162	4.559	5.066

FRENCH CAMP SLOUGH

NOAA14 Precipitation Frequency Depths

Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Farmington Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
10 min	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
15 min	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
30 min	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
60 min	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
3 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
6 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
12 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
24 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
48 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
72 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
96 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822

Farmington Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.104	0.143	0.173	0.214	0.245	0.275	0.307	0.349
10 min	0.150	0.205	0.248	0.307	0.351	0.395	0.439	0.499
15 min	0.181	0.247	0.300	0.371	0.424	0.478	0.531	0.603
30 min	0.252	0.344	0.418	0.515	0.589	0.664	0.739	0.839
60 min	0.341	0.466	0.566	0.699	0.799	0.899	1.001	1.138
3 hour	0.546	0.718	0.862	1.062	1.220	1.383	1.555	1.795
6 hour	0.731	0.956	1.145	1.409	1.618	1.836	2.067	2.390
12 hour	0.977	1.303	1.570	1.937	2.223	2.514	2.818	3.231
24 hour	1.331	1.800	2.177	2.680	3.059	3.439	3.826	4.340
48 hour	1.648	2.191	2.619	3.181	3.599	4.011	4.423	4.963
72 hour	1.879	2.469	2.931	3.533	3.978	4.413	4.847	5.410
96 hour	2.029	2.644	3.125	3.749	4.207	4.655	5.099	5.673

FRENCH CAMP SLOUGH

NOAA14 Precipitation Frequency Depths

Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Upper Watershed Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.620	0.622	0.624	0.625	0.625	0.622	0.621	0.619
10 min	0.618	0.625	0.623	0.624	0.625	0.623	0.621	0.620
15 min	0.618	0.623	0.627	0.625	0.625	0.624	0.622	0.619
30 min	0.620	0.624	0.625	0.625	0.625	0.624	0.622	0.620
60 min	0.619	0.625	0.626	0.626	0.625	0.624	0.623	0.620
3 hour	0.616	0.620	0.623	0.624	0.625	0.625	0.625	0.624
6 hour	0.613	0.618	0.621	0.623	0.624	0.624	0.625	0.625
12 hour	0.612	0.618	0.621	0.624	0.625	0.626	0.626	0.627
24 hour	0.609	0.617	0.621	0.623	0.624	0.625	0.626	0.626
48 hour	0.606	0.612	0.616	0.618	0.619	0.620	0.620	0.621
72 hour	0.604	0.609	0.612	0.614	0.615	0.616	0.617	0.617
96 hour	0.601	0.605	0.608	0.610	0.611	0.612	0.612	0.613

Upper Watershed Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.079	0.108	0.132	0.163	0.186	0.208	0.232	0.262
10 min	0.112	0.156	0.188	0.233	0.267	0.299	0.332	0.376
15 min	0.136	0.188	0.229	0.282	0.323	0.363	0.402	0.454
30 min	0.190	0.261	0.318	0.392	0.448	0.504	0.559	0.633
60 min	0.257	0.354	0.431	0.532	0.608	0.683	0.759	0.858
3 hour	0.409	0.542	0.654	0.806	0.928	1.052	1.183	1.363
6 hour	0.545	0.719	0.865	1.068	1.228	1.394	1.572	1.818
12 hour	0.727	0.980	1.186	1.471	1.690	1.915	2.146	2.465
24 hour	0.986	1.351	1.644	2.031	2.323	2.615	2.913	3.305
48 hour	1.215	1.631	1.963	2.392	2.710	3.026	3.336	3.750
72 hour	1.381	1.829	2.182	2.639	2.976	3.307	3.638	4.060
96 hour	1.483	1.946	2.312	2.782	3.127	3.466	3.796	4.231

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.721	0.722	0.723	0.724	0.724	0.722	0.722	0.721
10 min	0.720	0.724	0.723	0.723	0.724	0.723	0.722	0.721
15 min	0.720	0.723	0.725	0.724	0.724	0.723	0.722	0.721
30 min	0.721	0.723	0.724	0.724	0.724	0.723	0.722	0.721
60 min	0.721	0.724	0.724	0.724	0.724	0.723	0.723	0.721
3 hour	0.719	0.721	0.723	0.723	0.724	0.724	0.724	0.723
6 hour	0.718	0.720	0.722	0.723	0.723	0.723	0.724	0.724
12 hour	0.717	0.720	0.722	0.723	0.724	0.724	0.724	0.725
24 hour	0.716	0.720	0.722	0.723	0.723	0.724	0.724	0.724
48 hour	0.714	0.717	0.719	0.720	0.721	0.721	0.721	0.722
72 hour	0.713	0.716	0.717	0.718	0.719	0.719	0.720	0.720
96 hour	0.712	0.714	0.715	0.716	0.717	0.717	0.717	0.718

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.092	0.126	0.153	0.188	0.216	0.242	0.269	0.306
10 min	0.131	0.180	0.218	0.270	0.309	0.347	0.386	0.438
15 min	0.158	0.218	0.265	0.327	0.374	0.420	0.466	0.529
30 min	0.221	0.302	0.368	0.454	0.519	0.584	0.649	0.736
60 min	0.299	0.411	0.499	0.615	0.704	0.791	0.881	0.998
3 hour	0.477	0.630	0.758	0.934	1.074	1.218	1.370	1.579
6 hour	0.638	0.837	1.006	1.239	1.423	1.615	1.821	2.105
12 hour	0.852	1.141	1.379	1.704	1.958	2.215	2.482	2.850
24 hour	1.159	1.577	1.912	2.357	2.691	3.029	3.369	3.823
48 hour	1.432	1.911	2.291	2.786	3.157	3.518	3.880	4.359
72 hour	1.630	2.151	2.557	3.086	3.479	3.860	4.245	4.738
96 hour	1.757	2.297	2.718	3.266	3.670	4.060	4.448	4.956

FRENCH CAMP SLOUGH

NOAA14 Precipitation Frequency Depths

Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.623	0.620	0.620	0.619	0.618	0.618	0.617	0.619
10 min	0.622	0.618	0.620	0.618	0.618	0.618	0.618	0.619
15 min	0.623	0.619	0.617	0.618	0.617	0.617	0.617	0.619
30 min	0.624	0.619	0.619	0.618	0.618	0.618	0.619	0.619
60 min	0.623	0.619	0.617	0.617	0.617	0.616	0.617	0.618
3 hour	0.623	0.621	0.619	0.618	0.618	0.618	0.618	0.618
6 hour	0.625	0.622	0.621	0.619	0.619	0.618	0.618	0.617
12 hour	0.625	0.622	0.620	0.617	0.616	0.615	0.614	0.614
24 hour	0.625	0.621	0.618	0.616	0.615	0.613	0.612	0.611
48 hour	0.624	0.621	0.619	0.618	0.617	0.615	0.615	0.614
72 hour	0.625	0.623	0.621	0.620	0.618	0.618	0.617	0.616
96 hour	0.626	0.624	0.623	0.622	0.621	0.620	0.619	0.618

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.092	0.118	0.141	0.171	0.197	0.222	0.250	0.290
10 min	0.132	0.169	0.202	0.246	0.281	0.319	0.358	0.415
15 min	0.160	0.205	0.242	0.297	0.340	0.385	0.433	0.503
30 min	0.224	0.287	0.340	0.415	0.475	0.539	0.607	0.702
60 min	0.298	0.383	0.453	0.553	0.634	0.717	0.808	0.936
3 hour	0.488	0.607	0.708	0.855	0.975	1.103	1.240	1.438
6 hour	0.671	0.826	0.959	1.146	1.299	1.458	1.632	1.874
12 hour	0.903	1.115	1.290	1.531	1.722	1.918	2.123	2.411
24 hour	1.238	1.535	1.774	2.098	2.346	2.592	2.847	3.191
48 hour	1.543	1.917	2.212	2.605	2.897	3.180	3.474	3.854
72 hour	1.780	2.217	2.555	2.999	3.319	3.643	3.959	4.371
96 hour	1.974	2.459	2.837	3.323	3.676	4.019	4.357	4.793

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Farmington Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.655	0.653	0.652	0.651	0.649	0.650	0.649	0.651
10 min	0.655	0.651	0.652	0.651	0.650	0.650	0.650	0.650
15 min	0.657	0.652	0.649	0.650	0.649	0.649	0.649	0.650
30 min	0.656	0.652	0.651	0.649	0.650	0.650	0.650	0.651
60 min	0.657	0.652	0.649	0.649	0.649	0.649	0.649	0.649
3 hour	0.656	0.653	0.651	0.649	0.649	0.649	0.649	0.649
6 hour	0.656	0.653	0.651	0.650	0.649	0.648	0.648	0.647
12 hour	0.656	0.652	0.649	0.647	0.646	0.644	0.643	0.642
24 hour	0.656	0.651	0.649	0.646	0.644	0.643	0.642	0.640
48 hour	0.655	0.652	0.649	0.648	0.646	0.645	0.644	0.643
72 hour	0.655	0.653	0.651	0.649	0.648	0.647	0.646	0.645
96 hour	0.656	0.654	0.653	0.651	0.650	0.650	0.649	0.648

Farmington Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.097	0.125	0.148	0.180	0.206	0.234	0.263	0.305
10 min	0.139	0.178	0.212	0.259	0.296	0.335	0.377	0.436
15 min	0.169	0.216	0.255	0.313	0.358	0.405	0.456	0.528
30 min	0.236	0.302	0.358	0.436	0.500	0.567	0.638	0.738
60 min	0.315	0.403	0.476	0.582	0.667	0.755	0.850	0.983
3 hour	0.514	0.639	0.745	0.898	1.023	1.158	1.303	1.510
6 hour	0.704	0.867	1.005	1.203	1.362	1.529	1.711	1.965
12 hour	0.947	1.168	1.351	1.605	1.806	2.009	2.223	2.521
24 hour	1.299	1.609	1.863	2.200	2.457	2.719	2.987	3.342
48 hour	1.619	2.013	2.320	2.732	3.033	3.335	3.637	4.036
72 hour	1.865	2.323	2.678	3.139	3.480	3.814	4.145	4.576
96 hour	2.069	2.577	2.973	3.478	3.848	4.214	4.568	5.026

FRENCH CAMP SLOUGH

NOAA14 Precipitation Frequency Depths

Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Upper Watershed Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.718	0.717	0.717	0.716	0.715	0.714	0.713	0.714
10 min	0.717	0.716	0.717	0.716	0.716	0.714	0.714	0.713
15 min	0.719	0.716	0.716	0.715	0.714	0.714	0.713	0.713
30 min	0.719	0.717	0.716	0.715	0.715	0.715	0.714	0.713
60 min	0.719	0.717	0.715	0.715	0.714	0.714	0.714	0.713
3 hour	0.717	0.716	0.715	0.715	0.715	0.714	0.714	0.714
6 hour	0.716	0.715	0.715	0.714	0.714	0.713	0.713	0.713
12 hour	0.715	0.714	0.713	0.712	0.712	0.711	0.711	0.710
24 hour	0.714	0.713	0.712	0.711	0.710	0.709	0.709	0.708
48 hour	0.712	0.711	0.711	0.710	0.710	0.709	0.708	0.708
72 hour	0.711	0.711	0.710	0.710	0.709	0.709	0.708	0.708
96 hour	0.711	0.711	0.710	0.710	0.710	0.709	0.709	0.709

Upper Watershed Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.106	0.137	0.163	0.198	0.227	0.257	0.289	0.334
10 min	0.152	0.196	0.233	0.285	0.326	0.368	0.414	0.478
15 min	0.185	0.237	0.281	0.344	0.393	0.446	0.501	0.579
30 min	0.258	0.332	0.394	0.480	0.550	0.623	0.700	0.809
60 min	0.344	0.443	0.525	0.641	0.733	0.831	0.935	1.079
3 hour	0.561	0.700	0.818	0.989	1.128	1.274	1.433	1.661
6 hour	0.768	0.950	1.104	1.322	1.499	1.683	1.882	2.165
12 hour	1.032	1.279	1.484	1.766	1.990	2.218	2.459	2.788
24 hour	1.414	1.763	2.043	2.422	2.709	2.998	3.298	3.697
48 hour	1.760	2.195	2.541	2.993	3.333	3.666	3.999	4.444
72 hour	2.025	2.530	2.921	3.434	3.808	4.180	4.543	5.023
96 hour	2.242	2.802	3.233	3.794	4.203	4.596	4.991	5.499

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.687	0.685	0.685	0.684	0.682	0.682	0.681	0.683
10 min	0.686	0.684	0.685	0.684	0.683	0.682	0.682	0.682
15 min	0.688	0.684	0.683	0.683	0.682	0.682	0.681	0.682
30 min	0.688	0.685	0.684	0.682	0.683	0.683	0.682	0.682
60 min	0.688	0.685	0.682	0.682	0.682	0.682	0.682	0.681
3 hour	0.687	0.685	0.683	0.682	0.682	0.682	0.682	0.682
6 hour	0.686	0.684	0.683	0.682	0.682	0.681	0.681	0.680
12 hour	0.686	0.683	0.681	0.680	0.679	0.678	0.677	0.676
24 hour	0.685	0.682	0.681	0.679	0.677	0.676	0.676	0.674
48 hour	0.684	0.682	0.680	0.679	0.678	0.677	0.676	0.676
72 hour	0.683	0.682	0.681	0.680	0.679	0.678	0.677	0.677
96 hour	0.684	0.683	0.682	0.681	0.680	0.680	0.679	0.679

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.102	0.131	0.155	0.189	0.217	0.246	0.276	0.320
10 min	0.145	0.187	0.223	0.272	0.311	0.352	0.396	0.458
15 min	0.177	0.226	0.268	0.329	0.376	0.426	0.478	0.554
30 min	0.247	0.317	0.376	0.458	0.525	0.596	0.669	0.773
60 min	0.330	0.423	0.501	0.612	0.700	0.794	0.893	1.031
3 hour	0.538	0.670	0.781	0.943	1.076	1.217	1.369	1.587
6 hour	0.736	0.908	1.055	1.262	1.432	1.607	1.798	2.065
12 hour	0.991	1.224	1.417	1.687	1.898	2.115	2.341	2.655
24 hour	1.356	1.686	1.954	2.313	2.583	2.859	3.145	3.520
48 hour	1.691	2.105	2.430	2.863	3.183	3.501	3.818	4.243
72 hour	1.945	2.427	2.802	3.289	3.647	3.997	4.344	4.803
96 hour	2.157	2.692	3.105	3.639	4.026	4.408	4.779	5.266

Attachment 6- H. ITR Comment Forms for French Camp Slough HEC-HMS Modeling

Lower San Joaquin River Feasibility Study

HYDROLOGIC MODELING REVIEW –FRENCH CAMP SLOUGH WATERSHED

Task Order 6 – LSJRFS Work-In-Kind Hydrology

Reviewer:	Domenichelli & Associates
Review Date:	11-23-10
PBI Response Date:	12-21-10
DA Backcheck:	01-04-11

Memorandum Comments:

1. Section 6.4.3 Reservoirs: In the 5th paragraph the memorandum states that the rating curves were estimated by entering geometries into HEC-RAS. Consider providing the results as an appendix

   PBI Response: Hydraulic calculations associated with assigning reservoir storage/discharge relationships are now included in Attachment B.

   DA Response: Accepted

2. Section 6.4.4 Diversions: The memorandum states that 'In all cases, diversion flows were proportionally based on channel geometries'. This is true in all cases except for Duck Creek. The Duck Creek bifurcation has a structure to control the flow diverted to Littlejohns Creek.

   PBI Response: Section 6.4.4 has been updated.

   DA Response: Accepted

3. Section 6.5 Model Calibration:
   a. An adjustment factor of 0.60 for the constant loss rates is large. When comparing sub-basins comprised entirely of soil group D for the French Camp Slough (0.015 after adjustment) to Calaveras River (0.023 after adjustment) soil loss rates are decreased by 35%. Given the proximity of Calaveras River to Duck Creek this difference is significant. When considering the effect of a 0.6 adjustment factor it would convert the PBI assumed loss rate for a type B soil from the typical range (per table 2) to the range for a Type C soil. This is not consistent with the use of NRCS soils data for the basis of assigning loss rates.

