Appendix A

Engineering

Post-Authorization Change Report

Sacramento River Bank Protection Project
Phase II – 80,000 Linear Feet

U.S. Army Corps of Engineers
Sacramento District

November 2019

This page intentionally left blank.

Figures

Tables

Acronyms and Abbreviations

ALT	Alternative
AR	Alternatives Report, 2009, Corps of Engineers
BO	Biological Opinion
CDFW	California Department of Fish and Wildlife
CEQA	California Environmental Quality Act
CFS	Cubic Feet per Second
Corps	United States Army Corps of Engineers
DDR	Design Documentation Report
Delta	Sacramento-San Joaquin Delta
DWR	California Department of Water Resources
EIS/EIR	Environmental Impact Statement/Environmental Impact Report
ER	Engineer Regulation
ESA	Endangered Species Act
ETL	Engineer Technical Letter
FRR	Field Reconnaissance Report
IWG	Interagency Working Groups
IWM	Instream Woody Material
JPSI	Justified Priority Site Inventory (previously called Reduced Array Plan)
MSWL	Mean Summer Water Level
NEPA	National Environmental Policy Act
NMFS	National Marine Fisheries Service
O&M	Operations and Maintenance
PACR	Post Authorization Change Report
PBPP	Programmatic Bank Protection Plan
PDT	Project Delivery Team
PPA	Project Partnership Agreement
PSI	Priority Site Inventory (previously called Initial Array Plan)
RM	River Mile
SAM	Standard Assessment Methodology
SRBPP	Sacramento River Bank Protection Project
SRFCP	Sacramento River Flood Control Project
USACE	United States Army Corps of Engineers
USFWS	United States Fish and Wildlife Service
VFZ	Vegetation Free Zone
WRDA 2007	Water Resources Development Act of 2007

This page intentionally left blank.

1.0 Purpose and Scope

1.1 Purpose

This Engineering Appendix is prepared as part of the Post Authorization Change Report (PACR) to the Sacramento River Bank Protection Project (SRBPP). The SRBPP was originally authorized in 1960 as bank protection work along the Sacramento River to protect the existing banks and levee elements of the Sacramento River Flood Control Project (SRFCP). Phase II was authorized in 1974, and provided 405,000 linear feet (LF) of bank protection. The Water Resources Development Act of 2007 (WRDA 2007) added 80,000 LF to Phase II. The PACR supports revisions to the SRBPP to add 80,000 LF of bank protection to Phase II as authorized. The PACR demonstrates that the SRBPP Phase II 80,000 LF is technically sound, is compliant with U.S. Army Corps of Engineers (Corps) policy, and meets environmental regulations.

The project purpose, as stated in the 1973 SRBPP, California-Second Phase, Report of the Chief of Engineers, is Flood Risk Management (FRM) to protect the existing levee system of the SRFCP. The report states that "each year streambanks and levees at additional unprotected locations throughout the Sacramento River Flood Control Project are subjected to erosion which carries away useful land, deposits sediment in downstream flood and navigation channels, damages valuable riparian vegetation and wildlife habitat, and ultimately threatens to destroy the integrity of the flood protection project and produce disastrous flooding." Thus, bank protection provides multiple beneficial effects.

To conform to Corps planning, engineering, and policy guidance, the project purpose should be associated with a basic Corps mission. Since bank protection supports the SRFCP, which was constructed primarily for flood control, Corps guidance as it applies to flood risk management projects is followed in this Engineering Appendix and PACR.

1.2 Approval

This Engineering Appendix defines the specific design concepts and establishes a baseline cost estimate for the 80,000 LF. This Engineering Appendix is prepared in accordance with Engineering Regulation (ER) 1110-2-1150, Engineering and Design for Civil Works Projects, and other Corps regulations. The designs are in compliance with Engineer Technical Letter (ETL) 1110-2-583: Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures. The earlier vegetation management ETL 1110-2-571 (10 April 2009) was used for the design effort. The ETLs expired and have been replaced byEP 1110-2-18. The designs herein continue to comply with USACE vegetation management policy.

Since this Engineering Appendix supports the PACR, it will be approved along with the PACR, likely at the MSC (Major Subordinate Command) level. The PACR forms the basis for the Project Partnership Agreement (PPA) between the Corps and the project non-Federal Sponsor, the State of California Central Valley Flood Protection Board (CVFPB).

1.3 Priority Site Inventory Concepts

This Engineering Appendix (EA) establishes design concepts for bank protection measures at 106 erosion sites (Note: Previous documents list 107 erosion sites; however, a discrepancy has been found in the 2009 Alternatives Report regarding the site at Natomas Cross Canal 3.0L and the site has been removed from this document) totaling approximately 80,000 LF. The erosion sites and corresponding designs were originally chosen during the development of the Corps' 2009 Alternatives Report (AR) prepared by Kleinfelder – Geomatrix. The designs in the AR were developed before the Corps vegetation management policies were established in ETL1110-2-571. Sixty-seven of the erosion sites were found to be in compliance with the ETL and would require minimum design changes, while the 39 remaining sites were found to be non- compliant. This EA retains the bank protection designs of the 67 compliant sites. For the 39 remaining sites, the designs were revised so that all sites are ETL compliant. The new design measures are set-back levees, adjacent levees, and stone protection with no vegetation. Two sites are revised designs of riparian and wetland benches.

The aggregate of bank protection designs at erosion sites, together with on-site and off-site environmental mitigation, present a prototypical plan known as the Priority Site Inventory (PSI). This plan provides the scope and guidelines for specific bank protection plans that will be developed and constructed once the PACR is approved and the PPA is signed.

Due to the dynamic and uncertain nature of erosion, sites needing bank protection are identified and selected on an annual basis. Since it is impossible to predict future erosion, the PSI provides a representation of what erosion repair will be constructed in future years. Therefore, the actual sites and bank erosion measures that will be constructed during the implementation phase will vary from the sites and measures presented in this PSI.

The PSI is a prototype for the SRBPP Phase II 80,000 LF, which is managed as a bank protection program. As a program, erosion sites are typically identified, monitored, and repaired on an annual basis. The description of the full process of monitoring erosion, development of bank protection designs and cost estimates, financing, environmental compliance and construction is provided in the PACR and is labeled the Programmatic Bank Protection Plan (PBPP). The PSI demonstrates how effective, fully mitigated bank protection may be achieved throughout the SRFCP system. Even though the erosion sites vary year to year, the PSI promotes a broad, system-wide perspective and avoids a piecemeal site-by site planning approach. Setback levees, for example, provide environmentally complete bank protection at one or more sites and can provide mitigation for other sites.

By including a variety of representative sites throughout the Sacramento River system, the PSI demonstrates that effective bank protection measures may be applied to any sites throughout the project area. It further demonstrates that bank protection may be achieved in compliance with EP 1110-2-18 and other design guidelines.

The erosion protection design process included early consideration of environmental impacts and mitigation. This is important because erosion protection measures can potentially impact state and Federally listed fish species. Additionally, in light of EP 1110-2-18, bank protection may result in the loss of high value riparian vegetation. To avoid or mitigate for losses, the bank protection design process included modeling changes to fish habitat and accounting for losses of riparian vegetation. The design of bank protection at actual sites was a collaborative team effort between engineering and environmental disciplines. Bank protection designs were tested against the Standard Assessment Methodology (SAM) model to determine a design's effect on several focal fish species, including state and Federal-listed threatened and endangered species that may occur in the SRBPP area. Effects to riparian vegetation were avoided or mitigated on-site, or mitigated off-site. Environmental impacts are discussed in the programmatic Environmental Impact Statement and Environmental Impact Report (EIS/EIR) that accompany the PACR.

Through the PSI, the Engineering Appendix provides conceptual designs, drawings, real estate requirements, and costs for bank protection. The cost estimates include preliminary real estate costs and environmental mitigation costs, and serve as a representation of what the 80,000 LF of bank protection might cost.

The PSI is also used to determine economic feasibility of the various economic sub-basins as discussed in the PACR main report. After determining the feasibility of each basin, a Reduced Array Plan (RAP) is developed. The RAP consists of only LF within economically feasible sub-basins. Costs from the RAP are extrapolated on a per linear foot basis to 80,000 linear feet to obtain a project cost for the SRBPP.

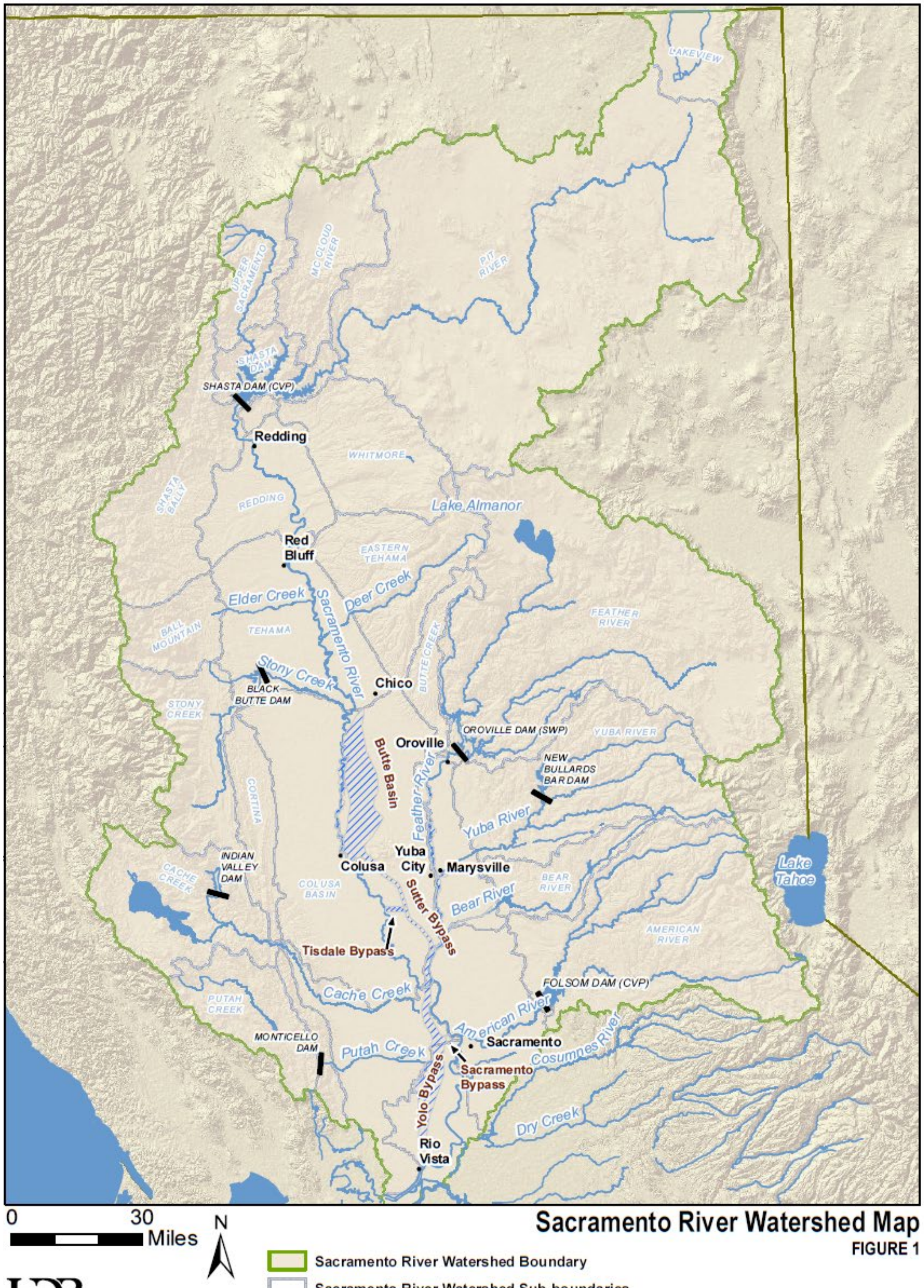
1.4 Location

The SRBPP program area extends south-to-north along the Sacramento River from the town of Collinsville at River Mile (RM) 0, upstream to Chico at RM 194, and includes reaches of lower Elder and Deer Creeks. The SRBPP program area also includes Cache Creek, the lower reaches of the American River (RM 0-23), Feather River (RM 0-61), Yuba River (RM 0-11), and Bear River (RM 0-21), as well as portions of Threemile, Steamboat, Sutter, Miner, Georgiana, and Cache Sloughs.

Sacramento River Watershed

The Sacramento River Watershed drains the northern part of the Central Valley into the middle and lower reaches of the Sacramento River (**Figure 1**). The Sacramento River is approximately 327 miles long and drains over 27,000 square miles of land. The upper watershed of the Sacramento River region includes the drainages above Lake Shasta and Lake Oroville. The valley drainages include the upper Colusa and Cache Creek watershed on the west side of the valley, and the Feather River and American River watersheds on the east side of the valley.

Land uses in the Sacramento River Basin are principally agricultural, silvicultural, and open space, with urban development focused around the City of Sacramento. Other urban developed areas include Marysville, Davis, Woodland, Vacaville, Dixon, Redding, Chico, Yuba City and various Sierra Nevada foothill towns. Agriculture is the dominant land use followed by urban development. About 2,300 mi^2 in the watershed are devoted to agricultural use.



Sacramento River Watershed Map

FIGURE 1

HDR

ONE COMPANY | *Many Solutions*

COE JV O2Sac Riv Bank EIR/EIS Project No. 86356

1.5 Scope

Functional Scope

As described in the 1973 Chief's Report, the SRBPP is a long-range program of bank protection and levee setbacks to protect the existing banks and levees within the SRFCP. Bank protection in the form of erosion repairs will be either on the waterside berm or the levee if there is no berm. Critical areas must continue to be protected to maintain the safety of the SRFCP. The SRBPP does not specifically include other levee corrective measures such as seepage and cutoff walls, slope stability, or raising low spots along the levee crests. However, these may be included to meet USACE standards, such as with the construction of a setback levee. Incidental improvement in levee seepage conditions is possible if the repair results in a lengthening and preservation of the levee's seepage path.