PBI Response: See response to 3(b). An adjustment factor of 0.6 is no longer used.

DA Response: Accepted

b. Use of the Lone Tree Creek gage to determine the entire watershed's soil loss rates is the cause for the large adjustment factor. The Lone Tree Creek gage considers only a handful of sub-basins of similar condition and may not accurately reflect the losses in the upper watershed of Littlejohns creek. Additionally, the method of using a HEC-RAS model extended to ALERT Gage 205 (Lone Tree Creek gage) produces an unknown amount of possible error when converting river stage data to flow. Consider using an adjustment factor and corresponding loss rates similar to the Calaveras River modeling effort where more reliable stream flow data was used.

PBI Response: Agreed. There are too many uncertainties associated with the current calibration. There was very little concurrent rainfall/runoff data to choose from in the French Camp watershed and no rating curve had been established for ALERT Gage 205.

The French Camp Slough's subbasins now use an adjustment factor of 0.85 which was established through calibration of the neighboring Calaveras River watershed.

Section 6.5 has been updated.

DA Response: Accepted

c. Model Sub-basins where farming is prevalent can be expected to have higher loss rates and additional ponding than sub-basins in the eastern portion of the watershed. There are many flat areas where water ponds up to a foot of depth in a field before it discharges into drainage ditches. Consider higher loss rates for the farming areas. This is discussed further in the Model Comments section.

PBI Response: See response to Model Comment #4.

DA Response: Accepted

d. Figure 8: The "projected data points" are unnecessary for the calibration of the model. Consider removing the points from the graph.

PBI Response: See response to 3(b). The calibration technique has been modified and this figure has now been removed.

DA Response: Accepted

Model Comments:

4. In the original Tidewater HEC-HMS model sub-basin sizes were reduced based on the estimated percentage of the sub-basin that would not drain. This reduction in the sub-basin size was done to model the effects of ponding in the fields (especially on farms required to retain all runoff. ie. dairies). The new model re-established basin sizes but did not take into consideration the portions of the basin which do not drain. The element description states the percentage of the basin which does not drain in both the Tidewater model and the new FCS model.

   PBI Response: PBI's calculated subbasin areas now take in to consideration the percent of area estimated to be isolated. These percentages are based on field investigations conducted for the Tidewater Model and are now presented in Appendix A.

   DA Response: The table in Appendix A does not match the percentage of 'No Drain' listed in the subbasin description in the FCS model or the Tidewater model (Ex: basin LT B4 in the model lists 10% no drain but in Appendix A there is no LT B4). Check Appendix A for consistency with the model element description.

   PBI Response: The 'No Drain' subbasin descriptions were removed from the PBI Model. These descriptions were left over from the Tidewater Model and were not up to date.

5. The French Camp Slough model has many storage areas and diversions where the channels encounter embankments due to highways and railroads. Any increase in model flow due to lower loss rates gets stored in these storage areas. The storage upstream of the highways and railroads is increased. The new model has nearly doubled the storage calculated in the Tidewater model which would result in more extensive flooding in areas such as at Highway 99. Highway 99 may no longer be a zone x and the FEMA maps will be expanded in all areas where ponding occurs. As stated in previous comments relative to loss rates, the 0.6 adjustment factor results in significant impacts to the flood plain and should be analyzed in more detail.

   PBI Response: See response to 3(b). An adjustment factor of 0.6 is no longer used.

   DA Response: Accepted

6. Consider changing the datum to NAVD88 for the HEC-HMS storage elevation curves and the future HEC-RAS model. FEMA maps updated in 2009 for the FCS project area are in NAVD88. The previous hydrologic model and FEMA maps were in NGVD29.

PBI Response: Agreed. All elevation-storage functions were converted from NGVD29 to NAVD88 using CORPSCON software. The conversion is now mentioned in *Section 6.4.3: Reservoirs* and a CORPSCON output table is included in Attachment F.

DA Response: Accepted

Attachment 6- I. SPK Comment Forms for French Camp Slough HEC-HMS Modeling

Corps of Engineers, Hydrology Section, Review of French Camp Slough HEC-1 to HEC-HMS model conversion and preliminary report.

31 January 2011

Steven F. Holmstrom, P.E.

The Technical Memorandum for the Lower San Joaquin River Feasibility Study French Camp Slough HEC-HMS modeling DRAFT hydrology report has been reviewed and the following comments are provided.

22. It should be noted in paragraph 6.1.3 Topography that the vertical elevation datum used is NAVD (1988).

    PBI Response: Agreed.

23. The Design Storms procedure described in paragraph 6.3 should reflect the guidance in the “Storm Distribution Procedure” transmitted on January 21, 2011. It is noted that this guidance has not been reviewed and accepted by Peterson-Brustad, Inc. However, upon review and acceptance by all members of the study team, that procedure should be integrated into the report(s).

    PBI Response: Agreed.

24. In paragraph 6.4.3 Reservoirs, the firm name in the fourth paragraph in that section should be changed to “David Ford Consulting Engineers, Inc.”.

    PBI Response: Agreed.

Attachment 7- A. PBI Internal Review Comments and Responses

PBI Internal Review Comment / Response Log

PROJECT: **Lower San Joaquin River Feasibility Study**
REVIEW DOCUMENT(S): **Draft F3 Hydrology Appendix**

DATE: **7/11/2012**

SUBMITTED BY: **Michael Rossiter, PE, CFM**
REVIEWER: **Dave Peterson, PE**

REFERENCE			**COMMENT**				**RESPONSE**				
			Comment Codes: **M**=Mandatory Response; **S**=Suggested Correction; **Q**=Question; **G**=General Comment;				Response Codes: **A**=Agree, will revise; **D**=Disagree, see explanation; **F**=Follow up required; **G**=General Response				
Comment No.	Dwg/Sec	Page/Sht	Code	Description	By	Date	Code	Explanation	By	Date	Backcheck By/Date
1	2.0	11	**M**	Explain why the 72hr design storm was used.	DAP	7/7/12	**A**	Section 2.0 introduction is updated	MJR	7/11/12	DAP / 7/11/12
2	2.3	13	**M**	Explain what you mean by a "standard" 24 hr storm that you compared results with	DAP	7/7/12	**A**	Section 2.3 is updated	MJR	7/11/12	DAP / 7/11/12
3	3.3.1	19	**M**	Need to discuss how basins outside of levees without pump stations are treated. How are they modeled?	DAP	7/7/12	**A**	Section 3.3.1 updated.	MJR	7/11/12	DAP / 7/11/12
4	3.7	36	**M**	Table 3-5 is confusing. Reorganize table to show that you ran all AEP events for all development and storm centering conditions.	DAP	7/7/12	**A**	Table updated.	MJR	7/11/12	DAP / 7/11/12
5	4.3.2	44	**M**	Need more detail on flow split in Paragraph 1.	DAP	7/7/12	**A**	Section 4.3.2 updated.	MJR	7/11/12	DAP / 7/11/12
6	4.3.2	44	**M**	Add pump station location on to Figure 4-2.	DAP	7/7/12	**A**	Figure updated.	MJR	7/11/12	DAP / 7/11/12
7	4.7	58	**M**	Table 4-5 is confusing. Reorganize table to show that you ran all AEP events for all development and storm centering conditions.	DAP	7/7/12	**A**	Table updated.	MJR	7/11/12	DAP / 7/11/12
8	4.7.2	58	**M**	Explain further why you assigned this equivalent record length for the Mosher model	DAP	7/7/12	**A**	Section 4.7.2	MJR	7/11/12	DAP / 7/11/12
9	4.7.1	58	**M**	Tables 4-6 and 4-7 need to include results at the mouth (after Atlas Tract)	DAP	7/7/12	**A**	Tables updated.	MJR	7/11/12	DAP / 7/11/12
10	5.3.1	65	**M**	Need to discuss how basins outside of levees without pump stations are treated. How are they modeled?	DAP	7/7/12	**A**	Section 5.3.1 updated.	MJR	7/11/12	DAP / 7/11/12
11	5.3.1	66	**M**	Add location of Bellota Dam to Figure 5-2	DAP	7/7/12	**A**	Figure updated.	MJR	7/11/12	DAP / 7/11/12
12	5.7	84	**M**	Table 5-7 is confusing. Reorganize table to show that you ran all AEP events for all development and storm centering conditions.	DAP	7/7/12	**A**	Table updated.	MJR	7/11/12	DAP / 7/11/12
13	5.7.1	86	**M**	To avoid confusion, only report results for the Average storm centering	DAP	7/7/12	**A**	Table updated.	MJR	7/11/12	DAP / 7/11/12
14	6.7	109	**M**	Table 6-6 is confusing. Reorganize table to show that you ran all AEP events for all development and storm centering conditions.	DAP	7/7/12	**A**	Table updated.	MJR	7/11/12	DAP / 7/11/12
15	6.7.1	111	**M**	To avoid confusion, only report results for the Average storm centering	DAP	7/7/12	**A**	Tables updated.	MJR	7/11/12	DAP / 7/11/12

Attachment 8- A. SPK Review of Draft F3 Hydrology Appendix

Corps of Engineers, Hydrology Section, Review of LSJRFS Draft F3 Hydrology Report dated 07122012
16 July 2012
Steven F. Holmstrom, P.E.

PBI Responses: 26 July 2012, Michael Rossiter, P.E.

The draft F3 Hydrology Report for the Lower San Joaquin River feasibility study has been reviewed and the following comments are provided.

1. Sections 3.1.2, 4.1.2, 5.1.2 add description of vertical datum that was used. Since 30-meter DEM's and DWR LiDAR data are being used there is at least some elevation data used in the watershed.

   PBI Response: Description of vertical datum was added to the listed sections.

2. Section 4.3.2, "… an additional three pumps at 25.1 cfs …". From the table 4-1 it is apparent that the 25.1 cfs is for each pump for a total of 75.3 cfs. The reference in paragraph 4.3.2 must be made clear that the 25.1 cfs is for each pump.

   PBI Response: Section 4.3.2 updated with re-wording of pump description.

3. Table 4-7, the 1/200 AEP flow value is less in the future condition than in the existing condition. That appears to be inconsistent with other values in the table. Verify and correct table 4-7.

   PBI Response: This inconsistency was due to the Atlas Tract pump station which is coded into the future conditions model. The pump station has 4 pumps which were set to sequentially shut off when the inflow to the pump decreases. For the 200-yr event, the timing was such that the inflow to the Atlas Tract pump station decreased right as the main flood wave in Mosher Slough was passing through. So the pump shut off for ~15 minutes during the passing of the main channel peak flow.

   This was corrected by assigning a 60-minute minimum rest and minimum run time for Atlas Tract pumps #2, #3, and #4. All future conditions production runs were re-run and Table 4-7 was updated. The only flows that saw any change were for the 'Mosher Slough u/s of Bear Creek Confluence" location.

   The only changes to the HEC-HMS model was specifying the 60-minute minimum rest/run time for the Atlas Tract pump station. The previous version of the model has been replaced by this updated model on PBI's FTP site.

4. The flow values for the 1/10 and 1/25 AEP events do not match between tables 5-2 and table 5-7. The values must be verified and corrected as required.

   PBI Response: The minor discrepancies between the two tables for the 1/10 and 1/25 AEP events are the result of the overall peak identified in the USACE table (Table 5-2) having occurred outside of the HEC-HMS simulation window (31DEC1996-19JAN1997).
   …
   For example:
   The USACE hydrograph at Bellota has a peak flow of 9,529 cfs for the 1/10 AEP event which occurs at 21DEC1996-17:00. This is what is recorded in Table 5-2.

   The HEC-HMS model's 72hr design storm occurred between 31DEC1996-0:00 and 04JAN1997-0:00; the model simulation was run from31DEC1996-0:00 to 19JAN1997-0:00. The peak flow at Bellota during the model simulation window was 9,388 cfs and occurred on 02JAN1997-16:00. This is what is recorded in Table 5-7.
   …
   To avoid confusion, the 1/10 and 1/25 AEP peak flows in Table 5-2 were revised to match the modeled peak flows listed in Table 5-7. Table 5-2 is now introduced as: "The following table is based on the information in the USACE amendment and shows the flow-frequency relationship for modeled flows at the Bellota control point."
5. Section 3.3.1 and 5.3.1 "…nearly all cases, these basins drain through the culverts before the water surface elevation in the main channel would cause a closure of the headgate." Explain what the exceptions are and how they will or may be handled in the Hydraulic analysis task of the project.

   PBI Response: These paragraphs were re-worded to explain our assumption:

   "For subbasins that are on the outside of a levee which do not have pump stations, runoff is coded to enter the main channel at road crossings where there are through-levee culverts. The assumption is made that the culvert headgates will remain open and allow outside flow to enter the main channel. This assumption was made to remain conservative and to account for the potential replacement of culverts by pump stations in the future."

6. Since the future condition is the same as the existing condition for the Calaveras River, add a footnote (2) to table 5-7 stating that "there is no change from the existing to the future condition, therefore only one table is shown", and change "Existing Conditions" to read "Existing and Future Conditions (2)".

   PBI Response: Footnote added to Table 5-7. Title of Table 5-7 updated.

7. Table 6-6 and 6-7, The flows for Duck Creek at Highway 99 are lower for the Future condition than for the existing condition for the 1/200 and 1/500 AEP events. Verify the flows and correct the table(s) as required.

   PBI Response: There were 3 pump stations (PS-DC4, PS-DC5 and PS-DC6) added along Duck Creek for the future conditions model. These pump stations regulate flow coming from these future-developed subbasins. Regulated, future flows for Duck Creek at Hwy 99 therefore end up being less than the non-regulated, existing conditions flows for the larger 1/200 and 1/500 AEP events.

8. Section 6.7.2, The equivalent record length for the French Camp system is said to be 10-30 years, whereas the equivalent record length for the other basins is said to be 20-30 years. This appears to be inconsistent. Is there less confidence in the French Camp model or does the record length need to be corrected?

   PBI Response: French Camp Slough model parameters were adjusted based on a calibration of the neighboring Calaveras River watershed. Therefore there’s slightly less confidence in this model than for the other watersheds. The French Camp flows were categorized as flows that were “estimated with a rainfall-runoff-routing model with regional model parameters” which should have an equivalent record length of 10-30 years according to EM 1619. Section 6.7.2 was updated to clarify this.

   Additional Note: The Mosher Slough model parameters were adjusted based on a calibration of the neighboring Bear Creek watershed, however, Mosher Slough flows are largely dependent on pumped flows with known pump capacities and therefore still have a high level of confidence.

From: Holmstrom, Steven F SPK
Sent: Friday, August 10, 2012 2:07 PM
To: 'Michael Rossiter'
Cc: David Peterson (dpeterson@pbieng.com); Williams, Michelle R SPK; High, John M SPK
Subject: LSJR FS F3 Hydrology Appendix Reviewed and Back-checked (UNCLASSIFIED)
Signed By: steven.f.holmstrom@us.army.mil

Classification: UNCLASSIFIED
Caveats: NONE

Mike,

I have reviewed the comments and responses for the subject report as documented in attachment 8-A in the report.

I have no additional comments and I believe that all comments and responses are back-checked and resolved.

This is a fine report. Thank you for the effort.

Michelle and PDT: the final report has been copied to the "\\Amethyst\Projects\" drive in the following directory:
-\LSJRFS\H&H\Hydrology\LSJRFS Hydrology Report_v4_073012.pdf.

Steve
Steven F. Holmstrom, P.E.
CESPK-ED-HH
1325 J Street
Sacramento, CA 95816
(916) 557-7129 phone

Classification: UNCLASSIFIED
Caveats: NONE

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY
IN SUPPORT OF THE
INTERIM FEASIBILITY REPORT

REAL ESTATE PLAN
APPENDIX G

21 APRIL 2014
REVISED: 28 JULY 2014

PREPARED
BY THE
SACRAMENTO DISTRICT
REAL ESTATE DIVISION
SOUTH PACIFIC DIVISION

ANNEX A - Real Estate Policy Guidance Letter No. 31 - Real Estate Support to Civil Works Planning Paradigm (3X3X3)

1. PURPOSE OF THE REAL ESTATE PLAN

This Real Estate Plan (REP) presents the real estate requirements and costs for an Interim Feasibility Report for the Lower San Joaquin River Study. The information contained herein is tentative in nature for planning purposes only. At the time the REP was prepared, the Project Delivery Team (PDT) had reached the TSP milestone, and feasibility-level analysis was just beginning. Footprint maps which identify locations of access, staging, borrow, mitigation and other project features were not available. The information contained within this REP is based on assumptions made by the PDT and estimated acreages of project features. This REP does not fully conform to the requirements of Chapter 12 (ER 405-1-12). This report is for planning purposes only and will be revised for the final plan to conform to Chapter 12.