Geographic Scope

The geographic scope includes the banks and levees of the SRFCP. The SRFCP is along the Sacramento River from Elder Creek near Tehama to its confluence with the San Joaquin River in the Delta. The SRFCP includes a number of tributaries, sloughs, and bypass channels (**Figure 2**).

In 1982, Congress specifically authorized extension of the SRBPP upstream of the SRFCP levee system from RM 176 left/184 right to RM 194 (public law 97-377).

As summarized below, the SRBPP is separated into 4 geographic locations: 1A, 1B, 2, and 3. See Figure 2 of the PACR and refer to the EIS/EIR for further detail on these regions.

- **Region 1a -** Within Region 1a, the Sacramento River flows below Isleton (River Mile (RM) 20) into the Delta, forming a distribution network of sloughs and channels.
- **Region 1b -** Region 1b includes the mainstream Sacramento River from Isleton (RM 20) in the Delta, upstream past the city of Sacramento, to the Feather River confluence (RM 80) at Verona. Region 1b also includes the lower American River from the confluence with the Sacramento River upstream to RM 13, Natomas East Main Drain, Natomas Cross Canal, and Coon Creek Group Interceptor Unit 6.
- **Region 2 -** Within Region 2, the mainstream Sacramento River flows from Colusa (RM 143) downstream of the Colusa Bypass to the confluences with the Feather River and Sutter Bypass at Verona (RM 80). Feather River and its tributaries in Region 2 extend from the confluence with the Sacramento River to RM 31 at the Western Canal Left Bank.
- **Region 3 -** Region 3 includes the Sacramento River downstream of Chico Landing (RM 194) to Colusa (RM 143).

1.6 Sacramento River Flood Control Project

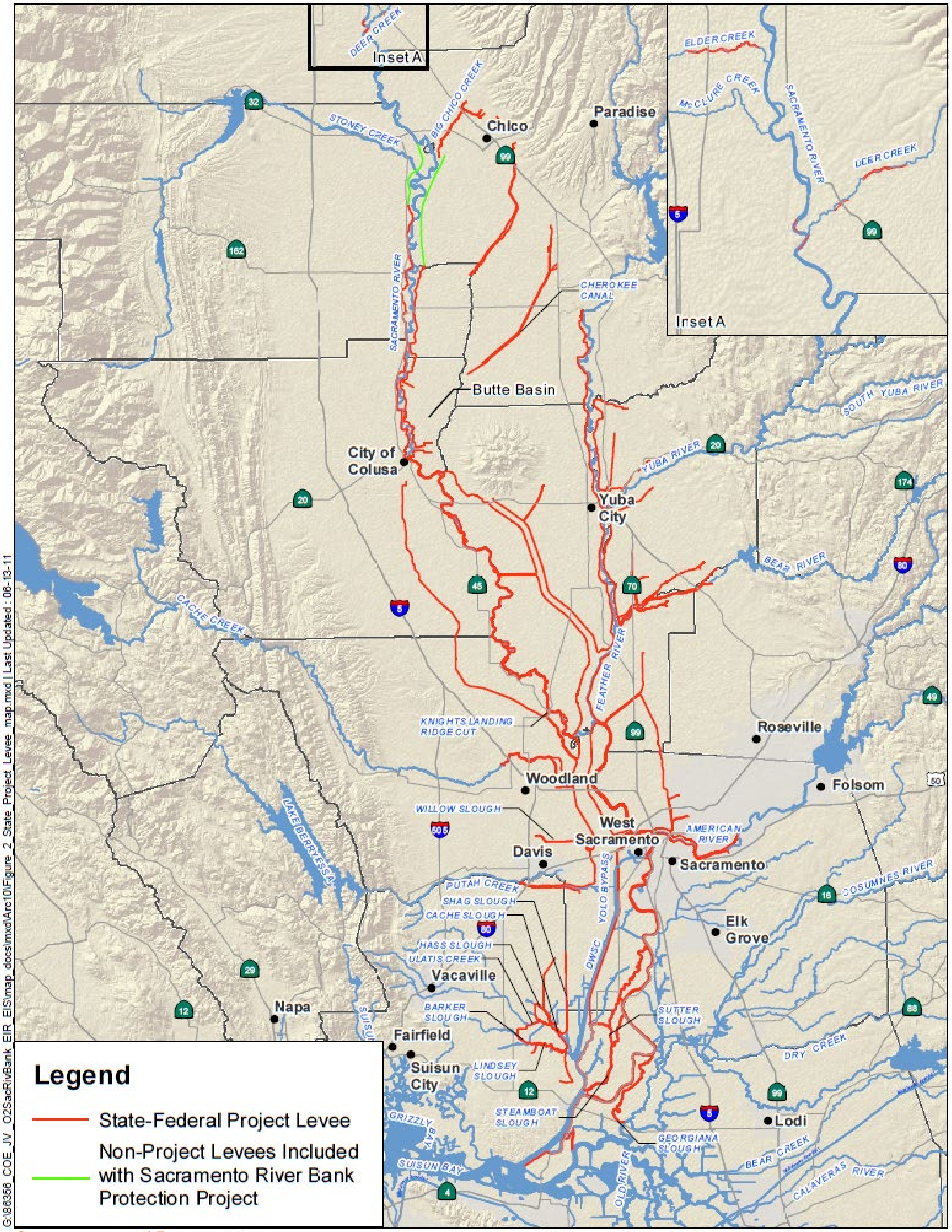
The SRFCP was authorized by the Flood Control Act of 1917 (Public Law 64-367) and includes a system of levees, weirs, pumping plants, and bypasses designed to safely convey Sacramento River and tributary flood flows. The project provides protection to about 2.1 million acres of highly productive agricultural land, as well as protection to the cities of Sacramento, West Sacramento, Yuba City, Marysville, Colusa, Gridley, and other communities. The SRFCP is operated and maintained by the Department of Water Resources, State of California. The Corps provides assurance that the project is maintained to Federal standards. The flood management system responsible for protecting these resources in the Sacramento Valley has expanded with the addition of projects, such as the Sacramento River and Major and Minor Tributaries Flood Control Project, the American River Common Features Project and the Sacramento River Flood Control System

Evaluation Project. This project includes the following major features (see **Figure 2**):

- Approximately 1,300 miles of levees along the Sacramento River extending from River Mile (RM) 0 at Collinsville to Chico Landing, RM 194, distributary sloughs, the lower reaches of the major tributaries (American, Feather, Yuba and Bear Rivers) and additional minor tributaries;
- The Moulton, Colusa, Tisdale, Fremont, and the Sacramento Flood Overflow Weirs; and
- The Butte Basin and Sutter and Yolo Bypasses and Sloughs.

1.7 Datum

All data provided in this report is based on the NAVD88 vertical datum. The North American Vertical Datum of 1988 (NAVD88) was established by the National Geodetic Survey. The datum was based on an adjustment of high order differential leveling throughout the United States with the final adjustment completed in 1988. The datum was based on adjusting the leveling network to a single benchmark near the great lakes. This datum is currently supported by the NGS. [This section added to final appendix by SPK.]



0 15 Miles

1 in = 15 miles

N

Sacramento River Flood Control Project Levees

FIGURE 2

HDR ONE COMPANY | *Many Solutions*™

COE JV O2Sac Riv Bank EIR/EIS Project No. 86356

2.0 Programmatic Bank Protection Plan Overview

Erosion along the Sacramento River is a dynamic, unpredictable process that demands flexibility to adapt to changing conditions. An PSI, rather than a typical specific plan and design, is necessary to provide the flexibility needed to respond to the variable characteristics of erosion. This PSI will be followed up by a series of specific, supplemental Design Document Reports (DDRs) that will provide a basis for design of bank protection at sites identified through the site selection process.

The PSI is representative of how and where the added 80,000 LF of bank protection will be constructed. The plan establishes bank protection measures at each of 106 erosion sites from the AR, totaling 77,436 LF, which approximates the 80,000 LF authorized in WRDA2007.

2.1 SRBPP Phase II Program

The SRBPP Phase II is a program developed for bank and levee rehabilitation responding to erosion problems that are identified in the field during annual reconnaissance and site selection. Erosion problems occur throughout the Sacramento River Flood Control System and are unpredictable. A plan of definitive bank protection cannot be developed due to the unpredictable nature of erosion. Therefore, an AIP is developed. The PSI provides a realistic representation of the measures, real estate requirements, construction footprint and costs for the 80,000 LF.

2.2 Priority Site Inventory Defined

The PSI identifies 106 actual erosion sites on the SRFCP that total roughly 80,000 LF. These 106 sites are used as a representative sample of what the Phase II SRBPP will have to address during implementation. Out of a pallete of bank protection measures developed by the Corps, one measure is applied to each site. A conceptual design and cost estimate is then developed for each site.

2.3 Priority Site Inventory Development Process

Development of the PSI follows a rational process to achieve a technically sound and complete plan. Measures are applied consistently throughout the system taking into account the unique characteristics of each site. The process builds on work already accomplished by the Corps, as presented in the AR. The AR did not define a vegetation free zone. Delineation of this zone is needed to develop bank protection that is in conformance with vegetation management policy. The PSI was developed taking into account the vegetation free zone so that as much on-site environmental mitigation as possible is included.

This process was done by a multidisciplinary team that included environmental specialists as well as engineers. A major aspect of this plan is avoiding or mitigating negative impacts to fish habitat. The Sacramento River and tributaries are spawning and juvenile rearing habitat for a number of migratory fish species listed under the Federal Endangered Species Act. The process includes evaluating the bank protection measures at sites using the SAM model, which determines gains and losses to fish habitat. The SAM model, as well as many of the bank protection measures discussed below, was developed through consultations between the Corps and the U.S. Fish and Wildlife Service and the National Marine Fisheries Service under provisions of the Endangered Species Act. These consultations were carried out for bank protection actions previous to the current 80,000 LF.

2.4 Implementation Phase

During the implementation phase, sites to receive bank protection will be identified on an annual basis. Geotechnical analyses, hydraulic analyses, and surveys will be conducted, and bank protection measures identified. Supplemental environmental documentation will be required, and a supplemental DDR will be prepared, as well as plans and specifications. Implementation is further discussed in **Section 2.3.3 Site Selection and Implementation** of the **PACR** and its **Site Selection and Implementation Procedure Appendix**, **Appendix B**.

3.0 Erosion Protection Measures

A number of erosion protection measures have been developed by the Project Development Team (PDT). A range of measures is formulated to meet the varying erosion and mitigation requirements at a variety of sites throughout the system. The measures may be implemented at a given erosion site. The measures are described in detail in the main report of the PACR, Engineering Appendix, and the
EIS/EIR. The EIS/EIR demonstrates how bank protection would be applied given a number of different levee and bank conditions.

Table 1 gives a summary/comparison listing the details associated with each repair measure. These measures were revised and expanded from what are listed as alternatives in the AR. For reference, **Table 2** lists the measures in the AR and matches them with the measures of this Engineering Appendix.

Table 1 - Repair Measures Summary

Details	Measure 1: Setback Levee	Measure 2 : Bank Fill Stone Protection with No On-site Vegetation	Measure 3: Adjacent Levee	Measure 4a: Riparian Bank with Revegetation and IWM above Summer/Fall Waterline	Measure 4b: Riparian Bench with Revegetation and IWM above and below Summer/Fall Waterline	Measure 4c: Riparian and Wetland Benches with Revegetation	Measure 5: Bank Fill Stone Protection with On-Site Vegetation
Revegetation Outside of VFZ				X	X	X	
Riparian Bank/Bench			X	X	X	X	
IWM above Summer/Fall Waterline				X	X		
IWM below Summer/Fall Waterline					X		
Installation of Stone Protection		X		X	X		x
Adjacent Levee Construction			X				
Setback Levee Construction	X						
Existing Levee Breach	X		X				

Table 2 – AR Report and EA Erosion Repair Measures Comparison

2009 Alternatives Report	Phase II 80,000 Linear Feet Engineering Documentation Report
Alt 1: No Action	No Action
Alt 2: Design 1 – Bank fill rock slope with revegetation	Measure 2: Bank Fill Stone Protection with No On-site Vegetation Measure 5: Bank Fill Stone Protection with On-Site Vegetation
Alt 3: Design 1 with Site Specific Modification	
Alt 4: Design 2 – Low riparian bench with revegetation and large wood material enhancements *above* the summer/fall waterline recommended for sites upstream of RM 30	Measure 4a: Riparian Bank with Revegetation and IWM above Summer/Fall Waterline
Alt 5: Design 2 with Site Specific Modification	
Alt 6: Design 3 – Low riparian bench with revegetation and large woody material enhancements above and below the summer/fall waterline recommended for sites upstream of RM 30	Measure 4b: Riparian Bench with Revegetation and IWM above and below Summer/Fall Waterline
Alt 7: Design 3 with Site Specific Modification	
Alt 8: Design 4 – Delta smelt design – low riparian and wetland benches with revegetation recommended for sites downstream of RM 30	Measure 4c: Riparian and Wetland Benches with Revegetation
Alt 9: Design 4 – With Site Specific Modification	
Alt 10: Setback Levee	Measure 1: Setback Levee
(No Alternative)	Measure 3: Adjacent Levee

.