2. PROJECT AUTHORIZATION

The general authority for flood control investigations in the San Joaquin River Basin arises under the Flood Control Act of 1936 (Public Law [PL] 74-738), Sections 2 and 6 and amended by the Flood Control Act of 1938 (PL 75-761). The Flood Control Act of 1936, Section 6 explicitly permits further reports to be authorized by congressional resolutions. Further studies of this river system were directed in the 8 May 1964 resolution adopted by the Committee on Public Works of the House of Representatives. The resolution reads:

"*Resolved by the Committee on Public Works of the House of Representatives, United States, that the Board of Engineers for Rivers and Harbors is hereby requested to review the reports on the Sacramento-San Joaquin Basin Streams, California, published in House Document No. 367, 81st Congress, 1st session, and other reports, with a view to determine whether any modifications to the recommendations contained therein are advisable at this time, with particular reference to further coordinated development of the water resources in the San Joaquin River Basin, California*."

The LSJRFS is being accomplished in accordance with the Section 905(b) Analysis (Water Resources Development Act (WRDA) 1986) dated 23 September 2004. The Section 905(b) Analysis was approved by the Commander, SPD on 10 June 2005. The Section 905(b) Analysis was prepared with funds identified in House Report 108-357 (Conference Report to accompany H.R. 2745 for the Energy and Water Development Appropriations Act of 2004) for use under the Sacramento-San Joaquin River Basins Comprehensive Study for a reconnaissance study to evaluate environmental restoration, flood protection, and related purposes for the Lower San Joaquin River. House Report 105-190, which accompanied the Energy and Water Development Appropriations Act of 1998 (PL 105-62) authorized the Sacramento and San Joaquin River Basins Comprehensive Study (Comprehensive Study).

The Section 905(b) Analysis determined that there was Federal interest in pursuing feasibility level investigations for potential flood risk reduction and ecosystem restoration projects in the Lower San Joaquin River area. This study has been focused on flood risk reduction through additional scoping and coordination with the non-Federal sponsors, resource agencies and local stakeholders.

This study will only partially address the Sacramento – San Joaquin Basin Streams, California Comprehensive Study authority. Therefore, the LSJRFS will be called an "Interim Feasibility Report" which indicates that the study is addressing the flood risk issues of a specific area within the authority, rather than the entire area authorized for study.

3. PROJECT DESCRIPTION AND LOCATION

TSP Alternative 7a

This REP identifies the real estate requirements and estimated costs for the Tentatively Selected Plan (TSP), Alternative 7a, North and Central Stockton, Delta Front, Lower Calaveras River, and San Joaquin River Levee Improvements excluding RD 17.

The North Stockton area is defined by the right bank levees of the Calaveras River and the levees along the Delta Front traveling northward along Tenmile Slough,, Fourteen Mile Slough, crossing Five Mile Creek, and traveling north to tie into the Federal project levee across Mosher Slough at the Atlas Tract.

The Central Stockton area is defined by the left bank levees of the Stockton Diverting Canal, the left bank of the Calaveras River, the right bank levees of the San Joaquin River, and right bank levees of French Camp Slough.

Design features of Alternative 7a include:

Levee Raises

Raising levee height will increase the level of performance of existing levees. The increase in levee height may require additional levee footprint area to meet design requirements for minimum levee slope and top width.

Levee Reshaping

Improvements to existing levees of the Delta Front, Calaveras River and San Joaquin River will restore them current USACE standards. Typically, the levees will have material added where necessary to recover design height and restore top width to 12 to 20 feet.

Cut-off Walls

This measure would be implemented to address through- and under-seepage issues that affect levee performance and safety. Installation of the cut-off wall is accomplished by degrading the levee to one-half height and creating the wall with a soil-bentonite mix. Once the mix has cured, the levee is restored to design height and side slopes to meet current design standards. The depth of the cut-off walls will typically be from 20 to 80 feet, but may vary depending on subsurface conditions and depth required to stop through and underseepage.

Deep Soil Mixing (Seismic)

This measure would be implemented to provide seismic stability to the Delta Front levees where required. The deep soil mixing (seismic) measure would involve installation of a grid of drilled soil-cement mixed columns aligned longitudinally with, and transverse to, the levee extending beyond the levee prism. This measure acts to minimize lateral deformation of the levee during seismic events.

Erosion Protection

This measure would consist of protection of the water-side banks of levees to prevent or reduce erosion due to high flows, tides, or wave action. Bank protection consists of rock sized to withstand expected flows, tidal action, and wave run-up placed on the levee.

Closure Structures

This measure would include construction of closure structures at the mouth of backwater sloughs such as Smith Canal and Fourteen Mile Slough to provide flood risk management from flood flows in the Lower San Joaquin River and Delta. The closure structures consist of side walls placed in the existing embankments, and a liftable gate crossing the waterway. Typical operation of the gate would have the gate resting on the bottom of the channel, and closed during high water events or maintenance.

4. NON FEDERAL SPONSORS

The San Joaquin Area Flood Control Agency and the Sacramento and San Joaquin Drainage District acting by and through the Central Valley Flood Protection Board of the State of California will be required to serve as the Non-Federal Sponsors (NFS) for construction and operation, maintenance, repair, rehabilitation and replacement responsibilities if this project is authorized. Both sponsors have legal authority to acquire and hold title to real property for the project under State of California Water Code Section 8590. The sponsors also have the power of eminent domain and “quick-take” authorities for this project.

5. LANDS, EASEMENTS AND RIGHTS-OF-WAY

The real estate cost estimate for the Sacramento District Real Estate Division identified general land use types and their values in the study area. The general land use types and their values were approved by the Sacramento District Real Estate Division in April 2014.

The inventory of lands, easements and rights-of-way required to support the project was created by viewing conceptual designs over real photographs by Engineering and Real Estate Divisions. These findings will be revised for the final plan to conform to Chapter 12 (ER 405-1-12).

The following table demonstrates the acreage; ownerships affected and proposed estate for each project feature. This information is tentative in nature and will be revised once the recommended plan is selected.

Table 1: TSP. The following Table 1 provides a summary of acres required and ownerships affected for the TSP Alternative 7a. The TSP alternative covers approximately 33 miles and includes 30.6 miles of cutoff wall, 3 miles of seismic deep soil mixing, 0.5 mile new levee, 6.8 miles of levee improvements, 4.9 miles of bank and erosion protection and 2 control structures. Their descriptions will be developed in the final plan to conform to Chapter 12.

Table 1: TSP Alternative

	TSP Alternative 7a		
REAL ESTATE	**North Stockton**	**Central Stockton**	**TOTAL**
CONSTRUCTION FOOTPRINT (AC)	98	60	**158**
LANDSIDE AFFECTED PARCELS (#)	343	137	**480**
PERMANENT RELOCATIONS (#)	214	80	**294**
BANK PROTECTION EASEMENT (AC)	9	0	**9**
PERPETUAL FLOOD PROTECTION LEVEE EASEMENT (AC)	56.2	56.0	**112.2**
TEMPORARY WORK AREA EASEMENT (AC)	155	111	**266**
BORROW EASEMENT (AC)	100	90	**190**

Access and Temporary Staging

The majority of staging areas for construction of this project will be located within the right-of-way for the levee footprint or existing right-of-way. Specific access and staging areas were not identified. During construction planning analysis indicates that public-owned properties exist and additional areas will need to be acquired. This information is tentative in nature and will be revised once the recommended plan is selected. A standard Temporary Work Area Easement will be acquired for the additional right-of-way necessary for access and staging

Staging areas for construction of the closure structures on Fourteenmile Slough and on Smith Canal would be immediately adjacent to the levees on either side of the closure structures. The Buckley Cove, Louis Park, and Dos Reis Park parking lots could be used for staging of materials and equipment.

Borrow

It is estimated that 1.8 million cubic yards of borrow material could be needed to construct the project. Because the project is in preliminary stages of design, detailed studies of borrow needs have not been completed. For the purposes of NEPA/CEQA a worst case scenario is being evaluated for the volume of borrow material needed. Actual volumes exported from any single borrow site would be adjusted to match for fill.

Potential locations for borrow material were identified by the San Joaquin Area Flood Control Agency, a project sponsor. Three publicly-owned, potential borrow areas include an area west of the Stockton East Water District water treatment plant. This is a 265 acre site and could potentially be excavated as deep as 20 feet. Another site would be at the Tidewater development near French Camp Slough and Highway 99. This site is a 93 acre basin with potentially 1,700 acre-feet of earth volume. At the Mariposa Lake development nestled between Mariposa Road and State Route 4 east of State Route 99 is another potential borrow site. The entire site comprises approximately 6 square miles and approximately 3,500 acres of the site would be available for borrow. The potential borrow material sites have not been field tested, therefore to ensure that sufficient borrow material would be available for construction the Corps looked at all recommended locations for 20 times the needed material. This would allow for sites that do not meet specifications or are not available for excavation of material.

The excavation limits on the borrow sites would provide a minimum buffer of 50 feet from the edge of the borrow site boundary. From this setback, the slope from existing grade down to the bottom of the excavation would be no steeper than 3H:1V. Excavation depths from the borrow sites would vary between 7-10 feet. The borrow sites would be stripped of top material and excavated to appropriate depths. Once material is excavated, borrow sites would be returned to their existing use whenever possible, or these lands could be used to mitigate for project impacts, if appropriate.

Mitigation

The sponsors will purchase credits from mitigation banks in the project area. For planning purposes approximately 56,180 mitigation acres needed at an estimated cost of $53,000,000 for Alternative 7a. The costs to purchase credits from mitigation banks is not a real estate cost.

NFS Owned Lands

Portions of the TSP levee footprints lie within easement interests held by the San Joaquin Area Flood Control Agency and the Sacramento and San Joaquin Drainage District. The NFS has the legal capability to provide the lands required for the TSP Plan.

This information is tentative in nature and will be revised once the recommended plan is selected and sponsor lands can be reviewed for interest owned and sufficiency to support project purposes.

The Non-Federal Sponsor will be notified in writing of the risks of acquiring right-of-way interests before execution of the construction agreement.

6. ESTATES

Non-standard estates are not anticipated for implementation of the TSP Plan. The NFS will acquire the minimum necessary interests in real estate to support the construction and subsequent operation and maintenance of the recommended plan and these standard estates are identified as follows:

BANK PROTECTION EASEMENT

A perpetual and assignable easement and right-of-way in, on, over and across the land hereinafter described for the location, construction, operation, maintenance, alteration, repair, rehabilitation and replacement of a bank protection works, and for the placement of stone, riprap and other materials for the protection of the bank against erosion; together with the continuing right to trim, cut, fell, remove and dispose therefrom all trees, underbrush, obstructions, and other vegetation; and to remove and dispose of structures or obstructions within the limits of the right-of-way; and to place thereon dredged, excavated or other fill material, to shape and grade said land to desired slopes and contour, and to prevent erosion by structural and vegetative methods and to do any other work necessary and incident to the project; together with the right of ingress and egress for such work; reserving, however, to the landowners, their heirs and assigns, all such rights and privileges as may be used without interfering with or abridging the rights and easement hereby acquired; subject, however to existing easements for public roads and highways, public utilities, railroads and pipelines.

FLOOD PROTECTION LEVEE EASEMENT

A perpetual and assignable right and easement in [the land described in Schedule A] to construct, maintain, repair, operate, patrol and replace a flood protection levee, including all appurtenances thereto; reserving, however, to the owners, their heirs and assigns, all such rights and privileges in the land as may be used without interfering with or abridging the rights and easement hereby acquired; subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines.

TEMPORARY WORK AREA EASEMENT

A temporary easement and right-of-way in, on, over and across [the land described in Schedule A] for a period not to exceed ______, beginning with date possession of the land is granted to the United States, for use by the United States, its representatives,

> agents, and contractors as a (borrow area) (work area), including the right to (borrow and/or deposit fill, spoil and waste material thereon) (move, store and remove equipment and supplies, and erect and remove temporary structures on the land and to perform any other work necessary and incident to the construction of the ______ Project, together with the right to trim, cut, fell and remove therefrom all trees, underbrush, obstructions, and any other vegetation, structures, or obstacles within the limits of the right-of-way; reserving, however, to the landowners, their heirs and assigns, all such rights and privileges as may be used without interfering with or abridging the rights and easement hereby acquired; subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines.
>
> ***BORROW EASEMENT***
>
> A perpetual and assignable right and easement to clear, borrow, excavate and remove soil, dirt, and other materials from [the land described in Schedule A] subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines; reserving, however, to the landowners, their heirs and assigns, all such rights and privileges in said land as may be used without interfering with or abridging the rights and easement hereby acquired.

7. EXISTING FEDERAL PROJECTS WITHIN THE STUDY AREA

There are federal projects in the study area. Their descriptions will be developed in the final plan to conform to Chapter 12.

8. FEDERALLY OWNED LANDS NEEDED FOR THE PROJECT

There are no known federally owned lands needed for this project.

9. NAVIGATIONAL SERVITUDE

The navigation servitude is the dominant right of the Government under the Commerce Clause of the U.S. Constitution to use, control and regulate the navigable waters of the United States and submerged lands thereunder.

The rock revetment measure will be constructed from the landside of the levee. The project does not require lands, easements or rights-of-way within any navigable watercourses. Therefore, the Federal Navigational Servitude will not be invoked for this project.

10. BASELINE COST ESTIMATE

The baseline cost estimate is the total costs of the lands combined with the cost of support and administrative activities to acquire those lands. The estimated total costs for Real Estate Acquisition for the TSP follows. The date of the approved cost estimate prepared by Sacramento

District Real Estate Division was April 2014. The costs include land payments as well as administrative costs and incremental costs associated with acquiring the real estate interests to include potential condemnations. Displaced persons and business may be entitled to relocation assistance benefits (P.L. 91-646, Title II as amended). The cost estimate is tentative in nature and will be revised once the recommended plan is selected and appropriate real estate interests are determined.

TSP ALTERNATIVE 7a COST ESTIMATE

	COST	CONTG %	FED TOTAL	NFS TOTAL	TOTAL PROJECT COST (FED + NFS)
FED RE Admin Support Account 01	$10,430,000	5%	**$10,952,000**		
FED Lands and Damages Account 01	0	0	**0**		
NFS RE Admin Support Account 01	$12,660,000	5%		**$13,293,000**	
NFS Lands and Damages Account 01	$79,057,000	35%		**$106,727,000**	
Total Project Cost (FED +NFS):					**$130,972,000**

11. UNIFORM RELOCATION ASSISTANCE (PL 91-646, TITLE II AS AMENDED)

Relocation assistance benefits to residents may be applicable, including storage of household goods, moving costs, lodging, incidentals, differential payments, etc. Businesses could be entitled to receive advisory services, reimbursement for actual reasonable moving costs, re-establishment costs which are capped at $10,000, and certain reasonable and necessary incidental costs associated with the relocation. Cost estimates will be revised after completion of feasibility-level design and appropriate real estate interests are determined.

A preliminary estimate of potential PL 91-646 displacements was prepared by the Sacramento District Real Estate and Engineering Divisions. The impacts and estimates relating to potential displacements, and the anticipated need to provide relocation assistance benefits, are provided exclusively for project cost estimating purposes only and are not intended to be relied upon for provision of benefits and/or payment of the estimates referenced herein. Should the project be authorized, a relocation plan will be provided by the NFS.