4.0 Plan Development Details

A rigorous process was conducted to evaluate each erosion site and assign a revised repair measure if required. Erosion repairs, as described in the AR, must be vetted for vegetation management compliance. If the repair alternative is non-compliant then a new repair measure must be defined.

This plan development defines five erosion protection measures and the process for which a protection measure is assigned to an individual site, or in some instances a group of sites. The process takes into consideration the geographical location, the quantity and quality of existing riparian and riverine aquatic habitat, channel hydraulics, and major structures (houses, pumping plants, etc.) adjacent to the landside toe of the levee. The overall goal is to balance programmatic cost with retaining existing habitat and reduce potential mitigation for fish and wildlife.

4.1 Develop Erosion Site Cross Sections

Step 1: Site selection

Most but not all erosion sites presented in the AR were identified in the annual Field Reconnaissance Report (FRR) prepared by the Corps in 2007. Each year, following the 1997 storm events, personnel from the Sacramento District Corps and the California Department of Water Resources (DWR) Division of Flood Maintenance (acting on behalf of the local sponsor, the Central Valley Flood Protection Board) conduct a field reconnaissance review of the Sacramento River Flood Control System. The primary purpose of the review is to monitor and document the condition of the previously identified erosion sites, inventory any new erosion sites, and identify critical erosion sites that appear to be an imminent threat to the structural integrity of the flood control system.

The sites are geographically distributed throughout the SRFCP area and are representative of varying conditions found in different reaches throughout the project. Sites are along the Sacramento River main-stem, Delta sloughs, and along a number of tributaries. These include Bear River, Feather River, Cache Creek, Georgiana Slough, Yolo Bypass, Cherokee Canal, Cache Slough, Deep Water Ship Channel, Deer Creek, Elder Creek, Knights Landing Ridge, Cut Lower American River, Natomas Cross Canal, Steamboat Slough, Sutter Slough, Willow Slough and Yuba River.

The AR recommended a selected repair alternative for each of the 106 erosion sites from a group of ten bank protection alternatives. The ten alternatives provided in the AR include the four designs proposed by the Corps Sacramento District (described in the Framework Memo) which is referred to in the AR as designs 1 through 4. Each of the four designs included an additional alternative with a site specific modification, as well as a no action and setback alternative. A description of each alternative can be found in the AR. The AR also includes an aerial view exhibit of each erosion site which provides the location of critical points such as the upstream and downstream limits, existing encroachments, location of cross section measured during site reconnaissance and location of site photo. In addition, the AR includes a conceptual repair cross section which provides the erosion surface at the most critical point and the selected repair alternative. The AR evaluated a minimum of three alternatives for most sites while considering the no action alternative for all sites. Each site was evaluated based on the following criteria:

- General Site Description
- Levee and Bank Conditions
- Existing Environmental Conditions and Constraints
- Site Features and Improvements
- Site Access
- Evaluation of Bank Protection Alternatives
- Input from Agencies
- Recommended Alternative, Conceptual Design and Preliminary Cost
  - For a more detailed description of the AR evaluation criteria refer to EA Appendix 2 Civil Design with MCACES Estimate.

Step 2: Site Reconnaissance

Site evaluation includes establishing the landside and waterside toe, delineating levee geometry such as levee crown elevation and width, side slopes, waterside levee geometry, i.e. benches and water surface elevation. Site evaluation also includes establishing the quantity and quality of vegetation and identifying any major structures that might be impacted by a repair alternative.

Sources of information:

- Alternatives Report – 80,000 linear feet (106 Sites) Sacramento River Bank Protection Project, 2009: This report and associated field notes provided existing levee geometry, mean summer water surface elevation and upstream and downstream existing levee and bank geometry conditions.
- Sacramento River HEC-RAS model. A steady state HEC-RAS model of unverifiable origin and purpose likely based on the USACE Comprehensive Study UNET model geometry. Despite the questions about this model, a spot check of the model indicates that the model can be used for the purposes of this programmatic document. However, the model should not be used in any future work efforts on this or other projects. The spot check and the results are documented in a separate memorandum of record with the subject of "Sacramento River Bank Protection Project, Phase II 80,000 LF, PACR/EA/EIS/EIR, Sufficiency of Hydraulic Model Used". The spot check indicates the geometry is likely similar or identical to the Comprehensive Study geometry. The origin of the hydrology of this model is not known and it is not certified. However this analysis does not use the hydrology information, only the model geometry.

- Revetment Database: US Army Corps of Engineers, 2007. Sacramento River Bank Protection Project Database. US Army Corps of Engineers, Sacramento District. The U.S. Fish and Wildlife Service (USFWS) and the National Marine Fisheries Service (NMFS) issued biological opinions (BOs) in 2001, under their jurisdiction pursuant to the Endangered Species Act (ESA), in response to the threatened and endangered status of several fish species that use the SRBPP area for habitat or passage.

  In early 2002, an interagency working group (IWG) comprised of representatives from the Corps, the California State Reclamation Board (the local sponsor for the SRBPP), Department of Water Resources (DWR), USFWS, NMFS, and California Department of Fish and Game (CDFG) developed protocols for collecting revetment data in the SRBPP act area (USFWS 2002). The IWG was established in 2001 to support the work of the SRBPP. Its primary goals are to identify, evaluate, design, and endorse conservation measures that are consistent with biological opinions.

  Development of levee and bank geometry was completed using the above sources and is presented in **Subappendix A2 Civil Design with Costs Estimates**. No additional topographic surveys or geotechnical evaluations were completed for this report, although these items may be required during design.

Step 3: Overlay Vegetation Free Zone (VFZ) on Site Alternatives

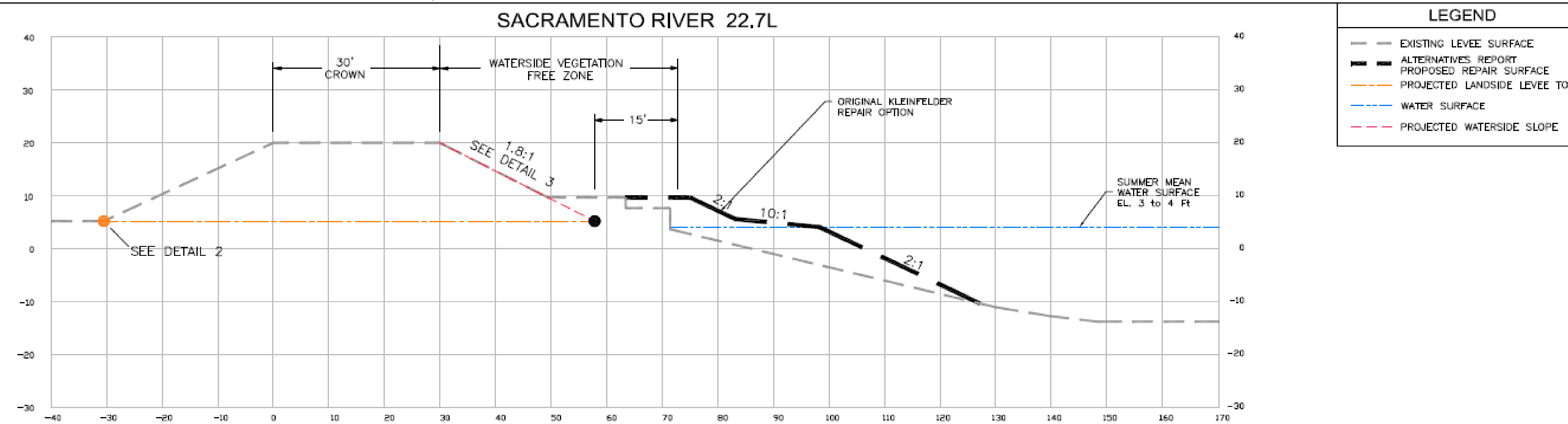
Using the procedures outlined in the PACR and the EIS/EIR, the levee and bank geometry and critical structure were defined for each site. The critical structure must be established to determine the VFZ. The VFZ is established by identifying the landside and waterside levee toes, then extending 15 feet outward from each toe to establish the VFZ boundary. The waterside levee toe is established by projecting the landside levee toe horizontally to the point where it intersects the projected 3:1 waterside levee slope. The entirety of the levee surface within this boundary would be prohibited from planting as defined in the ETL. The vegetation free zone is then overlaid on the levee erosion site **(Figure 4).**

Step 4: Retain Acceptable Sites

Each erosion site from the AR was evaluated for ETL compliance. With the VFZ defined for each site it was determined which of the repair alternatives, as presented in the AR, proposed planting within VFZ. The initial analysis revealed that some repair alternatives were clearly *not* within the VFZ while others were clearly within the VFZ. But a number of the sites were marginally encroaching into the VFZ. Therefore the initial analysis revealed that 34 sites were compliant, 33 sites marginally encroached and 39 sites would require an alternative repair measure. Upon further analysis of the marginal sites, it was determined that the repair as presented in the AR could be slightly modified by reducing the planting area which would make the site compliant with the ETL. As a result, only 39 sites would require a revised Alternative Repair Measure.

Figure 4- Conceptual Cross Section- Vegetation Free Zone Analysis

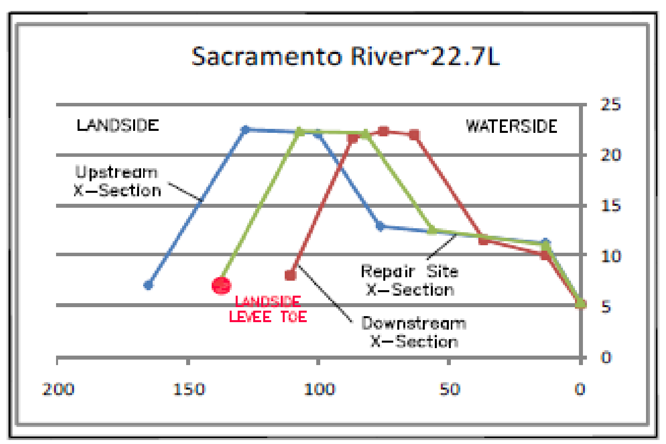
DETAIL 1



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.4
13.2	11.1
56.8	12.6
82.1	22.0
107.6	22.3
138.1	7.5

KEY DATA FOR DETERMINING LANDSIDE TOE

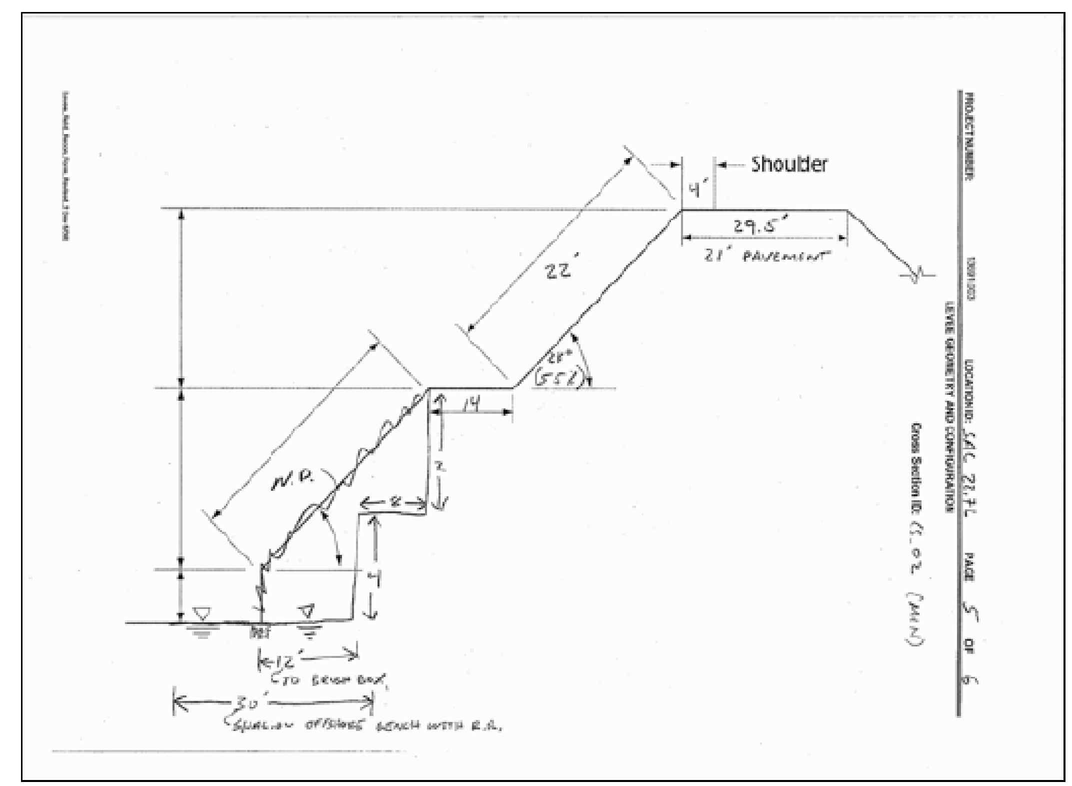
Comp Study Levee Crown Elevation	22.3
Comp Study Landside Toe Elevation	7.5
Comp Study Difference	14.8

Alternatives Report Analysis Levee Crown Elevation	20.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	5.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



This page intentionally left blank.

4.2 Bank Protection Measures Selection

Step 5: Assign Erosion Protection Measures to Non-ETL Compliant Sites

The AR used different versions of Alternative 2 Bank Fill Stone Protection Slope and Alternative 4 a, b, and c, banks with vegetation and in-stream woody material. For the sites which need a viable alternative, this evaluation attempted to apply Alternative 1 Setback Levee and Alternative 3 Adjacent Levee.

After completing the ETL compliance analysis, 39 of the 106 erosion sites required a revised alternative repair measure. Certain criteria were used to assign Alternative 1 or 3 to an erosion site. These criteria include the quantity and quality of the existing vegetation, the amount of existing waterside vegetation based on the riprap database, channel hydraulic impacts and landside structures. Of the 39 sites assessed, ten of the sites were assigned Alternative 1 – Setback Levee, and 16 sites were assigned Alternative 3 – Adjacent Levee. Refer to **Subappendix A2 Civil Design with Cost Estimates** for a detailed discussion.

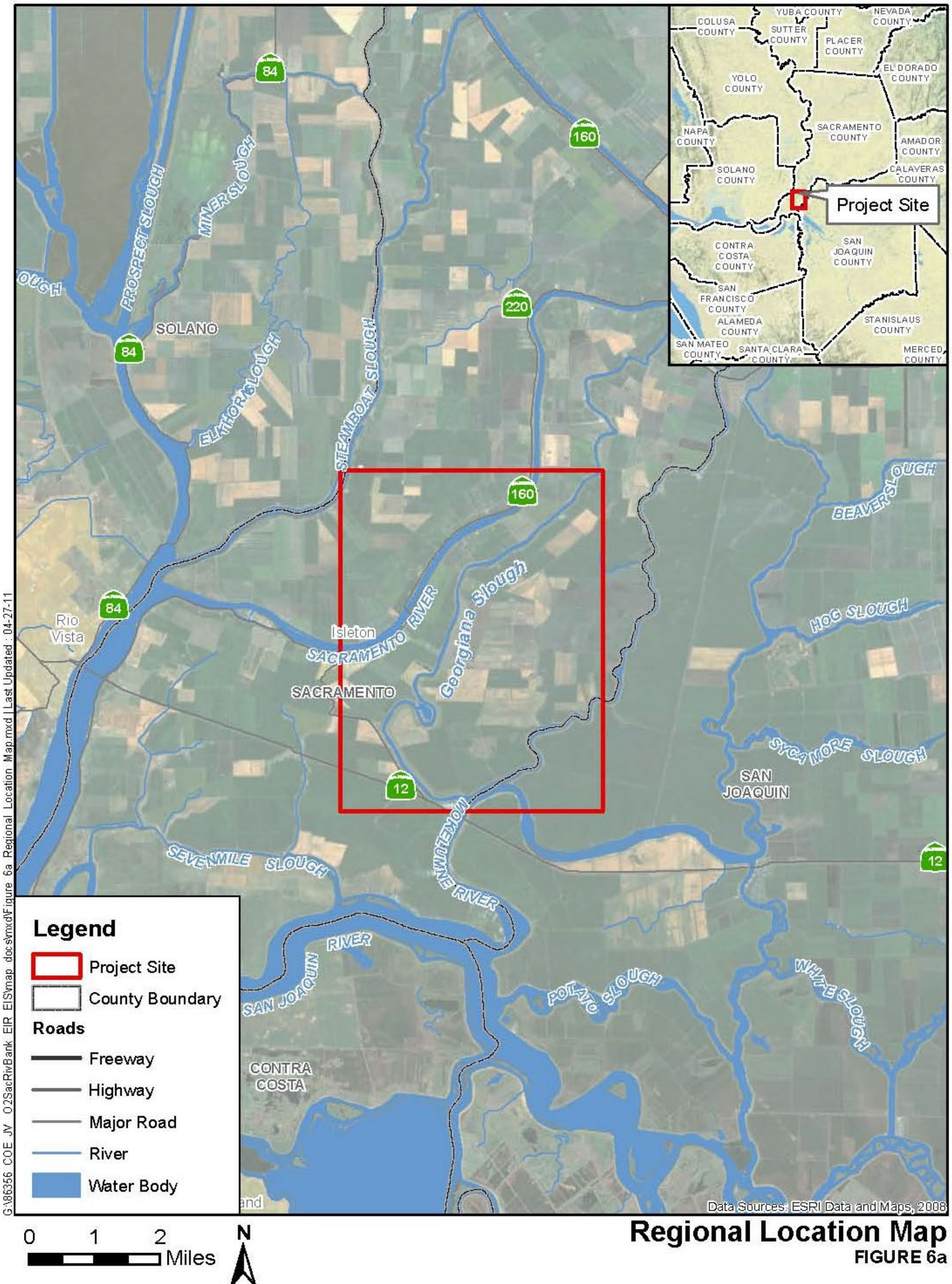
A summary of AR erosion sites that were combined or singularly assigned the setback or adjacent repair measure is summarized below in **Table 3**. **Figures 5 and 6** present the extents of the combined repair measures.

Table 3 – Summary of Sites Assigned Setback or Adjacent Repair Measure

Site Identification				Recommended Repair Measure
Georgiana Slough	RM	0.3	L	Combined Setback Levee
Georgiana Slough	RM	1.7	L	
Georgiana Slough	RM	2.5	L	
Georgiana Slough	RM	3.6	L	Combined Setback Levee
Georgiana Slough	RM	3.7a	L	
Georgiana Slough	RM	3.7b	L	
Georgiana Slough	RM	4.0	L	
Georgiana Slough	RM	4.3	L	Combined Adjacent Levee
Georgiana Slough	RM	4.5	L	
Georgiana Slough	RM	4.6	L	
Georgiana Slough	RM	5.3	L	
Georgiana Slough	RM	6.1	L	Single Site Adjacent Levee
Georgiana Slough	RM	6.4	L	Combined Setback Levee
Georgiana Slough	RM	6.6	L	
Georgiana Slough	RM	6.8	L	Single Site Adjacent Levee
Georgiana Slough	RM	8.3	L	Single Site Adjacent Levee
Steamboat Slough	RM	18.8	R	Single Site Adjacent Levee
Steamboat Slough	RM	23.9	R	Single Site Adjacent Levee
Steamboat Slough	RM	24.7	R	Single Site Adjacent Levee
Steamboat Slough	RM	25.0	L	Single Site Adjacent Levee
Steamboat Slough	RM	25.8	R	Single Site Adjacent Levee

Site Identification				Recommended Repair Measure
Steamboat Slough	RM	26.0	L	Single Site Adjacent Levee
Sutter Slough	RM	24.7	R	Single Site Setback Levee
Sacramento River	RM	22.7	L	Single Site Adjacent Levee
Sacramento River	RM	23.2	L	Single Site Adjacent Levee

Figure 5 - Regional Location Map

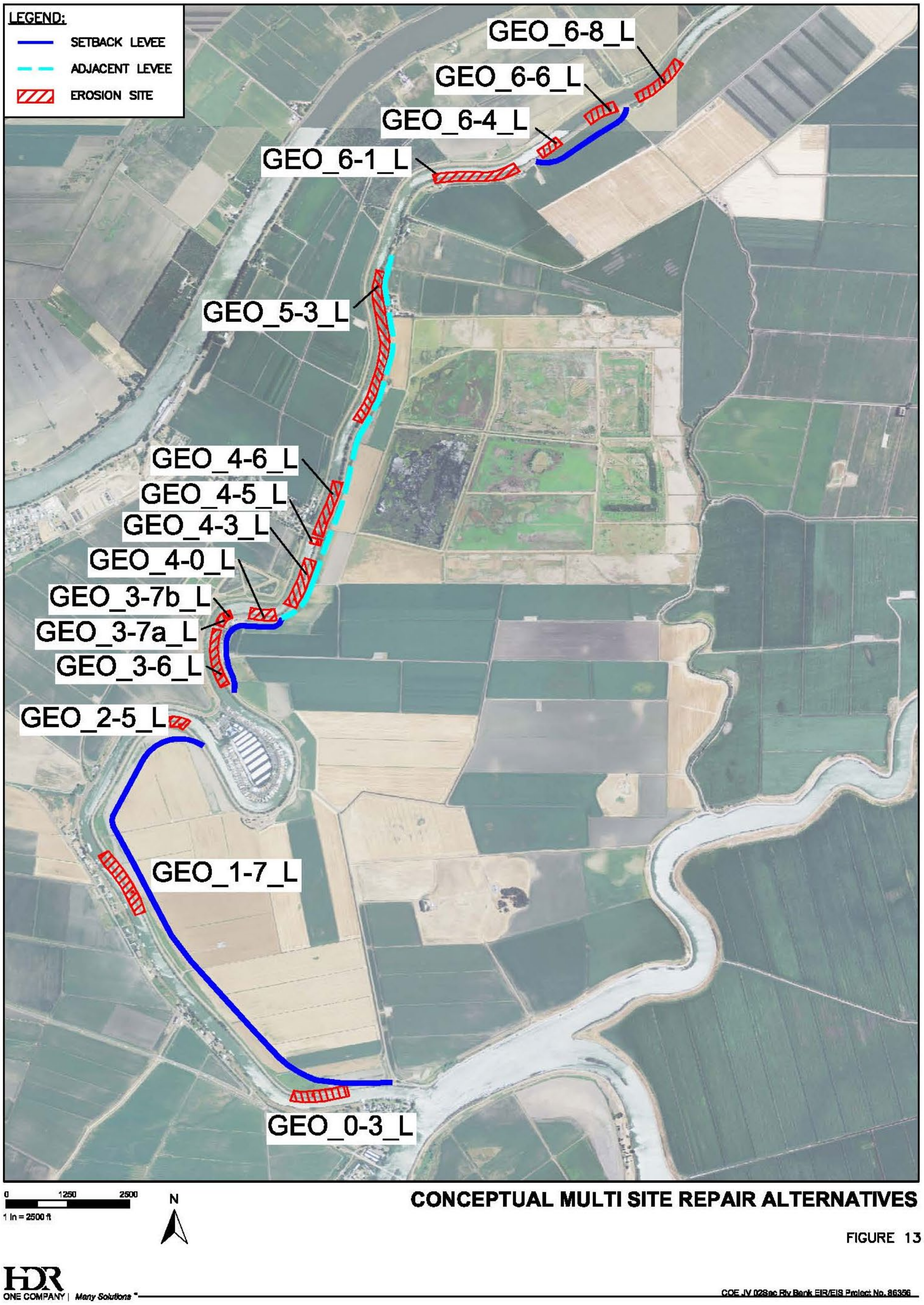


Regional Location Map
FIGURE 6a

HDR ONE COMPANY | *Many Solutions*

COE JV O2Sac Riv Bank EIR/EIS Project No. 86356

Figure 6- Conceptual Multi-Suite Erosion Locations



Step 6: Assign Measures to Remaining Sites

Sites with setback and adjacent levees were evaluated using the SAM model. Positive environmental effects at sites are noted so that a measure could also serve as near-site mitigation for another site with negative environmental effects. Where multiple erosion sites were grouped together for a multi-site setback or adjacent levee, the additional length between the actual erosion site boundaries was included in the calculation effects.

For the 14 remaining sites, an erosion protection measure is assigned that would minimize loss of fish habitat. This could be Alternative 1 Bank Fill Stone Protection Slope or similar to what was proposed in the AR. The environmental impacts of these measures would be evaluated and reported for each site or aggregate of sites.

A summary of the remaining AR erosion sites that were assigned an Measure other than the Setback or Adjacent repair measure is presented below in **Table 4**.

Table 4 - Summary of Sites Assigned ALT 2 or ALT 4c Repair Measure

Site Identification				Recommended Repair Measure
Cache Slough	RM	15.9	L	Measure 2
Cache Slough	RM	23.6	R	Measure 2
Knights Landing Ridge Cut	LM	0.2	R	Measure 2
Knights Landing Ridge Cut	LM	3.0	L	Measure 2
Knights Landing Ridge Cut	LM	3.1	L	Measure 2
Knights Landing Ridge Cut	LM	4.3	L	Measure 2
Knights Landing Ridge Cut	LM	5.3	L	Measure 2
Willow Slough	LM	6.9	R	Measure 2
Yolo Bypass	LM	0.1	R	Measure 2
Yolo Bypass	LM	2.0	R	Measure 2
Yolo Bypass	LM	2.8	R	Measure 2
Sacramento River	RM	21.5	L	Measure 4c
Sacramento River	RM	22.5	L	Measure 4c
Sacramento River	RM	24.8	L	Measure 2

4.3 Evaluate Mitigation Measures

Step 7: Evaluate Site's Impact to Fish Habitat

Once the VFZ is established the value of the sites' resulting diminishment of existing or potential vegetation may be determined.

Impacts to migratory fish were assessed by calculating the value of the existing riverbank habitat for rearing Chinook Salmon fry/juveniles, a species/life stage that is greatly associated with near shore habitat and is therefore susceptible to the effects of bank protection actions.

Habitat value was estimated using the relationships from SAM model, which relate several features of the river bank habitat to assumed responses from fish. The model and evaluation are described in the EIS/EIR.

There are two main variables in the SAM model that could be affected by VFZ restrictions: shade and aquatic vegetation. A reduction in trees reduces the amount of shaded cover, potentially increasing susceptibility to predation and, in smaller tributaries, increasing water temperature. A reduction in trees and other vegetation within the VFZ reduces the amount of inundated physical refuge habitat during higher water levels.

4.4 Develop Planning-Level Project Cost Estimate

Step 8: Develop Mitigation Plan

The aggregate environmental effects of all 106 sites were evaluated. The SAM model was used to determine effects to fish habitat. Losses in riparian and fish habitat were established by the environmental team.

In the lower regions of the study area, the Delta, the setback and adjacent levees provide net positive effects to fish habitat and riparian vegetation. Where multiple erosion sites were grouped together for a multi-site setback or adjacent levee, the additional length of non-eroded levee bank between the erosion sites was included in the calculation of effects. The positive effects of these levees were used to compensate for negative effects caused by bank protection at the other erosion sites in this region. Thus, in the lower Delta region no additional mitigation is required.