UNIFORM RELOCATION ASSISTANCE (PL 91-646)

Alternative	
TSP	335

12. ZONING ORDINANCES

There will be no application or enactment of zoning ordinances in lieu of, or to facilitate, acquisition for structural features of this project. Should plans be developed for non-structural features during feasibility-level design, it is possible that there will be certain building restrictions in areas where elevations or flood proofing measures are proposed, and in areas where there may be buy-out acquisitions.

13. ACQUISITION SCHEDULE

The following acquisition schedule for project features is based on the premise that the project will impact approximately 800 landowners for the levee alignment. It is assumed that the project will be constructed in sections over a 10-15 year period. An acquisition schedule will be prepared when the recommended plan is selected. The schedule below provides the total amount of time to complete the acquisition of real estate rights for mitigation and for the construction of the levee alignment and other project features based on the preliminary information available at this time. This schedule is only for planning purposes and will be updated for the final plan.

REAL ESTATE ACQUISITION SCHEDULE				
Project Name: Lower San Joaquin River Flood Reduction Project	COE Start	COE Finish	NFS Start	NFS Finish
Receipt of Preliminary Drawings from Engineering/PM	TBA	TBA	TBA	TBA
Receipt of Final Drawings from Engineering/PM	TBA	TBA	TBA	TBA
Formal Transmittal of Final Drawings and Instruction to Acquire LEERDS	TBA			
Conduct Landowner Meetings				6 months
Prepare/Review Mapping & Legal Descriptions				1 year
Obtain/Review Title Evidence				1 year
Obtain/Review Tract Appraisals				1 year
Conduct Negotiations				4 years
Condemnation				6 years
Prepare/Review Condemnations				
Perform Condemnations				
Obtain Possession				
Complete/Review PL 91-646 Benefit Assistance				2 years
Certify All Necessary LERRDS for Construction				TBA
Prepare and Submit Credit Requests				TBA
Review/Approve or Deny Credit Requests	TBA	TBA		
Establish Value for Creditable LERRDS	TBA	TBA		

14. FACILITY/UTILITY RELOCATIONS

Preliminary facility and utility relocation data was collected and detailed by the Sacramento District, Engineering Division. At the time of this report, feasibility-level analysis had yet to be performed. The estimated total costs of relocations for all alternatives range from $32,706,000 - $45,204,000.

Real Estate Guidance issued for 3x3x3 studies indicates that if the costs of relocation of facilities and utilities is less than 30% of project costs, a preliminary compensable interest report should not be prepared (refer to Real Estate Policy Guidance Letter Non. 31-Real Estate Support to Civil Works Planning Paradigm (3x3x3) dated January 10, 2013, attached as Exhibit A). Because the estimated cost of relocations does not exceed 30% of total project cost, an Attorney's Preliminary Opinion of Compensable Interest was not prepared for this project. Rather, once the recommended plan is selected and feasibility level of design is complete, a Relocations Report will be prepared and the Real Estate Plan will include a relocations assessment indicating which relocations are covered by the substitute facilities doctrine. A Final Attorney's Opinion of Compensability will be prepared before the project partnership agreement is executed for each utility/facility.

The Non-Federal Sponsor will perform these relocations as a part of its responsibility under the project authority. The Government will make a final determination of the relocations necessary for the construction, operation or maintenance of the project after further analysis, and completion and approval of the Final Attorney's Opinion of Compensability for each of the impacted utilities and facilities.

15. HAZARDOUS, TOXIC AND RADIO ACTIVE WASTE

At the time of this report, a Phase I Environmental Site Assessment has not been conducted. This discussion related to contaminates on lands within the project area will be revised after database searches are completed and a recommended plan is selected.

16. LANDOWNER CONCERNS

The project has received wide-spread support from the community; however, the attitudes of the landowners who will be directly affected by its construction are not known. The Non-Federal Sponsor is confident that they will be able to acquire the right-of-way required for the project.

17. PROJECT MAP

(See attached Exhibit A). These maps indicate the overall project site. Once specific sites are determined, maps will be generated and provided to the Non-Federal Sponsor.

ASSESSMENT OF NON-FEDERAL SPONSOR'S REAL ESTATE ACQUISITION CAPABILITY

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

Sponsor: San Joaquin Area Flood Control Agency
Sacramento and San Joaquin Drainage District acting by and through the Central Valley Flood Protection Board of the State of California

I. Legal Authority:

a. Does the sponsor have legal authority to acquire and hold title to real property for project purposes? **YES**

Please cite the authority: **STATE OF CALIFORNIA WATER CODE SECTION 8590**

b. Does the sponsor have the power of eminent domain for this project? **YES**

c. Does the sponsor have "quick-take" authorities for this project? **YES**

d. Are any of the lands/interests in land required for the project located outside the sponsor's political boundary? **NO**

e. Are any of the lands or interests in land required for the project owned by an entity whose property the sponsor cannot condemn? **NO**

II. Human Resource Requirements:

a. Will the sponsor's in-house staff require training to become familiar with the real estate requirements of Federal projects including P.L. 91-646, as amended? **NO**

b. If the answer to II. a. is "yes," has a reasonable plan been developed to provide such training? **N/A**

c. Does the sponsor's in-house staff have sufficient real estate acquisition experience to meet its responsibilities for the project? **YES**

d. Is the sponsor's project in-house staffing level sufficient considering its other workload, if any, and the project schedule? **YES**

e. Can the sponsor obtain contractor support, if required, in a timely fashion? **YES**

f. Will the sponsor likely request U.S. Army Corps of Engineers (USACE) assistance in acquiring real estate? **NO**

III. Other Project Variables:

a. Will the sponsor's staff be located within reasonable proximity to the project site? **YES**

b. Has the sponsor performed satisfactorily on other USACE projects? **YES**

Prepared by:

Elizabeth A. Youn
Realty Specialist
Acquisition and Management Branch

Date: 21 August 2014

Reviewed and Approved by:

Sharon Caine

Sharon Caine
Chief, Real Estate Division

EXHIBIT A – PROJECT MAP

US Army Corps of Engineers®
Sacramento District

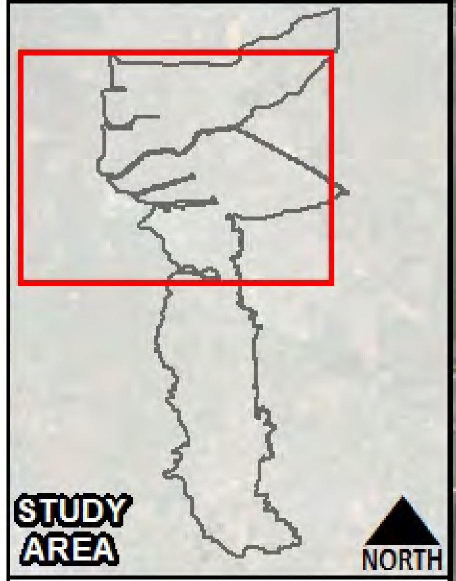
LOWER SAN JOAQUIN
Feasibility Study

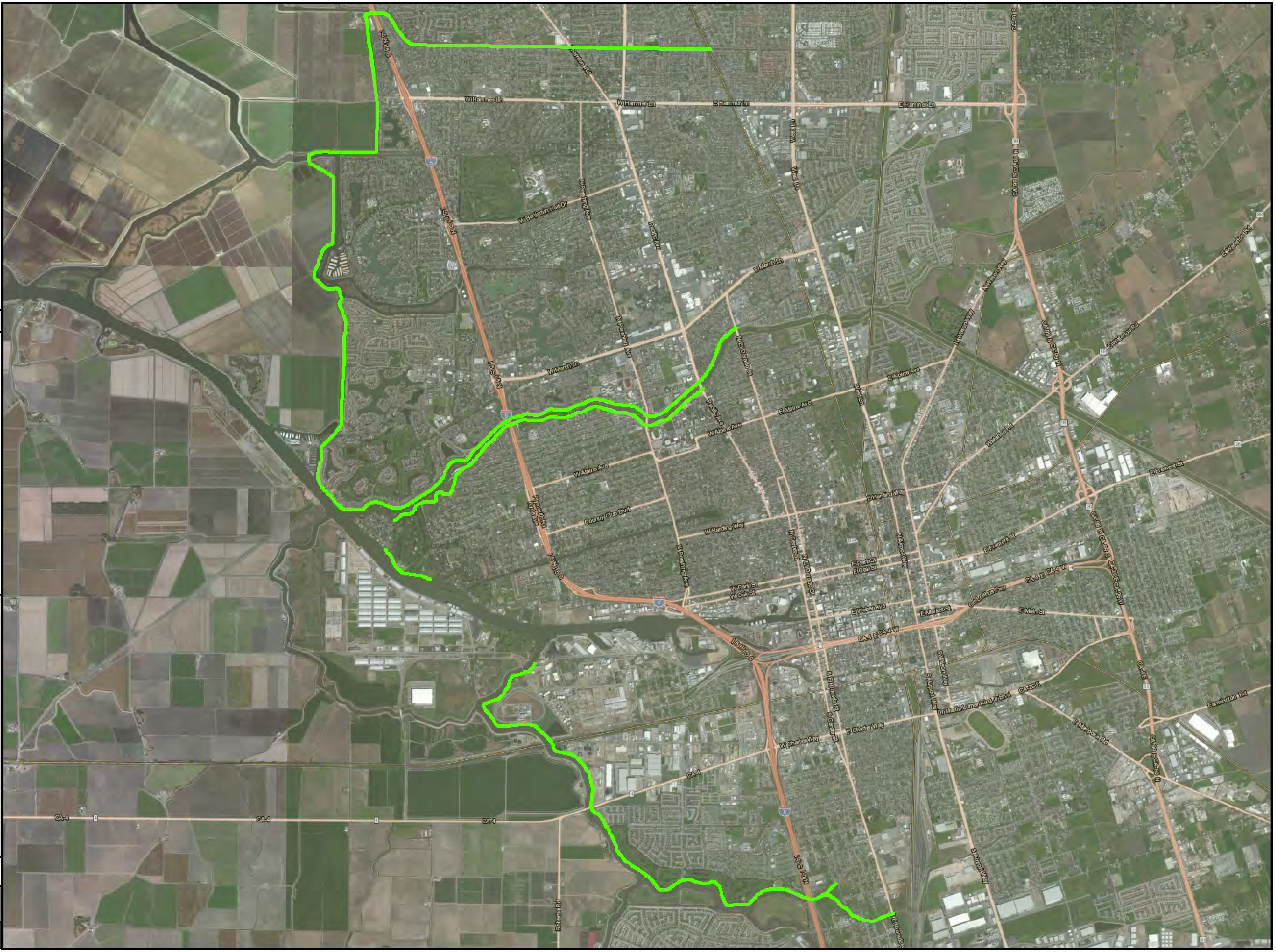
Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.





Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:50,000
Reference Map Scale 1:450,000

US Army Corps of Engineers®
Sacramento District

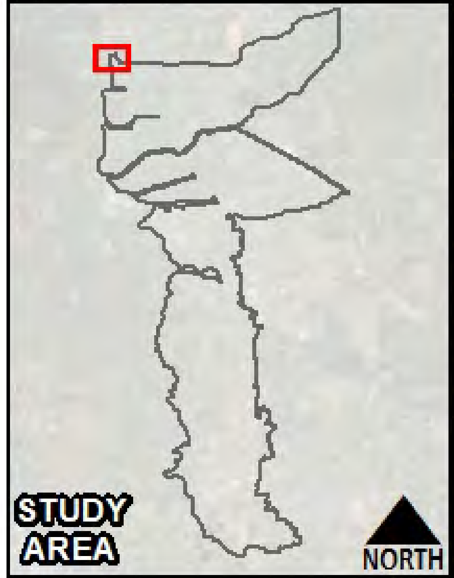
LOWER SAN JOAQUIN
Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

Map Page 2 of 22

US Army Corps of Engineers®
Sacramento District

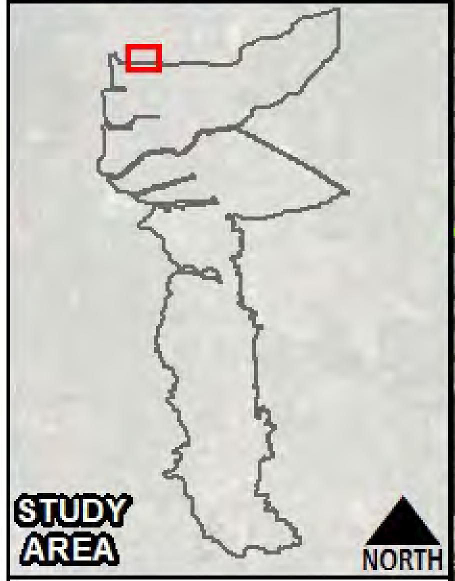
LOWER SAN JOAQUIN
Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

Map Page 3 of 22

US Army Corps of Engineers®
Sacramento District

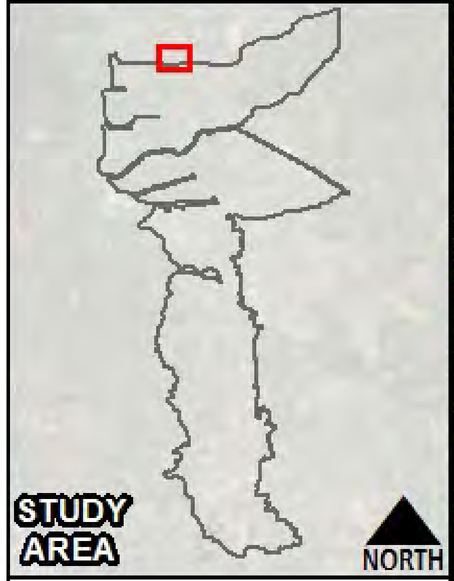
LOWER SAN JOAQUIN
Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

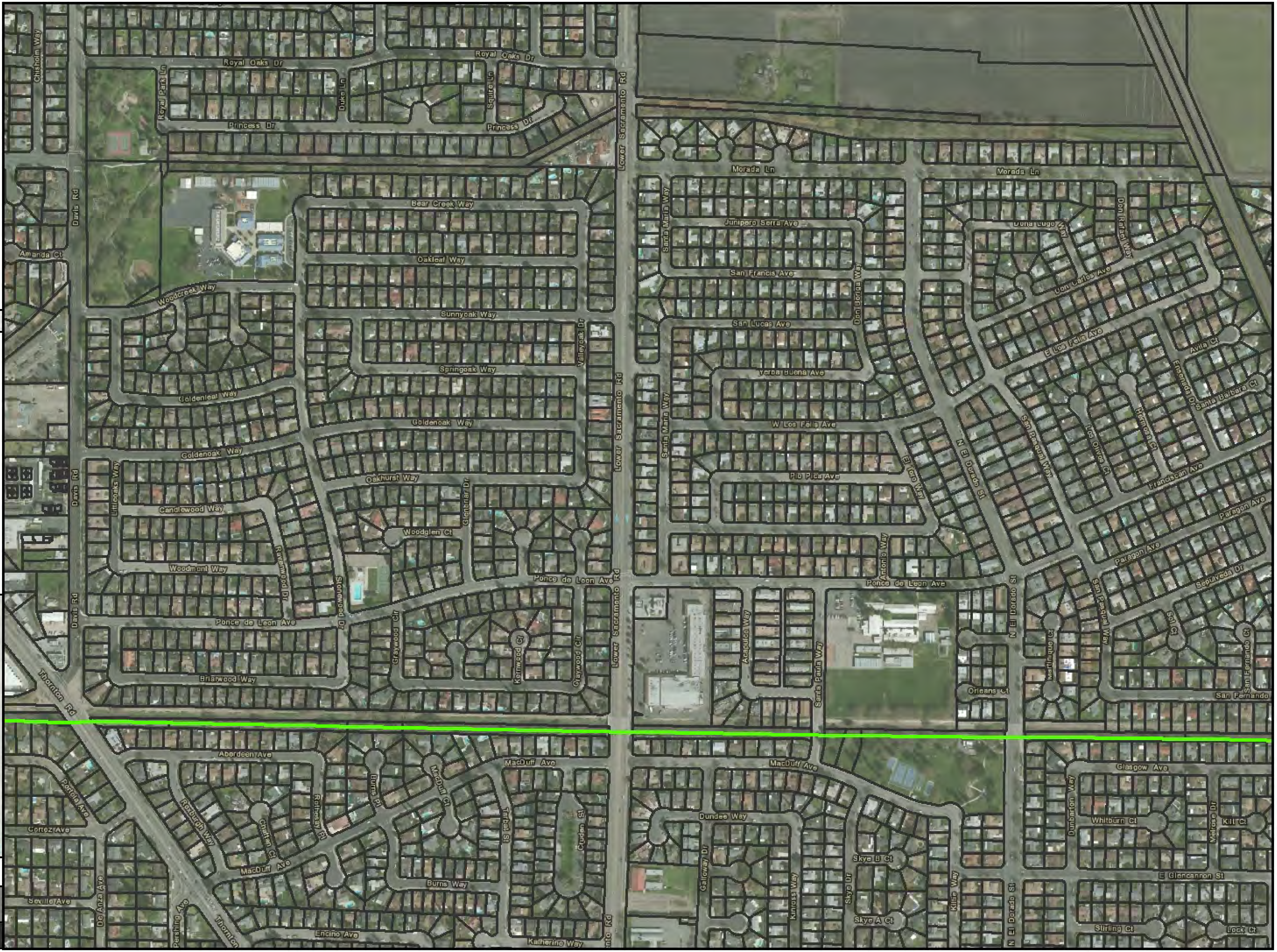
2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