In the regions upstream of the Delta, most mitigation occurs on-site. For biological reasons it was not considered appropriate to use the beneficial effects of Delta adjacent and setback levees to compensate for construction upstream and removed from the Delta region. No setback or adjacent levees were proposed in these regions. For some sites there is no realistic opportunity to construct setback or adjacent levees due to neighboring development. For many sites on-site mitigation was accomplished by taking opportunities to protect and restore vegetation on portions of banks beyond the VFZ.

The construction cost estimate does not assume mitigation costs for cultural resources. Cultural resources recovery costs are included in the total project cost as $1 million, about one-half percent of construction cost. Cultural resources recovery costs are added onto the project cost as shown in the PACR.

Step 9: Off-Site Mitigation Plan

The aggregate environmental effects of all 106 sites is evaluated and summarized in the Environmental document. In the lower Delta regions it is self mitigating. The setback and adjacent levees fully mitigate all regional erosion sites. In the regions upstream of the Delta, most mitigation may occur on-site. However some off-site mitigation areas will be required to provide full mitigation. Off-site mitigation will be considered to compensate for losses. Sites are identified as part of the NEPA and CEQA process and are described in the Environmental document. Any net positive effects to riparian vegetation will be reported. No cultural resources mitigation costs were added.

5.0 Description of Priority Site Inventory

5.1 Overall Description

This section provides a discussion of the numbers of alternatives at sites, site relationships and groupings, environmental mitigation.

Table 5 presents a summary of erosion site attributes which includes Region, Site Identification listed in the AR, Site Length from the AR, AR Repair Measure and Revised Repair Measure. A blank cell under 'Revised Repair Measure' means the AR recommended either No Action, or the site was ETL compliant and no revision to the site repair was necessary. The distribution of erosion sites within the Sacramento Flood Control System (**Figures 7-25**) presents each site, identified by the abbreviation of the tributary and its associated River or Levee Mile location. **Figure 26** shows the process used to screen the 106 sites for compliance to ETL 1110-2-571 (now EP 1110-2-18) and determine repair measures and cost opinions for the PBPP.

Table 5 – Summary of Erosion Site Attributes

Region	Site Identification				Site Length	Alternatives Report Repair Measure	Revised Repair Measure
1A	Cache Creek	LM	3.9	L	433	Setback Levee	
1A	Cache Slough	RM	15.9	L	182	Design 4	Measure 2
1A	Cache Slough	RM	22.8	R	630	Design 4	
1A	Cache Slough	RM	23.6	R	1209	Design 4	Measure 2
1A	Deep Water Ship Channel	LM	5.0	L	N/A	No Action	
1A	Deep Water Ship Channel	LM	5.01	L	N/A	No Action	
1A	Georgiana Slough	RM	0.3	L	1027	Design 4	Measure 1 Combined Setback Levee
1A	Georgiana Slough	RM	1.7	L	1250	Design 4	
1A	Georgiana Slough	RM	2.5	L	736	Design 4	
1A	Georgiana Slough	RM	3.6	L	1364	Design 4	Measure 1 Combined Setback Levee
1A	Georgiana Slough	RM	3.7a	L	209	Design 4	
1A	Georgiana Slough	RM	3.7b	L	268	Design 4	
1A	Georgiana Slough	RM	4.0	L	705	Design 4	
1A	Georgiana Slough	RM	4.3	L	1319	Design 4	Measure 3 Combined Adjacent Levee
1A	Georgiana Slough	RM	4.5	L	90	Design 4	
1A	Georgiana Slough	RM	4.6	L	1346	Design 4	
1A	Georgiana Slough	RM	5.3	L	3171	Design 4	
1A	Georgiana Slough	RM	6.1	L	1729	Design 4	Measure 3
1A	Georgiana Slough	RM	6.4	L	398	Design 4	Measure 1 Combined Setback Levee
1A	Georgiana Slough	RM	6.6	L	744	Design 4	
1A	Georgiana Slough	RM	6.8	L	1335	Design 4	Measure 3
1A	Georgiana Slough	RM	8.3	L	483	Design 4	Measure 3
1A	Georgiana Slough	RM	9.3	L	1228	Design 4	
1A	Knights Landing Ridge Cut	LM	0.2	R	768	Design 1	Measure 2

Region	Site Identification				Site Length	Alternatives Report Repair Measure	Revised Repair Measure
1A	Knights Landing Ridge Cut	LM	3.0	L	1279	Design 1	Measure 2
1A	Knights Landing Ridge Cut	LM	3.1	L	368	Design 1	Measure 2
1A	Knights Landing Ridge Cut	LM	4.3	L	577	Design 1	Measure 2
1A	Knights Landing Ridge Cut	LM	5.3	L	8564	Design 1	Measure 2
1A	Steamboat Slough	RM	18.8	R	485	Design 4	Measure 3
1A***	Steamboat Slough	RM	23.2	L	N/A	No Action	
1A	Steamboat Slough	RM	23.9	R	369	Design 4	Measure 3
1A	Steamboat Slough	RM	24.7	R	911	Design 4	Measure 3
1A	Steamboat Slough	RM	25.0	L	272	Design 4	Measure 3
1A	Steamboat Slough	RM	25.8	R	244	Design 4	Measure 3
1A	Steamboat Slough	RM	26.0	L	516	Design 4	Measure 3
1A	Sutter Slough	RM	24.7	R	1736	Design 4	Measure 1
1A	Sutter Slough	RM	26.5	L	568	Design 4	
1A	Willow Slough	LM	0.2	L	N/A	No Action	
1A	Willow Slough	LM	0.7	L	N/A	No Action	
1A	Willow Slough	LM	6.9	R	869	Design 1	Measure 2
1A	Yolo Bypass	LM	0.1	R	430	Design 1	Measure 2
1A	Yolo Bypass	LM	2.0	R	563	Design 1	Measure 2
1A***	Yolo Bypass	LM	2.5	R	148	Design 1	
1A	Yolo Bypass	LM	2.6	R	N/A	No Action	
1A	Yolo Bypass	LM	3.8	R	1860	Design 1	Measure 2
1B*	Lower American River	RM	7.3	R	446	No Action	
1B	Sacramento River	RM	21.5	L	162	Design 4	Measure 4c
1B	Sacramento River	RM	22.5	L	852	Design 4	Measure 4c
1B	Sacramento River	RM	22.7	L	309	Design 4	Measure 3
1B	Sacramento River	RM	23.2	L	589	Design 4	Measure 3
1B	Sacramento River	RM	23.3	L	257	Design 4	
1B	Sacramento River	RM	24.8	L	782	Design 4	Measure 2
1B	Sacramento River	RM	25.2	L	338	Design 4	
1B	Sacramento River	RM	31.6	R	446	Design 1	
1B***	Sacramento River	RM	35.3	R	197	Design 2	
1B***	Sacramento River	RM	35.4	R	96	Design 2	
1B	Sacramento River	RM	38.5	R	359	Design 1	
1B	Sacramento River	RM	56.5	R	373	Design 3	
1B	Sacramento River	RM	56.6	L	86	Design 2	
1B	Sacramento River	RM	56.7	R	665	Design 3	
1B*	Sacramento River	RM	58.4	L	707	Design 1	
1B***	Sacramento River	RM	60.1	L	455	Design 2	
1B	Sacramento River	RM	62.9	R	175	Design 3	
1B	Sacramento River	RM	63.0	R	87	Design 3	
1B	Sacramento River	RM	74.4	R	200	Design 3	

Region	Site Identification				Site Length	Alternatives Report Repair Measure	Revised Repair Measure
1B	Sacramento River	RM	75.3	R	2761	Design 1	
1B	Sacramento River	RM	77.7	R	224	Design 1	
1B	Sacramento River	RM	78.3	L	657	Design 1	
2	Bear River	RM	0.8	L	233	Design 1	
2	Cherokee Canal	LM	14.0	L	184	No Action	
2	Cherokee Canal	LM	21.9	L	1800	Design 1	
2	Feather River	RM	0.6	L	288	Design 2	
2**	Feather River	RM	5.0	L	910	Design 2	
2	Sacramento River	RM	86.3	L	3134	Design 1	
2***	Sacramento River	RM	86.5	R	72	Design 3	
2	Sacramento River	RM	86.9	R	289	Design 3	
2	Sacramento River	RM	92.8	L	200	Design 1	
2	Sacramento River	RM	95.8	L	190	Design 1	
2	Sacramento River	RM	96.2	L	560	Design 1	
2	Sacramento River	RM	99.0	L	160	Design 1	
2	Sacramento River	RM	101.3	R	352	Design 3	
2	Sacramento River	RM	103.4	L	N/A	No Action	
2	Sacramento River	RM	104.0	L	3459	Design 1	
2	Sacramento River	RM	104.5	L	301	Design 2	
2	Sacramento River	RM	116.0	L	612	Design 2	
2	Sacramento River	RM	116.5	L	2465	Design 2	
2	Sacramento River	RM	122.0	R	248	Design 3	
2	Sacramento River	RM	122.3	R	341	Design 3	
2	Sacramento River	RM	123.3	L	208	Design 3	
2	Sacramento River	RM	123.7	R	120	Design 2	
2	Sacramento River	RM	127.9	R	801	Design 1	
2	Sacramento River	RM	131.8	L	339	Design 2	
2	Sacramento River	RM	132.9	R	363	Design 2	
2	Sacramento River	RM	133.0	L	1291	Design 2	
2	Sacramento River	RM	133.8	L	197	Design 2	
2	Sacramento River	RM	136.6	L	615	Design 2	
2	Sacramento River	RM	138.1	L	1365	Design 2	
2	Yuba River	LM	2.3	L	1356	Setback Levee	
3	Deer Creek	LM	2.4	L	496	Design 1	
3	Elder Creek	LM	1.44	L	334	Design 2	
3	Elder Creek	LM	3.0	R	65	Design 2	
3***	Elder Creek	LM	4.1	L	N/A	No Action	
3	Sacramento River	RM	152.8	L	198	Design 3	
3	Sacramento River	RM	163.0	L	1213	Design 3	
3	Sacramento River	RM	168.3	L	546	Design 3	
3	Sacramento River	RM	172.0	L	525	Design 3	

* Sacramento River 58.4 and Lower American River 7.3 have been erroneously included in the analysis. These are not found in the erosion site inventory. They do not meet the requirements for an erosion site under SRBPP. Leaving them in the analysis, however, does not make a significant difference because of the programmatic nature of the bank protection plan and they still can function

as representative sites.

** Feather River 5.0L was erroneously referred to as Feather River 4.9L in the Alternatives Report and potentially other documents.

*** Sacramento River 35.3R, 35.4R, 60.1L, 86.5R, Elder Creek 4.1L, Steamboat Slough 23.2L, and Yolo Bypass 2.5R have been repaired.

Step 10: Real Estate Requirements

Areas of land required for setback and adjacent levees were calculated. The acquisition cost for these sites was estimated at $10,000 per acre, which is representative for agricultural land in the Sacramento Valley. No lands costs were included for sites with Bank Fill Stone Protection or with Riparian and Wetland Banks with Revegetation. No relocations costs were assumed for the cost estimate.

An acquisition challenge at some sites is the disposition of encroachments, both permitted and not permitted. Resolving permits and determining resultant relocation requirements at some sites may add to the cost of the project. This issue is discussed in more detail in **PACR Appendix C Programmatic Real Estate Plan**.

Step 11: Cost Estimate

The opinions of probable costs are summarized in **Table 6**. The summary is organized by region and each site is identified by tributary/channel name, the levee/river mile marker and which bank the repair resides on. Each total cost includes the following markups:

- Escalation – 2%
- Contingency – 20%
- Supervision, Inspection and Overhead – 8%
- Home Office Overhead – 8%
- Profit – 8%

◈ Bond – 1.25%

The total cost for the 77,436 LF of bank protection is $203,561,167 which gives an average liner foot cost of $2,629.

After this cost for the PSI was prepared a more detailed cost estimate was developed and is displayed in **Subappendix A2.d. Cost and Schedule Risk Analysis Report** This estimate was used for the benefit – cost analysis described in the Economic Appendix. The cost is at an alternative comparison level of detail.

These costs summarized below are initial cost opinions. More detailed cost analyses will be required on a site by site basis as these erosion sites are developed for construction. For a more detailed analysis of the cost opinions refer to **Subappendix A2.d Cost and Schedule Risk Analysis Report**.