Map Page 4 of 22



US Army Corps of Engineers®
Sacramento District

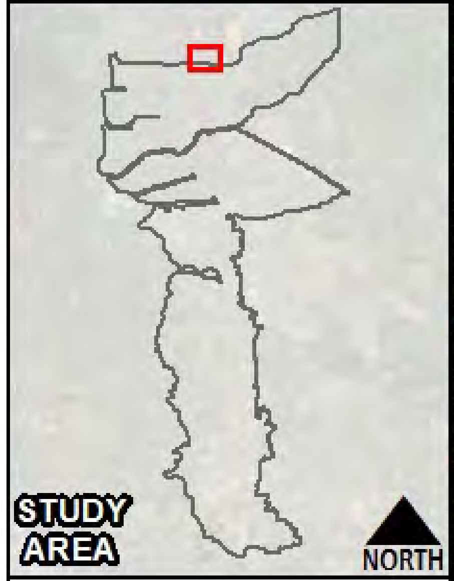
LOWER SAN JOAQUIN
Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

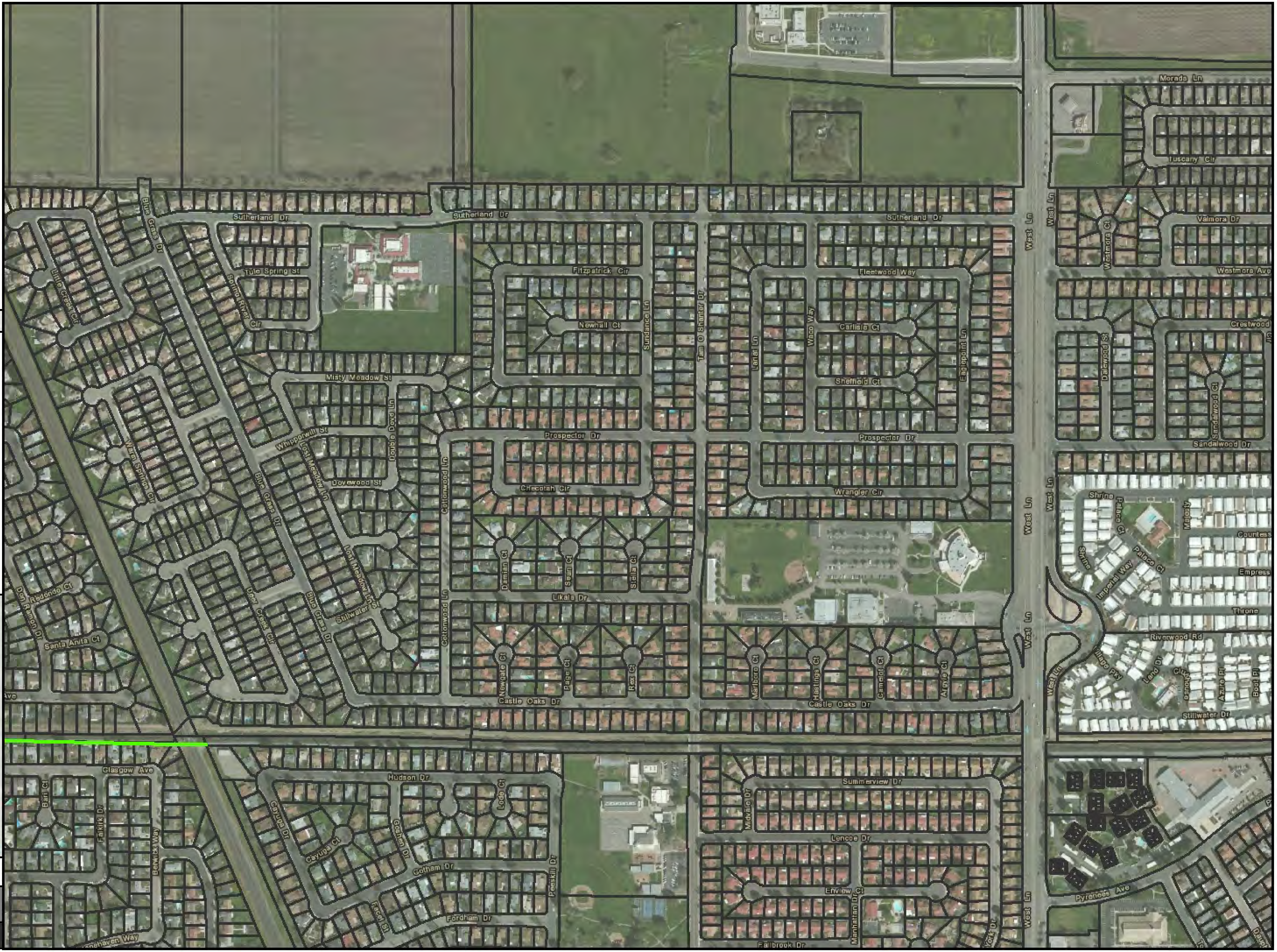
2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

Map Page 5 of 22



US Army Corps of Engineers®
Sacramento District

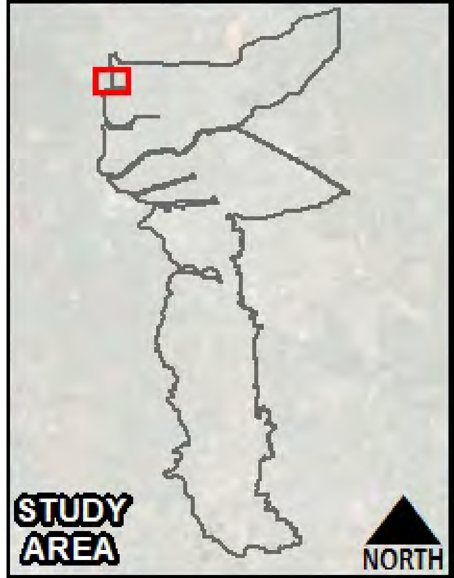
LOWER SAN JOAQUIN
Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

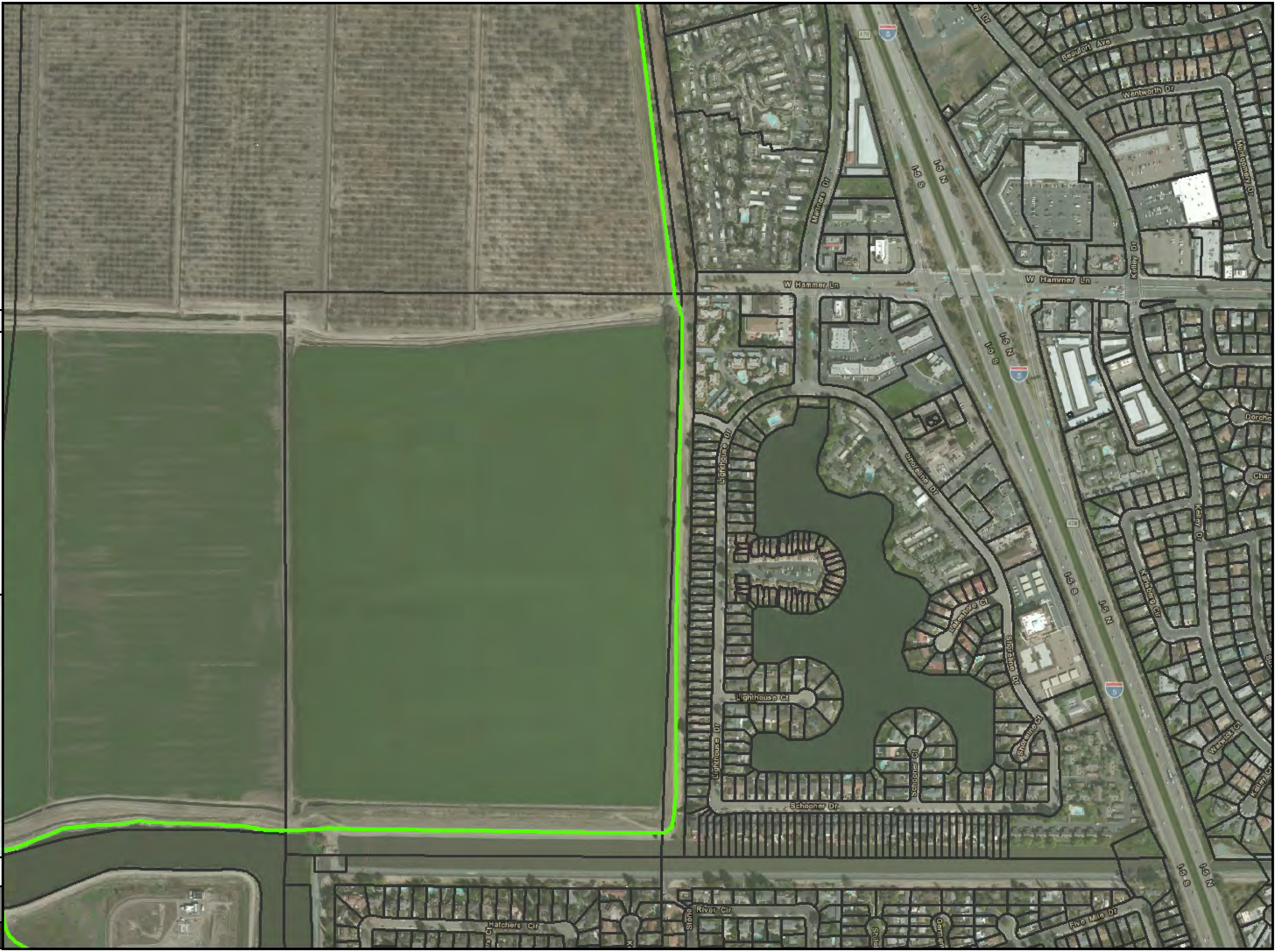
2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

Map Page 6 of 22



US Army Corps of Engineers®
Sacramento District

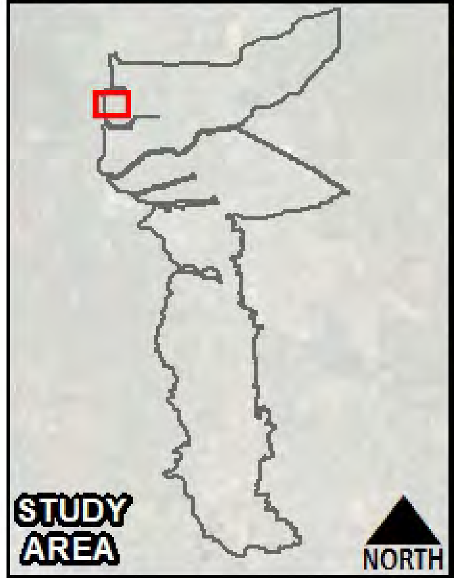
LOWER SAN JOAQUIN Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

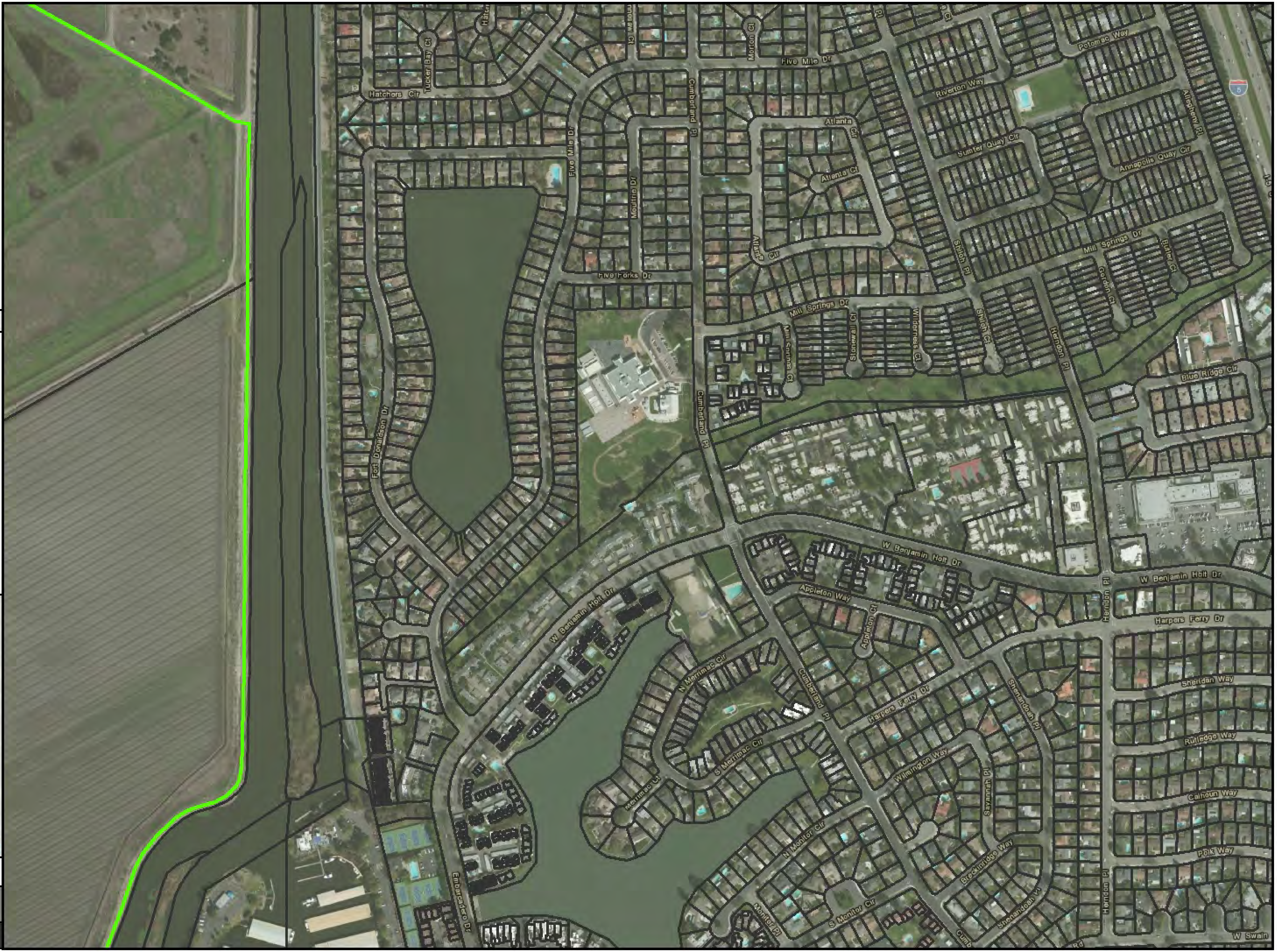
2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