Table 6 - First Cost Price Level Summarization

Region	Site Identification				First Construction Cost
1A	Cache Creek	LM	3.9	L	$638,661
1A	Cache Slough	RM	15.9	L	$1,619,596
1A	Cache Slough	RM	22.8	R	$527,206
1A	Cache Slough	RM	23.6	R	$1,376,525
1A	Deep Water Ship Channel	LM	5.0	L	$0
1A	Deep Water Ship Channel	LM	5.0	L	$0
1A	Georgiana Slough	RM	0.3	L	$30,143,038
1A	Georgiana Slough	RM	1.7	L	
1A	Georgiana Slough	RM	2.5	L	
1A	Georgiana Slough	RM	3.6	L	$6,331,912
1A	Georgiana Slough	RM	3.7a	L	
1A	Georgiana Slough	RM	3.7b	L	
1A	Georgiana Slough	RM	4.0	L	
1A	Georgiana Slough	RM	4.3	L	$16,809,762
1A	Georgiana Slough	RM	4.5	L	
1A	Georgiana Slough	RM	4.6	L	
1A	Georgiana Slough	RM	5.3	L	
1A	Georgiana Slough	RM	6.1	L	$3,572,860
1A	Georgiana Slough	RM	6.4	L	$3,838,557
1A	Georgiana Slough	RM	6.6	L	
1A	Georgiana Slough	RM	6.8	L	$2,710,953
1A	Georgiana Slough	RM	8.3	L	$1,037,195
1A	Georgiana Slough	RM	9.3	L	$4,551,611
1A	Knights Landing Ridge Cut	LM	0.2	R	$69,460

Region	Site Identification				First Construction Cost
1A	Knights Landing Ridge Cut	LM	3.0	L	$408,793
1A	Knights Landing Ridge Cut	LM	3.1	L	$177,096
1A	Knights Landing Ridge Cut	LM	4.3	L	$459,340
1A	Knights Landing Ridge Cut	LM	5.3	L	$3,263,940
1A	Steamboat Slough	RM	18.8	R	$1,552,251
1A***	Steamboat Slough	RM	23.2	L	$0
1A	Steamboat Slough	RM	23.9	R	$1,084,698
1A	Steamboat Slough	RM	24.7	R	$2,819,727
1A	Steamboat Slough	RM	25.0	L	$660,720
1A	Steamboat Slough	RM	25.8	R	$519,721
1A	Steamboat Slough	RM	26.0	L	$1,262,770
1A	Sutter Slough	RM	24.7	R	$5,804,608
1A	Sutter Slough	RM	26.5	L	$2,363,454
1A	Willow Slough	LM	0.2	L	$0
1A	Willow Slough	LM	0.7	L	$0
1A	Willow Slough	LM	6.9	R	$258,406
1A	Yolo Bypass	LM	0.1	R	$266,788
1A	Yolo Bypass	LM	2.0	R	$447,880
1A***	Yolo Bypass	LM	2.5	R	$83,442
1A	Yolo Bypass	LM	2.6	R	$0
1A	Yolo Bypass	LM	3.8	R	$1,902,181
1B*	Lower American River	RM	7.3	R	$0
1B	Sacramento River	RM	21.5	L	$563,325
1B	Sacramento River	RM	22.5	L	$1,869,692
1B	Sacramento River	RM	22.7	L	$733,394
1B	Sacramento River	RM	23.2	L	$1,422,810
1B	Sacramento River	RM	23.3	L	$1,169,341
1B	Sacramento River	RM	24.8	L	$3,395,102
1B	Sacramento River	RM	25.2	L	$1,004,012
1B	Sacramento River	RM	31.6	R	$3,084,148
1B***	Sacramento River	RM	35.3	R	$1,652,501
1B***	Sacramento River	RM	35.4	R	$340,496
1B	Sacramento River	RM	38.5	R	$2,522,344
1B	Sacramento River	RM	56.5	R	$1,262,827
1B	Sacramento River	RM	56.6	L	$290,378
1B	Sacramento River	RM	56.7	R	$5,695,436
1B*	Sacramento River	RM	58.4	L	$1,332,361
1B***	Sacramento River	RM	60.1	L	$2,841,635
1B	Sacramento River	RM	62.9	R	$402,035
1B	Sacramento River	RM	63.0	R	$451,201
1B	Sacramento River	RM	74.4	R	$499,086

Region	Site Identification				First Construction Cost
1B	Sacramento River	RM	75.3	R	$3,143,933
1B	Sacramento River	RM	77.7	R	$907,020
1B	Sacramento River	RM	78.3	L	$1,539,346
2	Bear River	RM	0.8	L	$675,163
2	Cherokee Canal	LM	14.0	L	$0
2	Cherokee Canal	LM	21.9	L	$1,158,689
2	Feather River	RM	0.6	L	$1,288,932
2**	Feather River	RM	5.0	L	$3,181,373
2	Sacramento River	RM	86.3	L	$6,011,173
2***	Sacramento River	RM	86.5	R	$243,224
2	Sacramento River	RM	86.9	R	$1,226,930
2	Sacramento River	RM	92.8	L	$1,355,902
2	Sacramento River	RM	95.8	L	$1,031,518
2	Sacramento River	RM	96.2	L	$3,926,336
2	Sacramento River	RM	99.0	L	$1,114,291
2	Sacramento River	RM	101.3	R	$1,579,059
2	Sacramento River	RM	103.4	L	$0
2	Sacramento River	RM	104.0	L	$13,306,210
2	Sacramento River	RM	104.5	L	$1,063,851
2	Sacramento River	RM	116.0	L	$1,271,528
2	Sacramento River	RM	116.5	L	$8,083,110
2	Sacramento River	RM	122.0	R	$606,015
2	Sacramento River	RM	122.3	R	$1,012,648
2	Sacramento River	RM	123.3	L	$567,168
2	Sacramento River	RM	123.7	R	$1,022,553
2	Sacramento River	RM	127.9	R	$2,108,298
2	Sacramento River	RM	131.8	L	$562,176
2	Sacramento River	RM	132.9	R	$1,402,910
2	Sacramento River	RM	133.0	L	$1,635,862
2	Sacramento River	RM	133.8	L	$976,181
2	Sacramento River	RM	136.6	L	$1,547,692
2	Sacramento River	RM	138.1	L	$4,093,959
2	Yuba River	LM	2.3	L	$1,227,930
3	Deer Creek	LM	2.4	L	$448,710
3	Elder Creek	LM	1.4	L	$717,833
3	Elder Creek	LM	3.0	R	$106,712
3***	Elder Creek	LM	4.1	L	$0
3	Sacramento River	RM	152.8	L	$1,260,297
3	Sacramento River	RM	163.0	L	$2,160,285
3	Sacramento River	RM	168.3	L	$1,869,826
3	Sacramento River	RM	172.0	L	$1,031,255

* Sacramento River 58.4 and Lower American River 7.3 have been erroneously included in the analysis. These are not found in the erosion site inventory. They do not meet the requirements for an erosion site under SRBPP. Leaving them in the analysis, however, does not make a significant difference because of the programmatic nature of the bank protection plan and they still can function as representative sites.

** Feather River 5.0L was erroneously referred to as Feather River 4.9L in the Alternatives Report and potentially other documents.

*** Sacramento River 35.3R, 35.4R, 60.1L, 86.5R, Elder Creek 4.1L, Steamboat Slough 23.2L, and Yolo Bypass 2.5R have been repaired.

Figure 7– Alternatives Report Erosion Sites

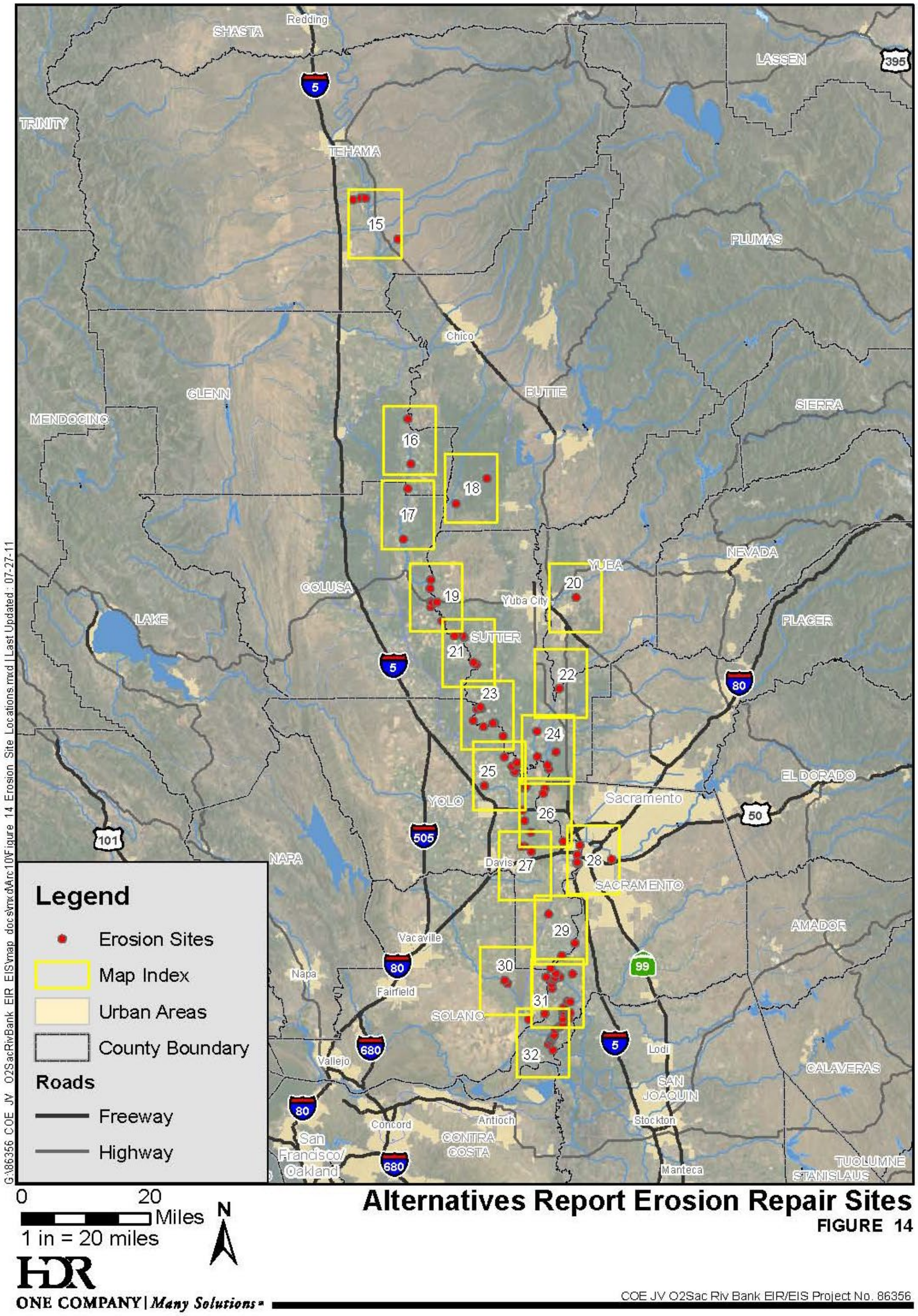


Figure 8- Alternatives Report Erosion Sites

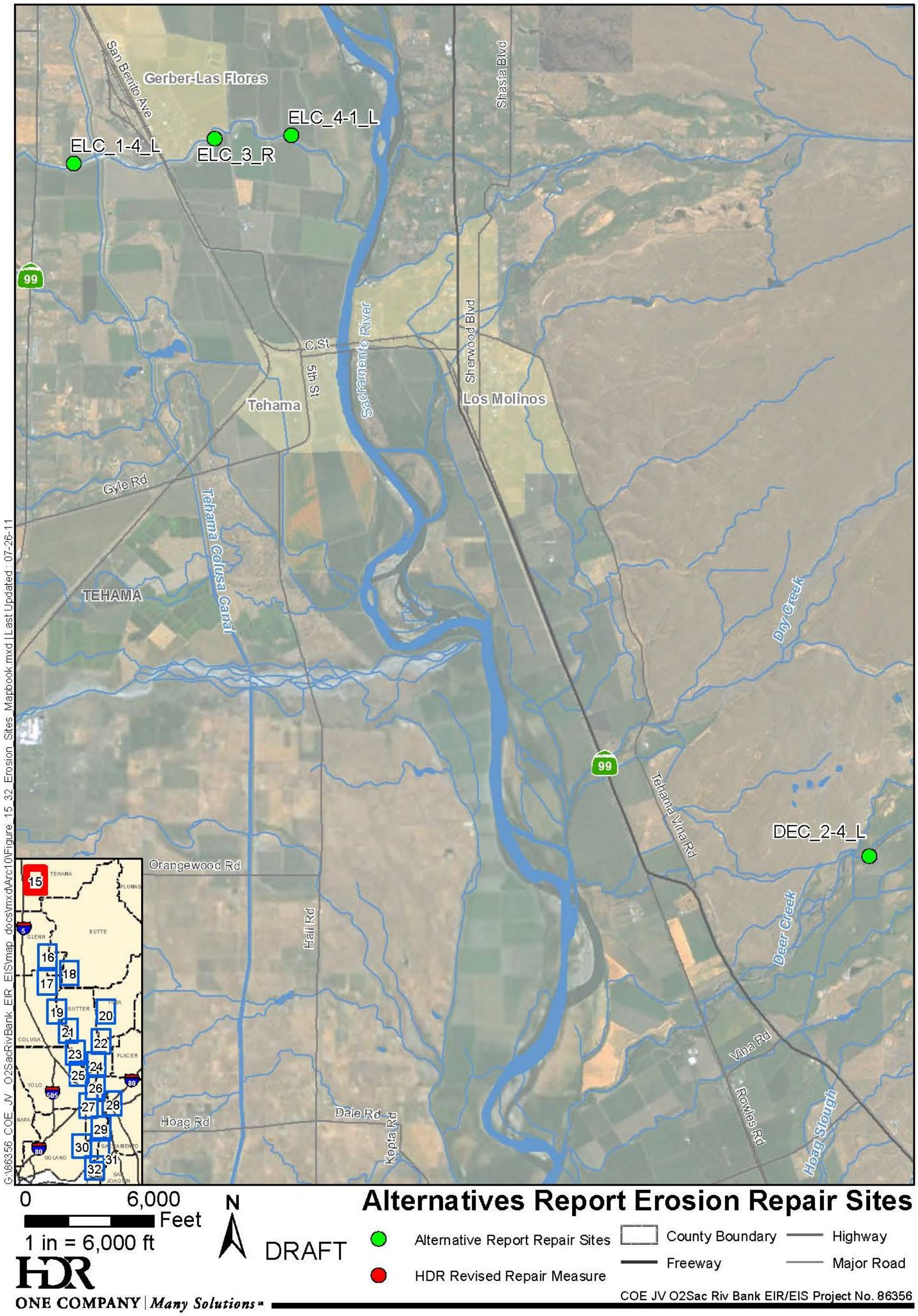


Figure 9- Alternative Report Erosion Sites

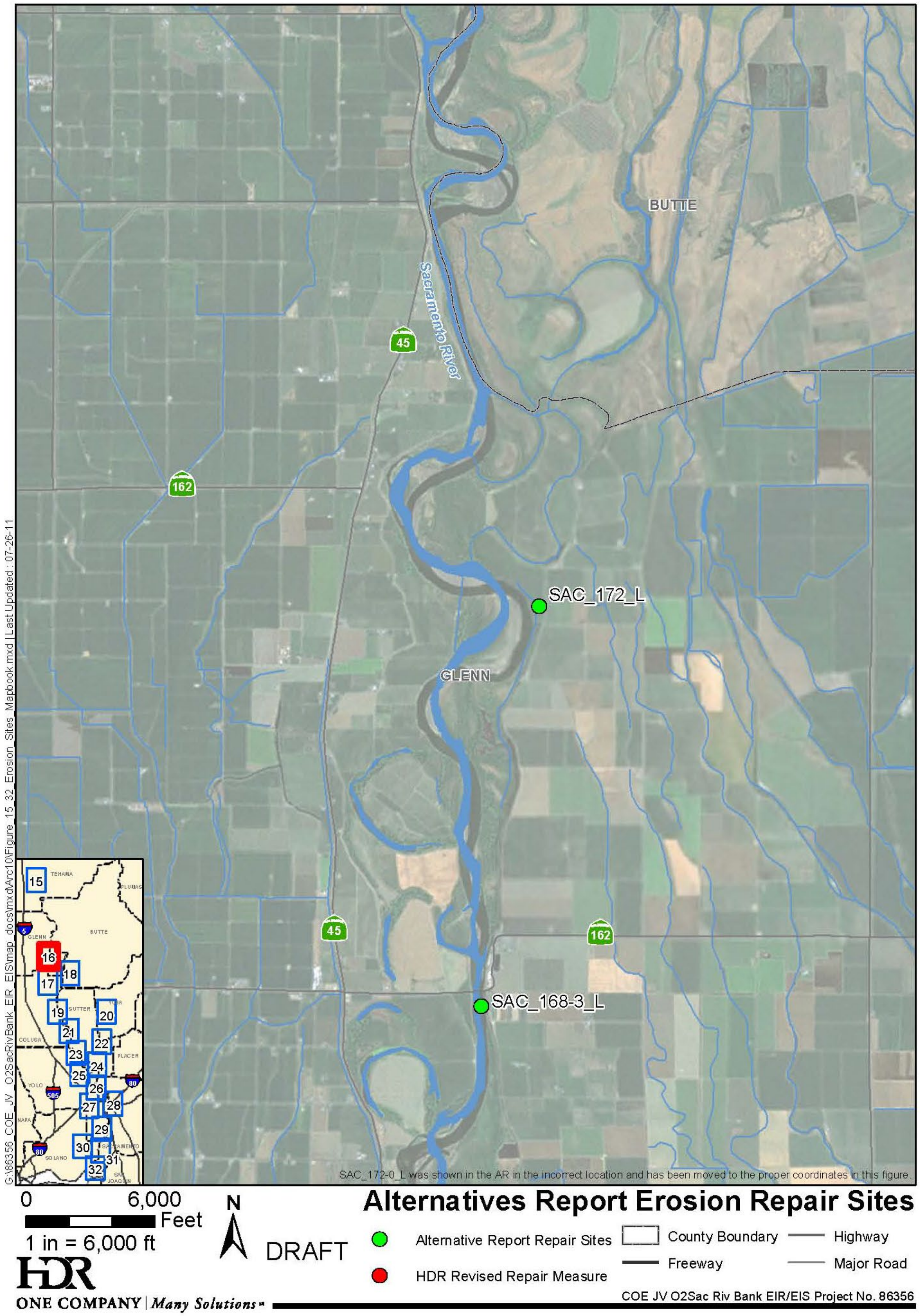


Figure 10- Alternatives Report Erosion Sites

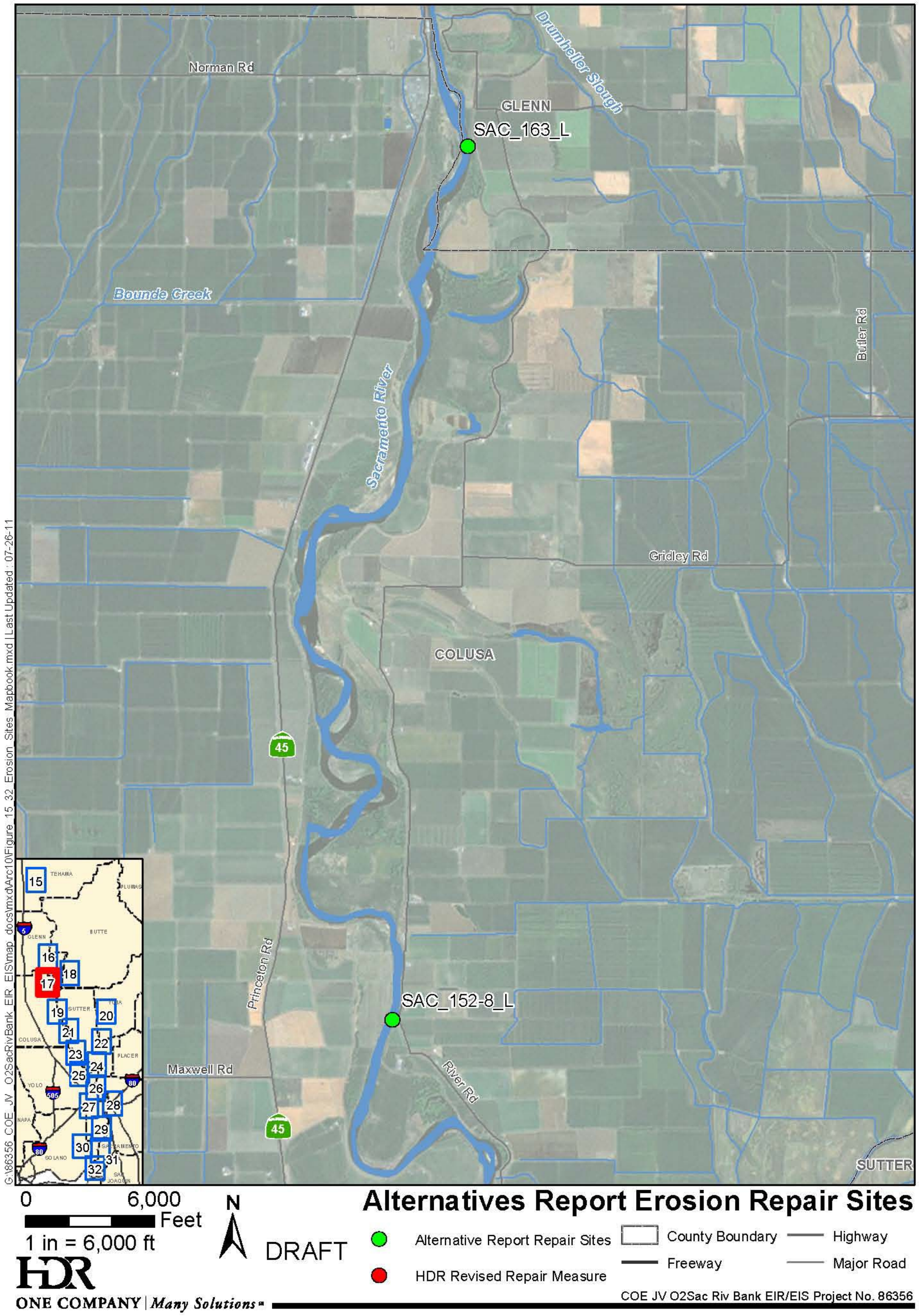


Figure 11 -Alternatives Report Erosion Sites

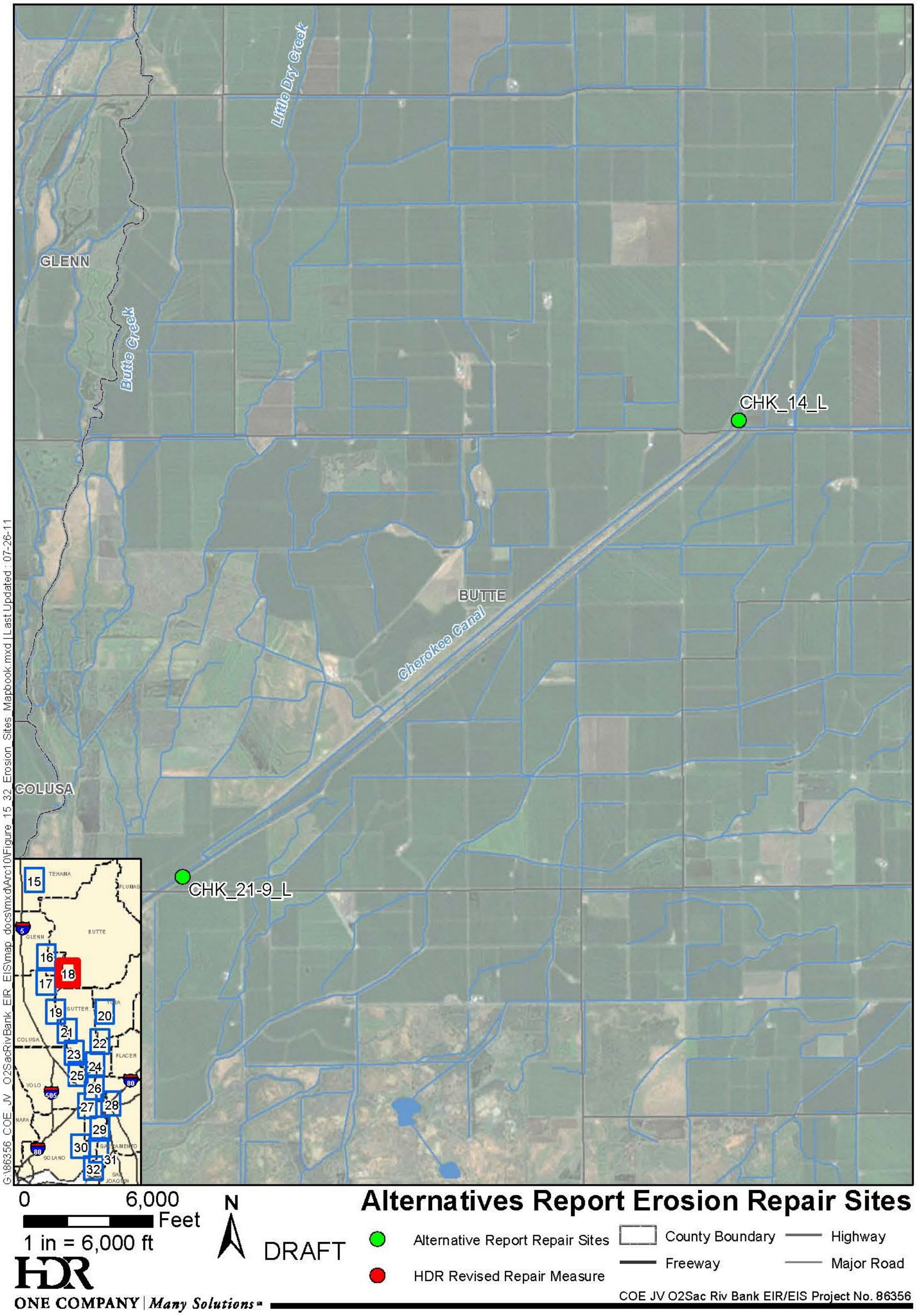