Map Page 7 of 22



US Army Corps of Engineers®
Sacramento District

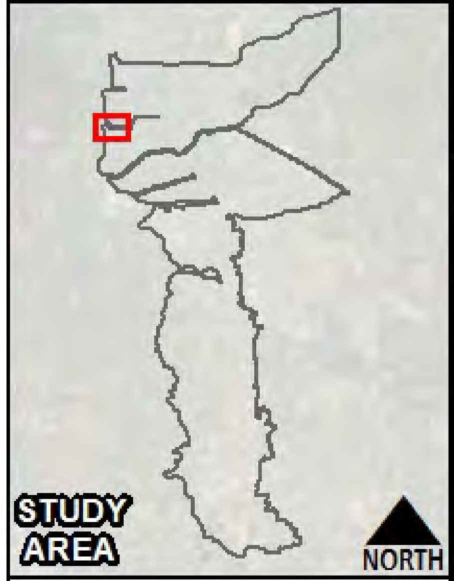
LOWER SAN JOAQUIN
Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

US Army Corps of Engineers®
Sacramento District

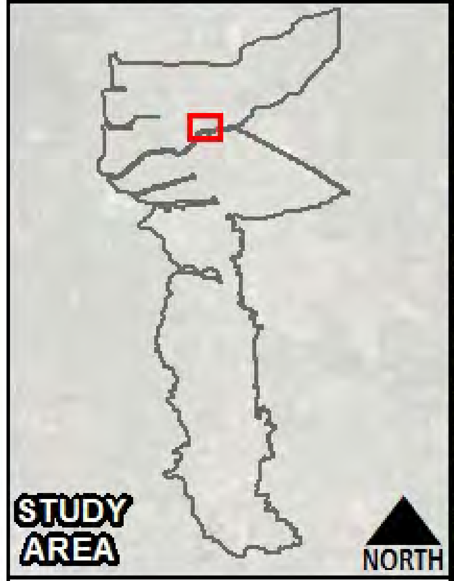
LOWER SAN JOAQUIN
Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

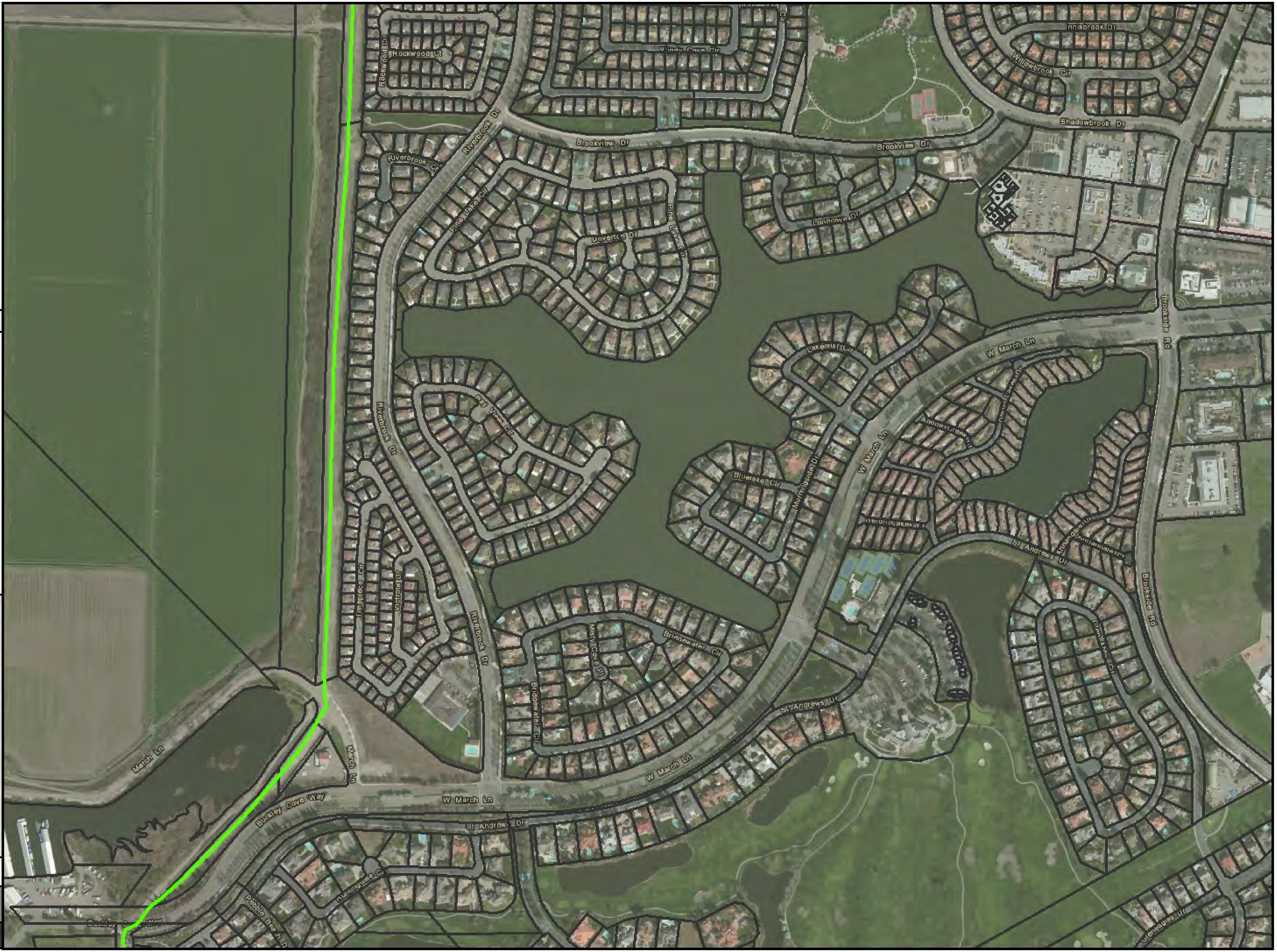
Main Map Scale 1:5,000
Reference Map Scale 1:450,000

Map Page 9 of 22

US Army Corps of Engineers®
Sacramento District

LOWER SAN JOAQUIN Feasibility Study

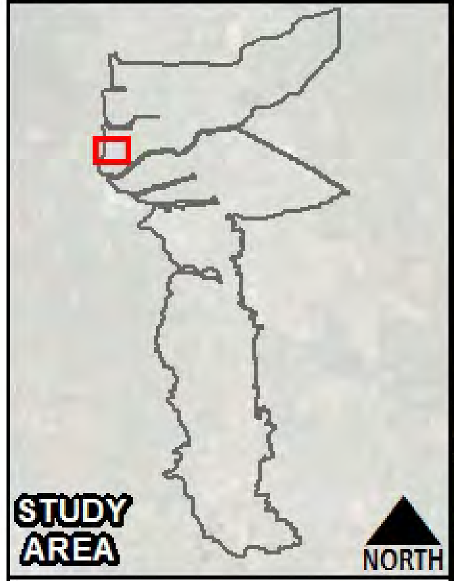
Alternative LS-7a



🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

US Army Corps of Engineers®
Sacramento District

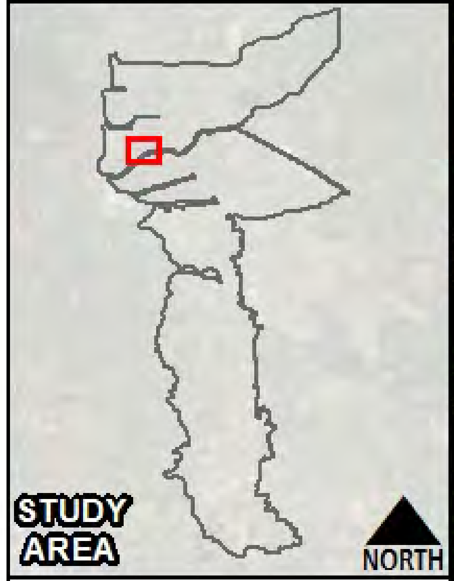
LOWER SAN JOAQUIN
Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

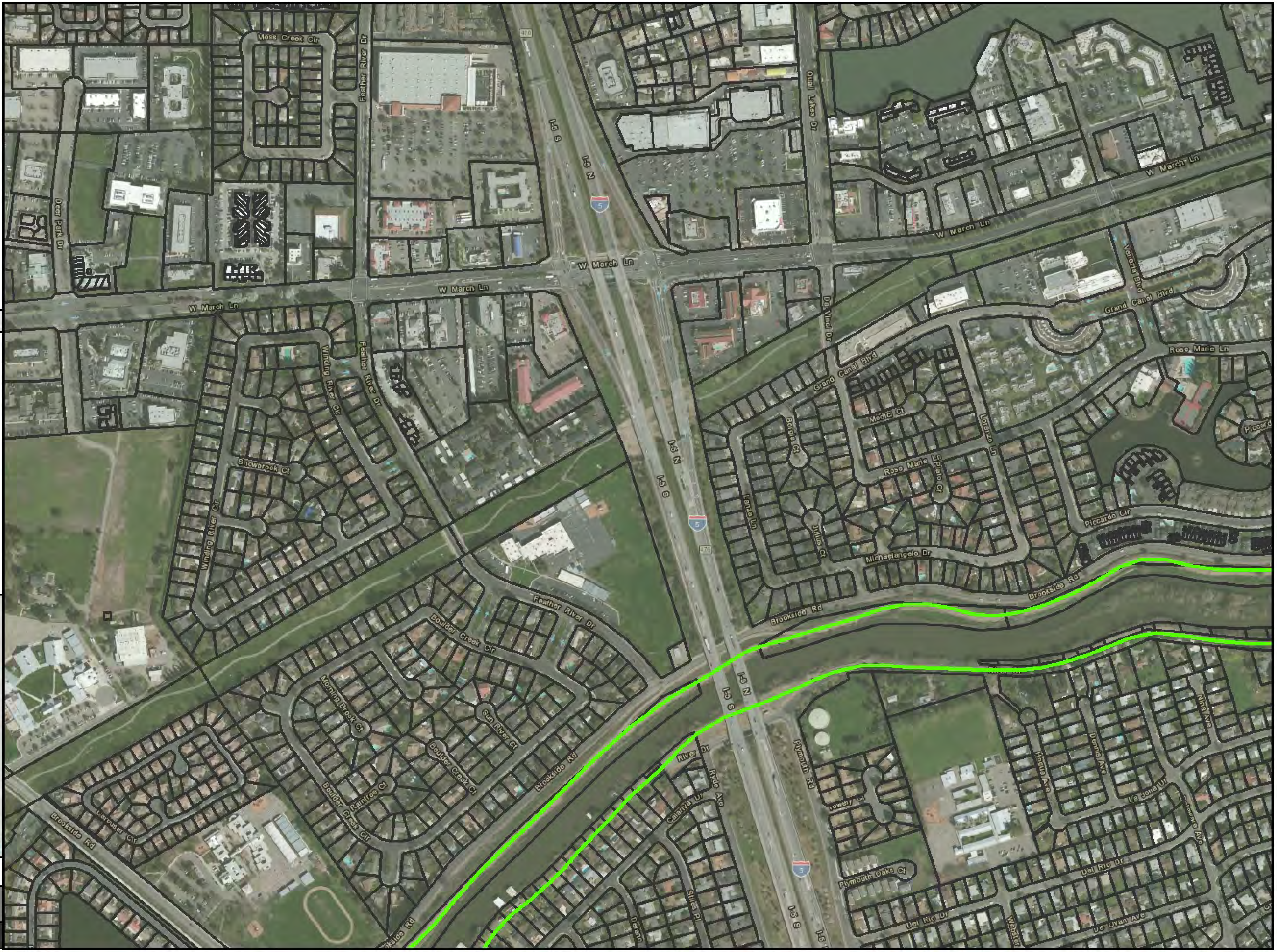
1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000



US Army Corps of Engineers®
Sacramento District

LOWER SAN JOAQUIN Feasibility Study

Alternative LS-7a

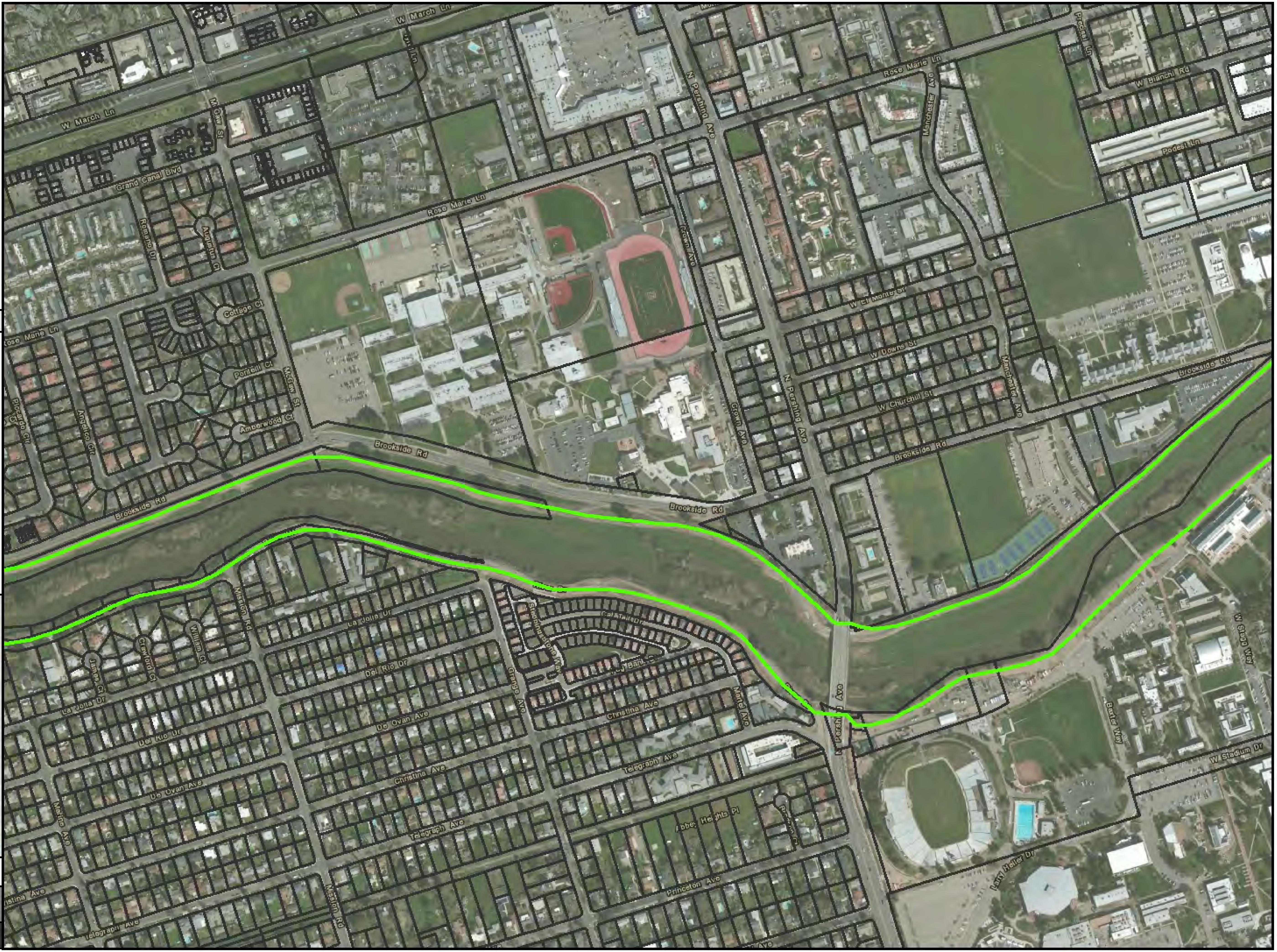
🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.

Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000



US Army Corps of Engineers®
Sacramento District

LOWER SAN JOAQUIN
Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

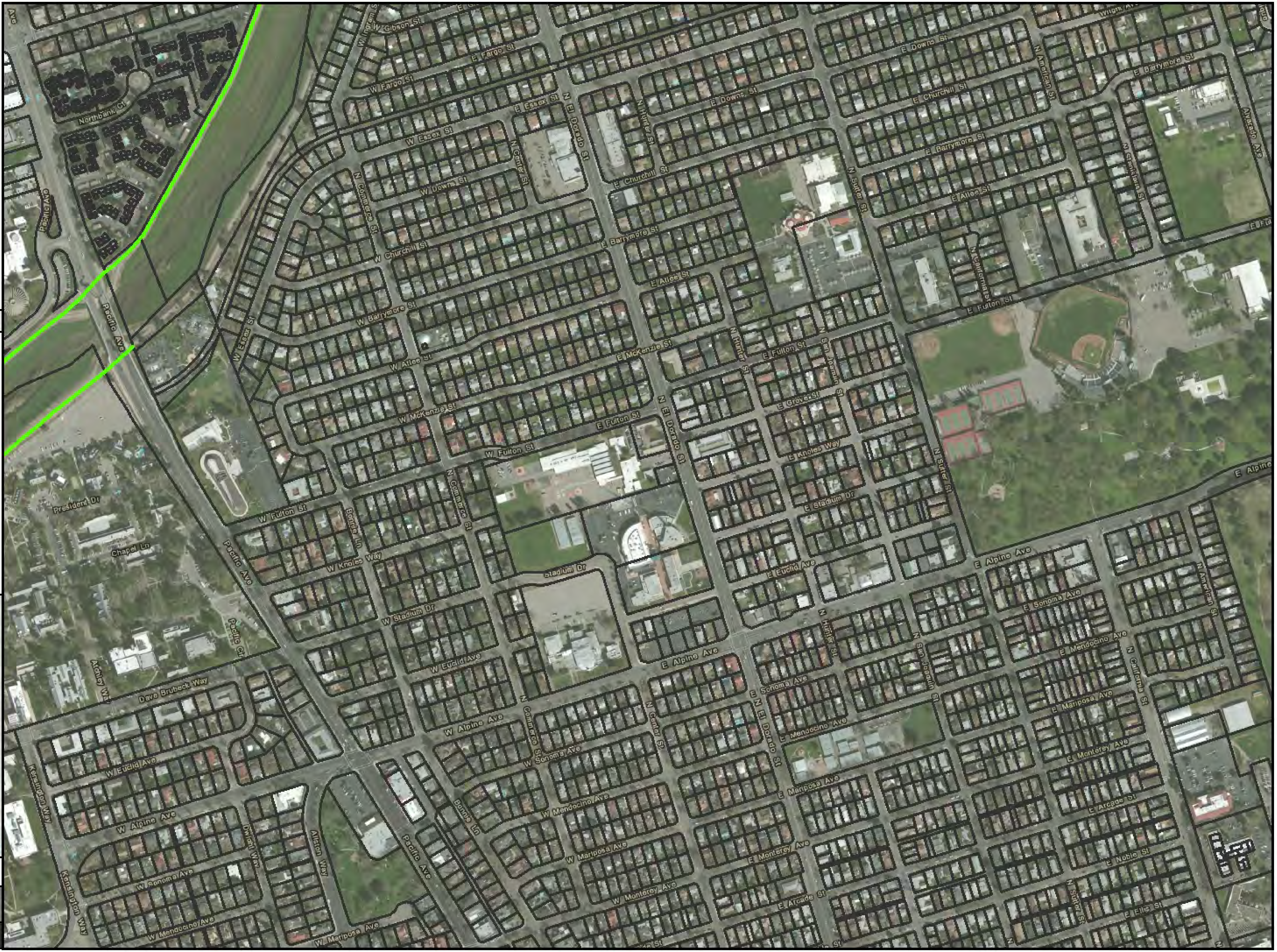
1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.

Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

Map Page 13 of 22



US Army Corps of Engineers®
Sacramento District

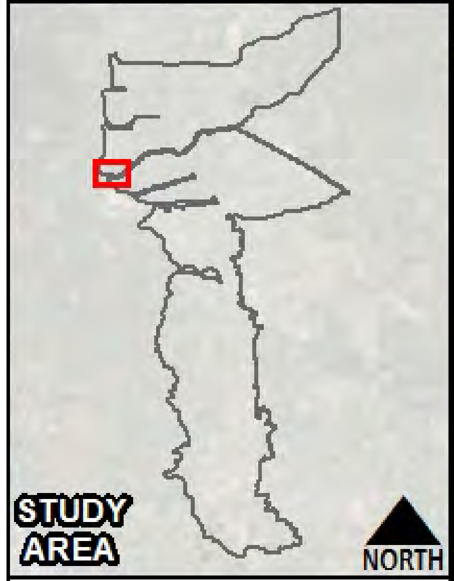
LOWER SAN JOAQUIN Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

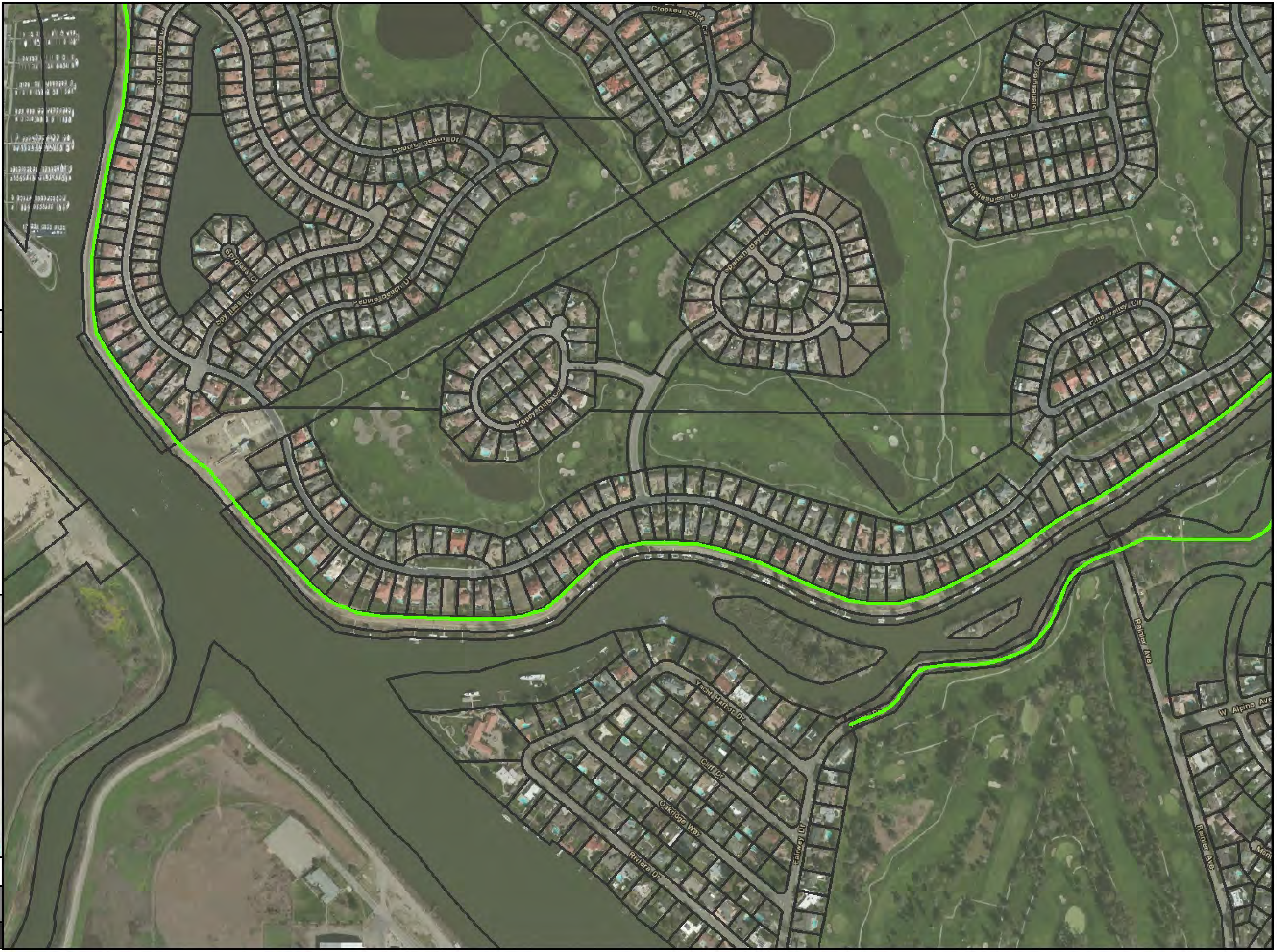
1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000



US Army Corps of Engineers®
Sacramento District

LOWER SAN JOAQUIN Feasibility Study

Alternative LS-7a

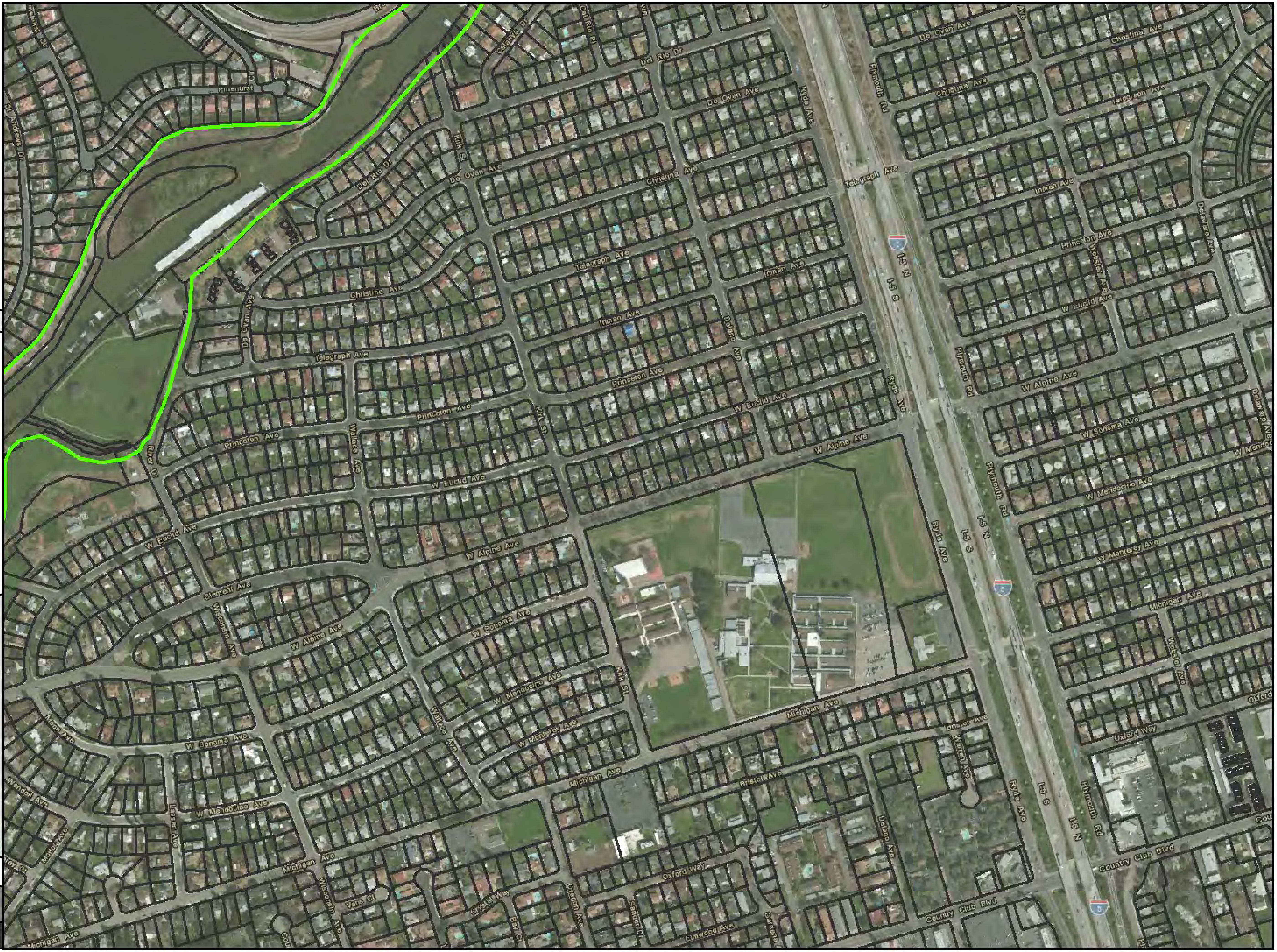
🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.

Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000



US Army Corps of Engineers® Sacramento District

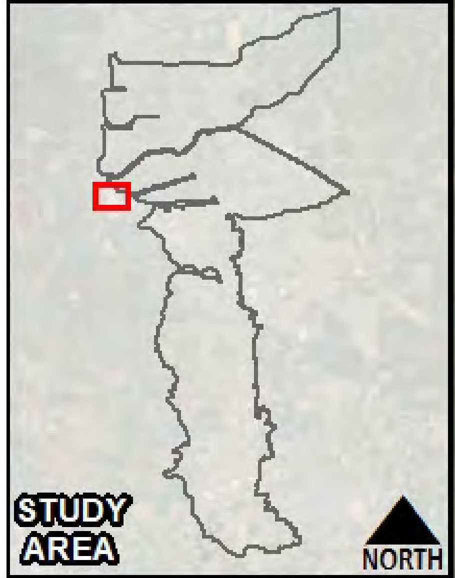
LOWER SAN JOAQUIN Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

Map Page 16 of 22

US Army Corps of Engineers®
Sacramento District

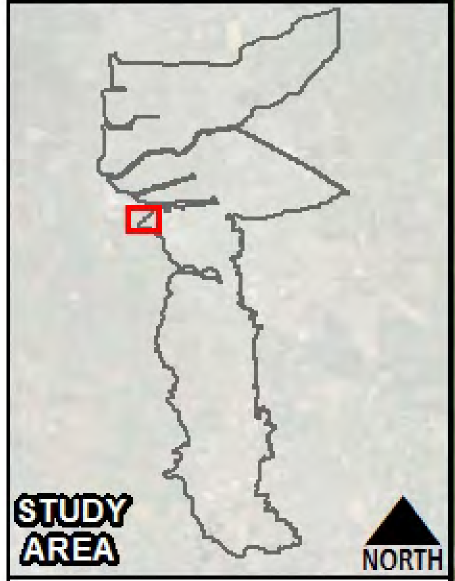
LOWER SAN JOAQUIN
Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

US Army Corps of Engineers®
Sacramento District

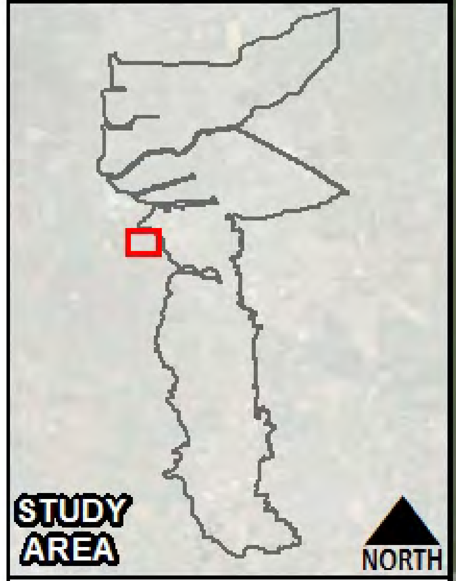
LOWER SAN JOAQUIN
Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

US Army Corps of Engineers®
Sacramento District

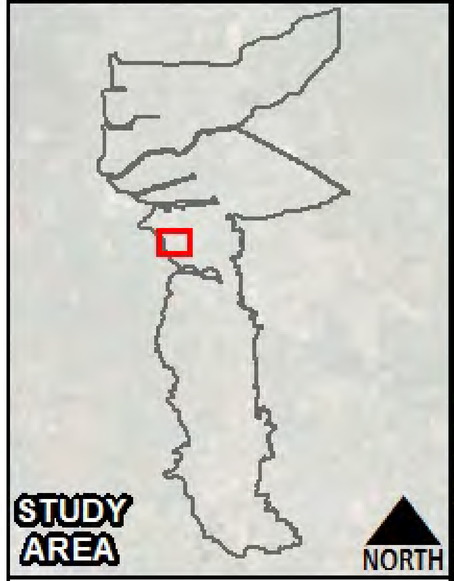
LOWER SAN JOAQUIN Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

US Army Corps
of Engineers®
Sacramento District

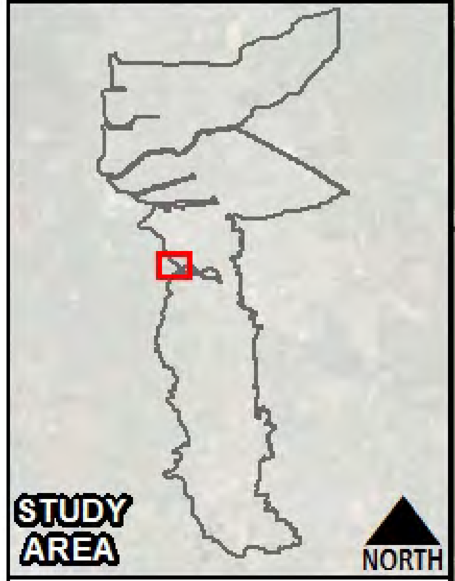
LOWER SAN JOAQUIN
Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

US Army Corps of Engineers®
Sacramento District

LOWER SAN JOAQUIN
Feasibility Study

Alternative LS-7a

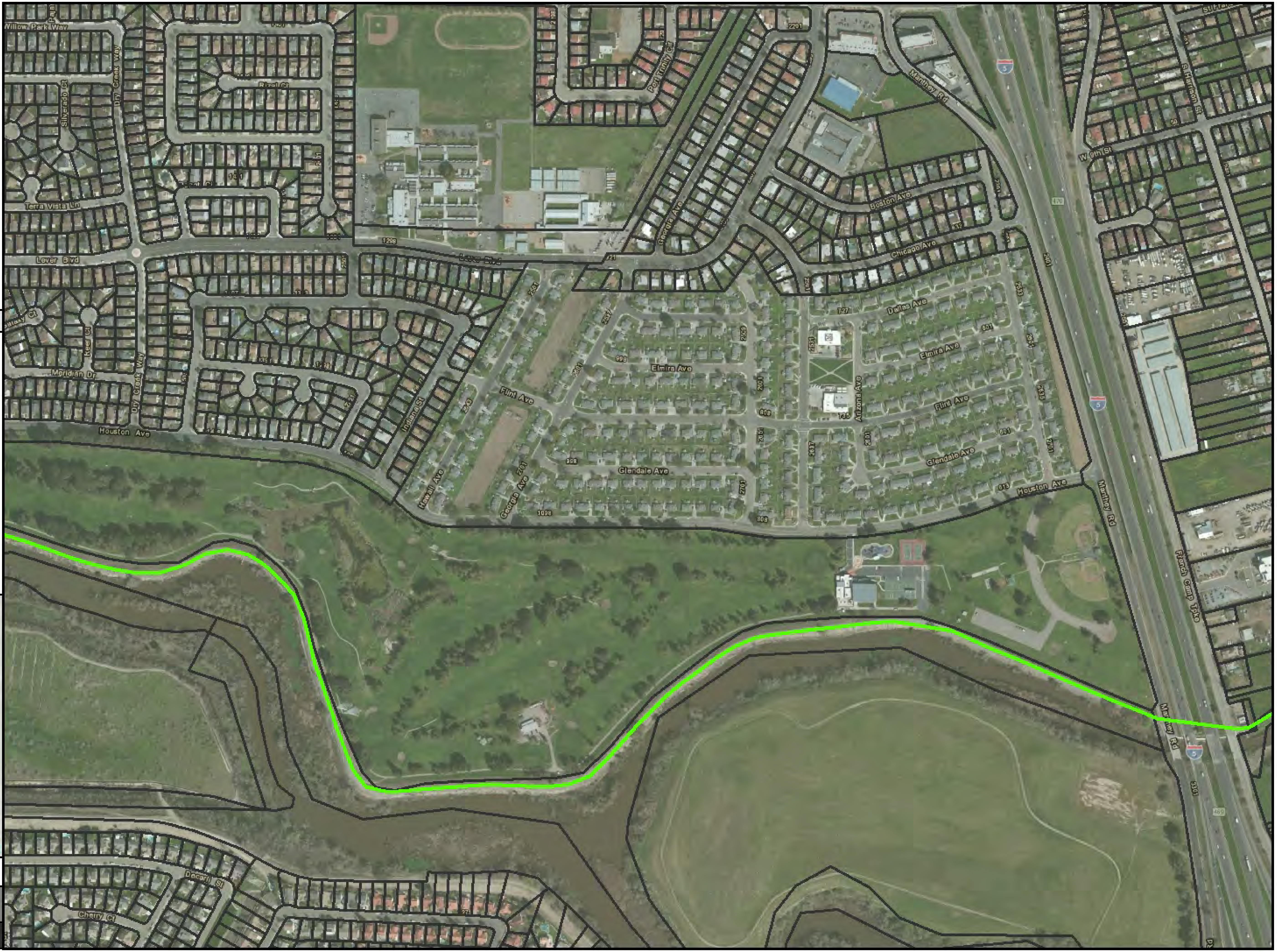
▬ Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.

Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000



US Army Corps of Engineers®
Sacramento District

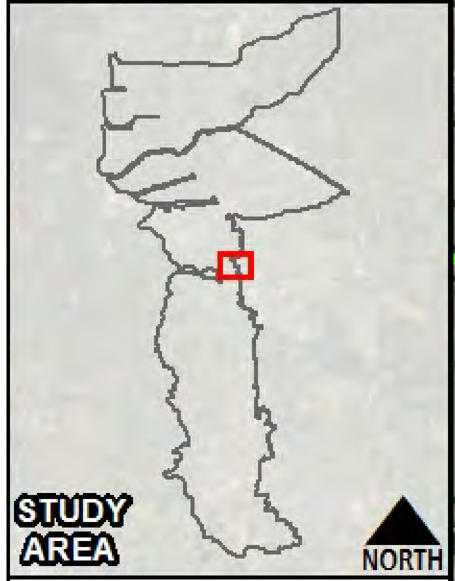
LOWER SAN JOAQUIN Feasibility Study

Alternative LS-7a

🟩 Proposed Project Footprint

1) This information is not intended as a substitute for a field survey by a licensed professional, or an application that requires legal or engineering accuracy.

2) Parcel boundary data is only a representation of ground features projected on to the Earth's surface by computer programs from raw data obtained from local government agencies and is not necessarily in whole, or in part, based upon any physical survey, study, or recording, professional or otherwise, of the covered properties.



Map Creator: Aaron Schlein, USACE, Sacramento District

Main Map Scale 1:5,000
Reference Map Scale 1:450,000

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDYIN SUPPORT OF THE INTERIM FEASIBILITY REPORT

APPENDIX D
ANNEX A
REAL ESTATE POLICY GUIDANCE LETTER NO. 31 REAL ESTATE SUPPORT TO CIVIL WORKS PLANNING RPARADIGN (3X3X3)

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
441 G STREET NW
WASHINGTON, D.C. 20314-1000

REPLY TO
ATTENTION OF:

CEMP-CR

JAN 10 2013

MEMORANDUM FOR SEE DISTRIBUTION

SUBJECT: Real Estate Policy Guidance Letter No. 31-Real Estate Support to Civil Works Planning Paradigm (3x3x3)

1. References.

a. Memorandum, CECW-CP, 8 February 2012, Subject: U.S. Army Corps of Engineers Civil Works Feasibility Study Program Execution and Delivery

b. ER 5-1-11, USACE Business Process, 1 November 2006

c. EC 405-1-04, Appraisal, 30 Dec 2003

d. ER 1105-2-100, Planning Guidance Notebook, 22 Apr 2000

e. ER 405-1-12, Chapter 12, Real Estate Roles and Responsibilities for Civil Works, Cost Shared and Full Federal Projects, Change 31, 1 May 1998

2. Purpose. In accordance with reference a, this memorandum provides interim policy and guidance for real estate efforts associated with feasibility studies under the new Planning Paradigm, "SMART Planning," and the 3x3x3 rule. In accordance with the 3x3x3 rule, all feasibility studies should be completed within three years, at a cost of no more than $3 million, utilize three levels of vertical team coordination, and be of a "reasonable" report size.

3. Background. Real Estate has been fully engaged in the implementation of the 3x3x3 by actively participating in each webinar, the planning modernization workshop, and serving as part of the HQ Transition Team. In accordance with references b-e, Real Estate involvement is essential to the development and implementation of any pre-authorization project. Paragraph 12-16 of reference e. outlines the significant topics that must be covered in a real estate plan (REP). The level of detail necessary to apply the requirements of real estate policy and guidance will vary depending on the scope and complexity of each project.

As outlined in Chapter 12, the minimum interests in real property necessary to support various types of projects must be identified. As projects are scoped at the beginning of the feasibility phase (via a Charette or other forum), it is essential that Real Estate become familiar with the project authority and purposes to make a determination of the minimum interests and estate(s), both standard and non-standard, necessary as projects are scoped and alternatives evaluated. If a non-standard estate will be needed, this should be discussed with MSC and HQ Real Estate as early as possible to ensure that the justification is sound and will serve the project purpose.

4. Policy. Typically, the attorney's preliminary opinion of compensability and gross appraisals are two areas that require more detail than may be readily available during the start of the feasibility phase, and are critical to determination of accurate estimates for real estate and total project costs. Due to the focus on 3 years or less for study duration, it will be essential for Real Estate to be adaptable and scale its requirements, decision making, and risk management in proportion to the significance of total project costs.

a. Gross Appraisals:

Specific to gross appraisals, EC 405-1-04 provides that cost estimates are utilized for preliminary planning of projects and in other cases, brief gross appraisals are acceptable. For purposes of the feasibility phase, the detail will vary as outlined below.

(1) For projects in which the value of real estate (lands, improvements, and severance damages) are not expected to exceed ten percent of total project costs (total cost to implement project), a cost estimate (or rough order of magnitude) will be acceptable for purposes of the feasibility phase.
(2) For projects in which the value of real estate (lands, improvements, and severance damages) do not exceed 30 percent of total project costs (total cost to implement project), a brief gross appraisal will be acceptable for purposes of the feasibility phase. A brief gross appraisal will follow format issued by Chief Appraiser.
(3) For projects in which the value of real estate (lands, improvements, and severance damages) exceed 30 percent of total project costs (total cost to implement project), a full gross appraisal will be prepared in accordance with the appraisal regulation and guidance provided by EC 405-1-04 and the Chief Appraiser.

b. Attorney's Opinion of Compensability:

As described in paragraph 12-17 of Chapter 12, utility/facility relocations may require preliminary attorney's opinions of compensability. While the practice of obtaining preliminary attorney's opinions of compensability provides a high degree of certainty with regard to project costs during the feasibility phase, such opinions can be time consuming and may provide more certainty than may be optimal for feasibility purposes when potential utility/facility relocation costs do not constitute a large percentage of total project costs. In support of the goals set out in the new planning paradigm described in reference a., Districts shall adhere to the following guidance:

(1) Where the estimated total cost to modify all project utility facility relocations, including the value of any additional lands that may be required to perform the relocations does not exceed 30 percent of estimated total project costs, the District Office of Real Estate shall, in lieu of an attorney s opinion of compensability prepare a real estate assessment. Such a real estate assessment, will address the following questions:

(a) Is the identified utility facility generally of the type eligible for compensation under the substitute facilities doctrine (e.g., school, highway, bridge, water and sewer systems, parks, etc.)

(b) Does the District have some valid data or evidence that demonstrates that it has identified an owner with a compensable interest in the property

If the answer to both questions is yes, then the District Office of Real Estate shall reflect the cost of providing a substitute facility in the Real Estate Plan (REP) and all other feasibility study cost estimates. If the answer to either or both questions is no, the District shall not reflect the cost of a substitute facility in the REP or other feasibility study cost estimates. However, the REP narrative should still include a discussion on the facility with results of analysis and project impact. For cost shared projects, the non-federal sponsor must be advised that the inclusion of substitute facilities costs in the REP or other use feasibility study estimates is for planning and budgeting purposes only and does not constitute a preliminary or final determination of compensability by the agency regardless of whether the cost of substitute facilities are reflected in the feasibility study documents. Using a real estate assessment does not eliminate the need to obtain a final attorney s opinion of compensability prior to execution of the PPA.

(2) Where the estimated total cost to modify all project facility relocations, including the value of any additional lands that may be required to perform the relocations, has public or political significance or the costs exceed 30 percent of estimated total project costs, a preliminary opinion of compensability shall be prepared for each owner s facilities. The level of documentation for each relocation item should be based on the significance of the relocation item to project formulation and estimated project costs.

Real Estate products, such as the REP, must be adaptable and scaled based on the project scope. Additionally, Real Estate must utilize the risk register to highlight areas where cost, schedule or uncertainty is greater in order to manage risk. Going forward, the Real Estate Division will continue to work closely with the Planning and Policy Division, Engineering and Construction Division, the Programs Integration Division and the National Law Firm on the Planning SmartGuide. This SmartGuide will provide more on procedures, tips, techniques and tools for

CEMP-CR
SUBJECT: Real Estate Policy Guidance Letter No. 31-Real Estate Support to Civil Works Planning Paradigm (3x3x3)

specific types of planning projects to aid in implementation of the new Planning Paradigm. All bulletins and updates on the SmartGuide can be found at: http://planning.usace.army.mil/toolbox/.

5. Duration. The policies stated herein will remain in effect until amended or rescinded by Policy Memorandums, Policy Guidance Letters, Engineers Circulars or Engineer Regulations.

FOR THE COMMANDER:

SCOTT L. WHITEFORD
DIRECTOR OF REAL ESTATE

DISTRIBUTION:
COMMANDER,
GREAT LAKES AND OHIO RIVER DIVISION (CELRD-PDS-R)
MISSISSIPPI VALLEY DIVISION (CEMVD-TD-R)
NORTH ATLANTIC DIVISION (CENAD-PD-E)
NORTHWESTERN DIVISION (CENWD-PDS)
PACIFIC OCEAN DIVISION (CEPOD-RE)
SOUTH ATLANTIC DIVISION (CESAD-PDS-R)
SOUTH PACIFIC DIVISION (CESPD-ET-R)
SOUTHWESTERN DIVISION (CESWD-ET-R)

CF:
COMMANDER,
DETROIT DISTRICT (CELRE-RE)
HUNTINGTON DISTRICT (CELRH-RE)
LOUISVILLE DISTRICT (CELRL-RE)
NASHVILLE DISTRICT (CELRN-RE)
PITTSBURGH DISTRICT (CELRP-RE)
MEMPHIS DISTRICT (CEMVM-RE)
NEW ORLEANS DISTRICT (CEMVN-RE)
ROCK ISLAND DISTRICT (CEMVR-RE)
ST. LOUIS DISTRICT (CEMVS-RE)
ST. PAUL DISTRICT (CEMVP-RE)
VICKSBURG DISTRICT (CEMVK-RE)
BALTIMORE DISTRICT (CENAB-RE)
NEW ENGLAND DISTRICT (CENAE-RE)
NEW YORK DISTRICT (CENAN-RE)
NORFOLK DISTRICT (CENAO-RE)

CEMP-CR
SUBJECT: Real Estate Policy Guidance Letter No. 31-Real Estate Support to Civil Works Planning Paradigm (3x3x3)

KANSAS CITY DISTRICT (CENWK-RE)
OMAHA DISTRICT (CENWO-RE)
PORTLAND DISTRICT (CENWP-RE)
SEATTLE DISTRICT (CENWS-RE)
WALLA WALLA DISTRICT (CENWW-RE)
ALASKA DISTRICT (CEPOA-RE)
HONOLULU DISTRICT (CEPOH-PP-RE)
JACKSONVILLE DISTRICT (CESAJ-RE)
MOBILE DISTRICT (CESAM-RE)
SAVANNAH DISTRICT (CESAS-RE)
ALBUQUERQUE DISTRICT (CESPA-RE)
LOS ANGELES DISTRICT (CESPL-RE)
SACRAMENTO DISTRICT (CESPK-RE)
FORT WORTH DISTRICT (CESWF-RE)
GALVESTON DISTRICT (CESWG-RE)
LITTLE ROCK DISTRICT (CESWL-RE)
TULSA DISTRICT (CESWT-RE)
CECC-R