Figure 12- Alternatives Report Erosion Sites

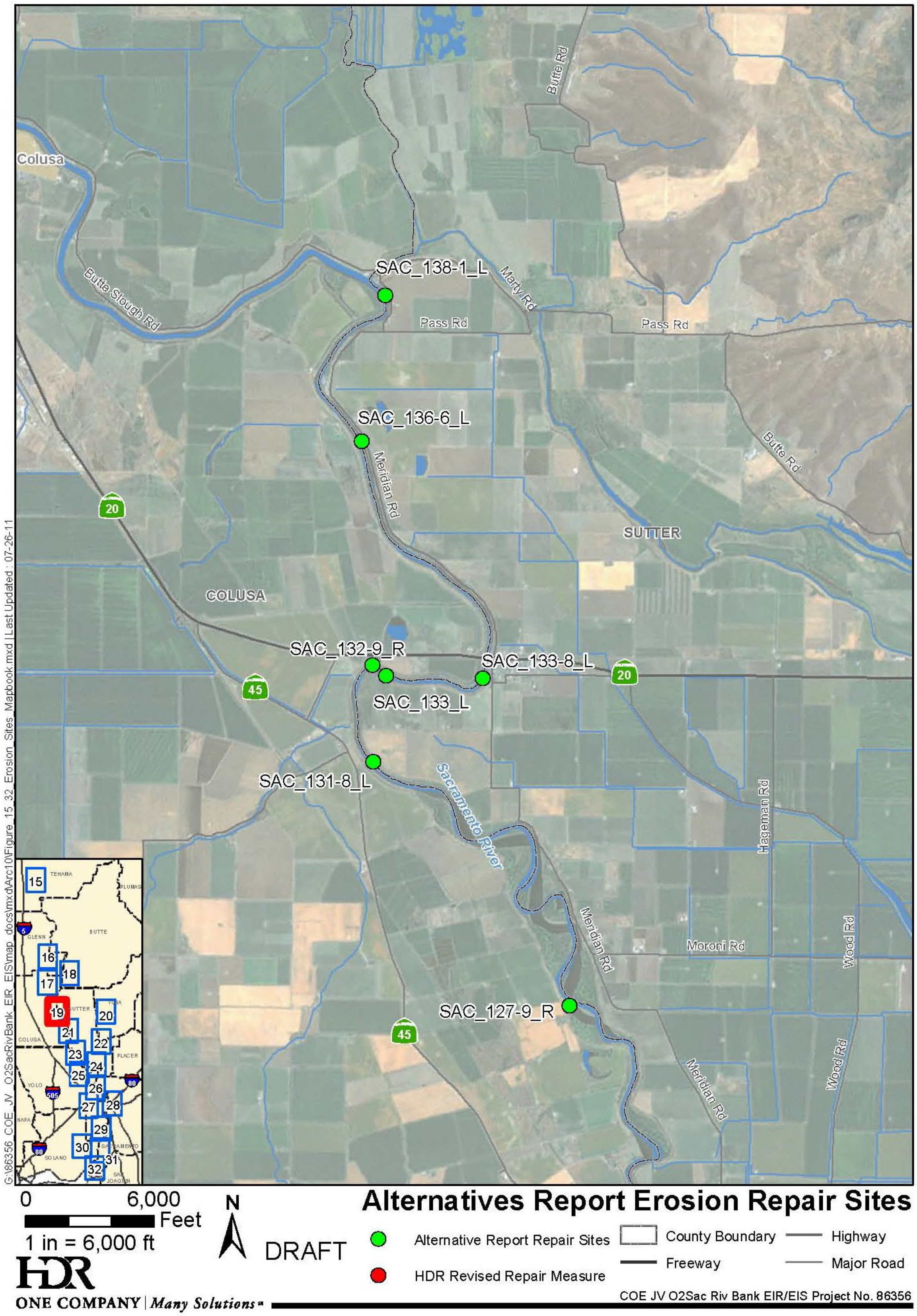


Figure 13 -Alternative Report Erosion Sites

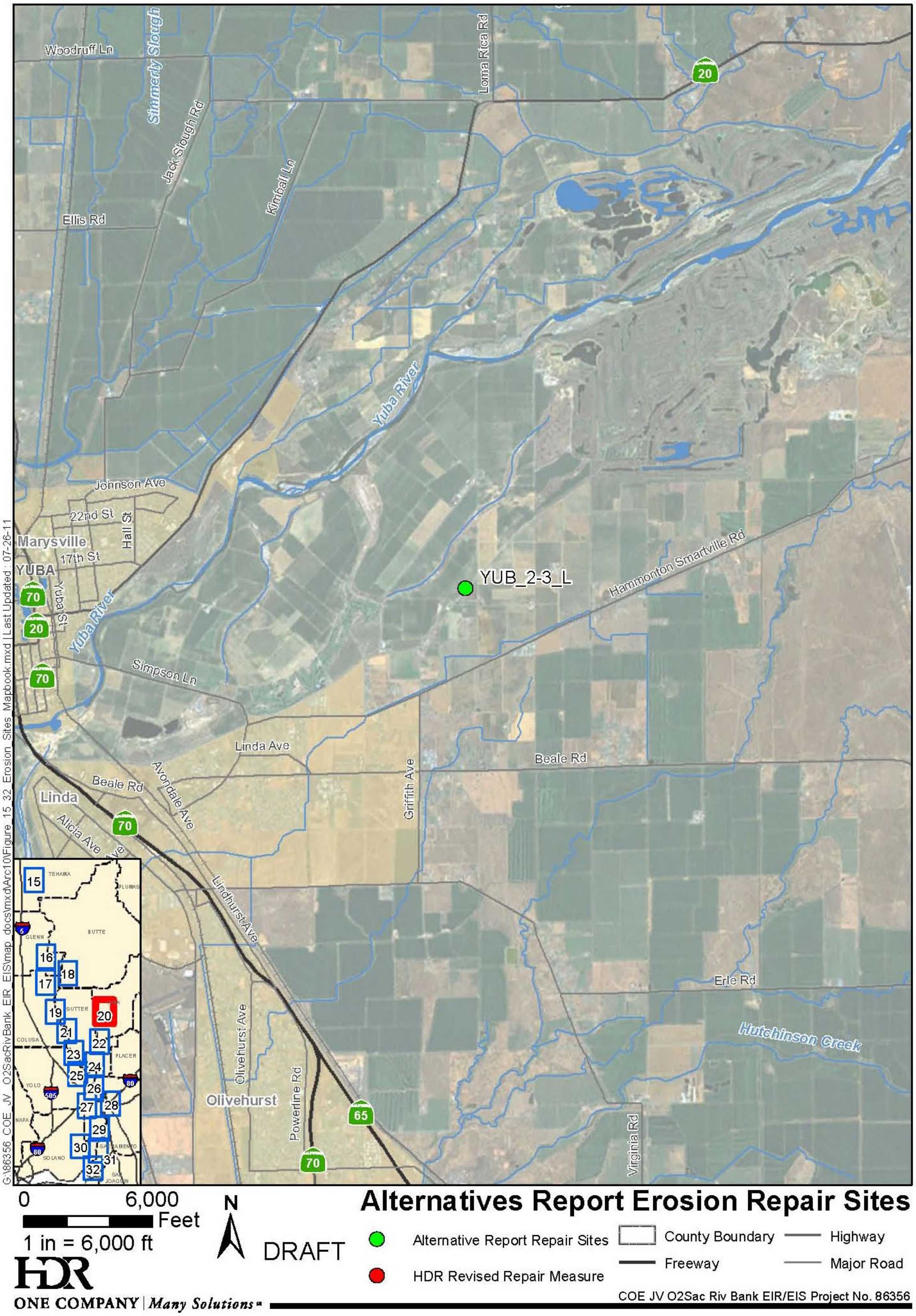


Figure 14- Alternatives Report Erosion Sites

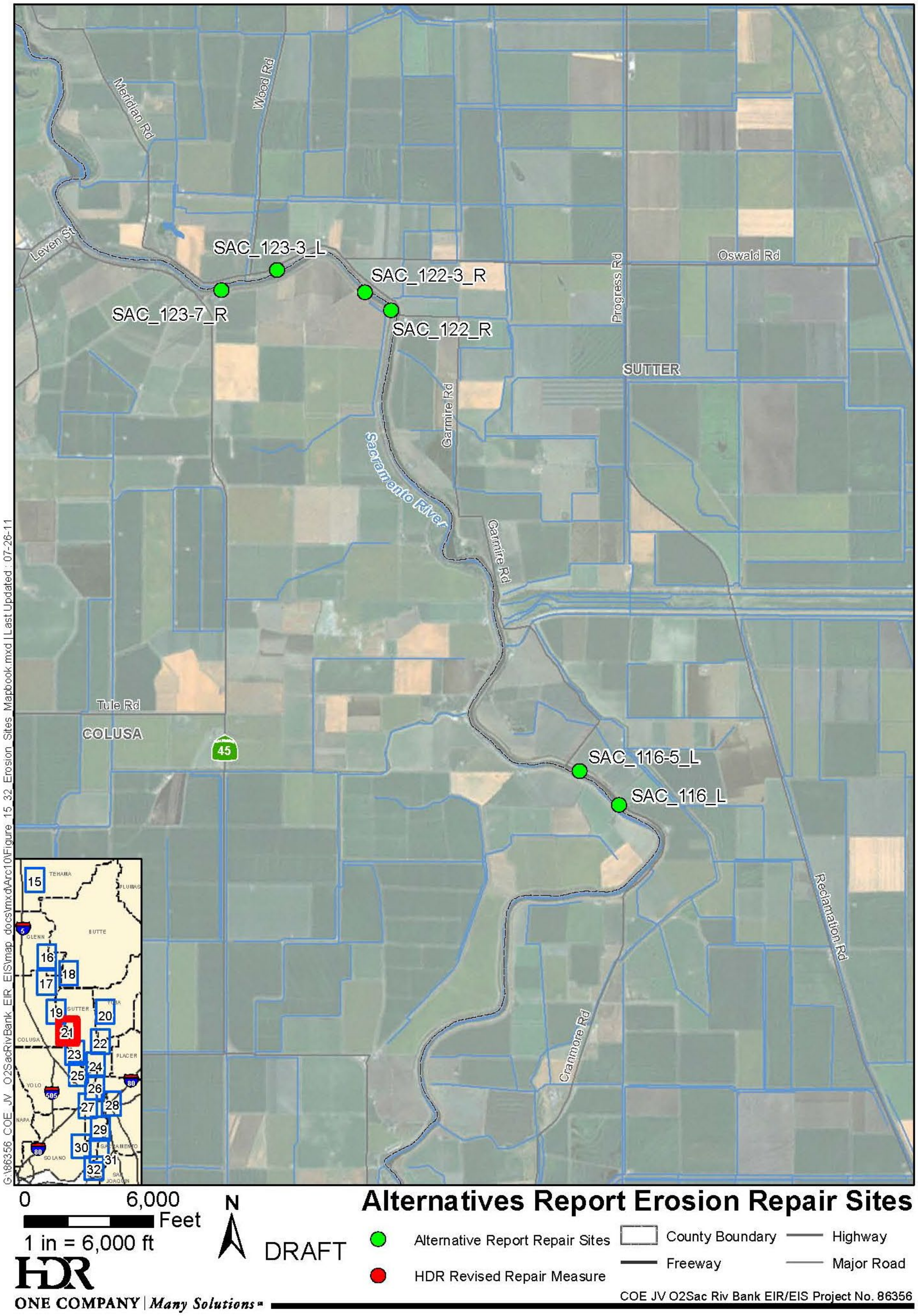


Figure 15 -Alternatives Report Erosion Sites

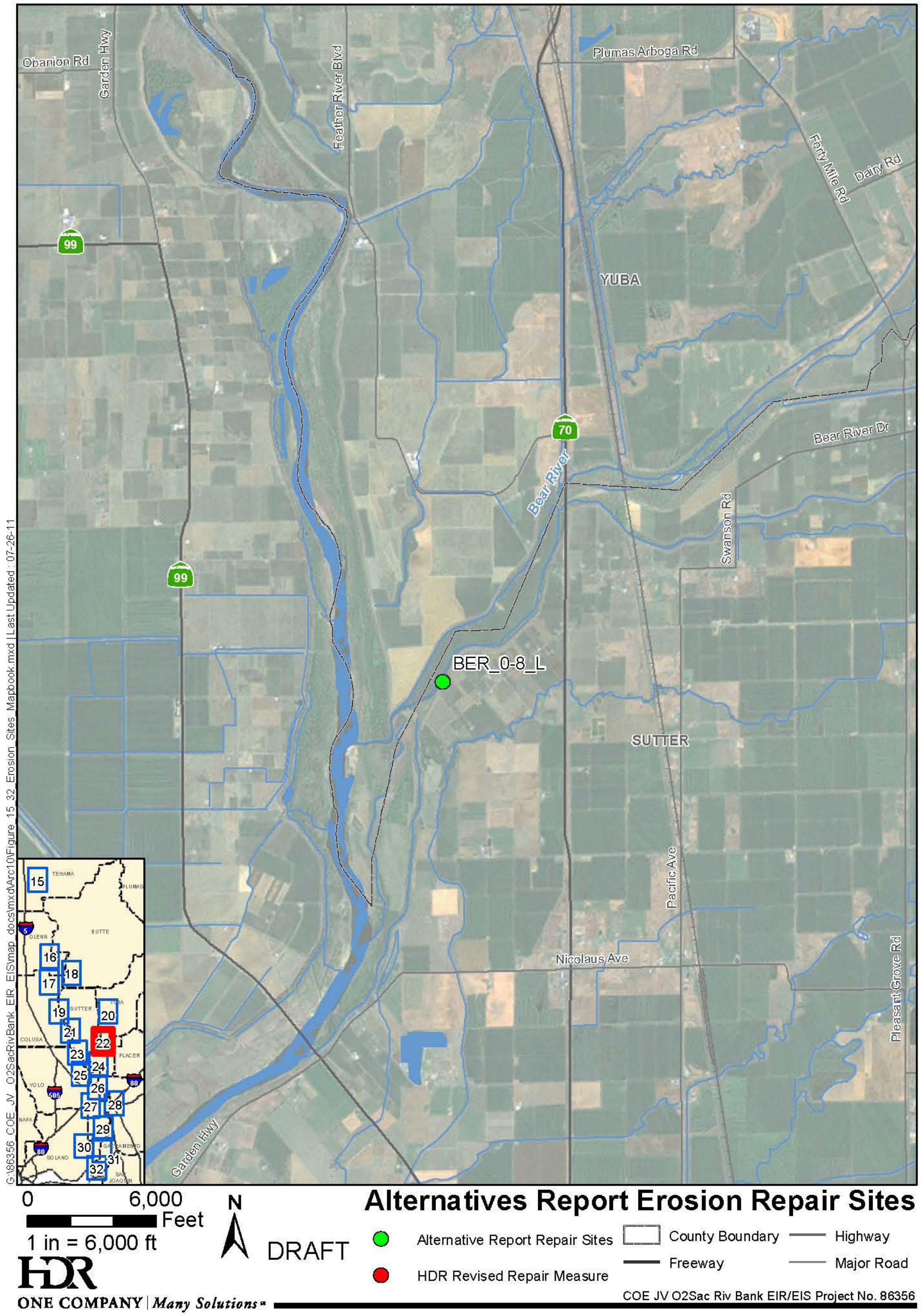


Figure 16- Alternatives Report Erosion Sites

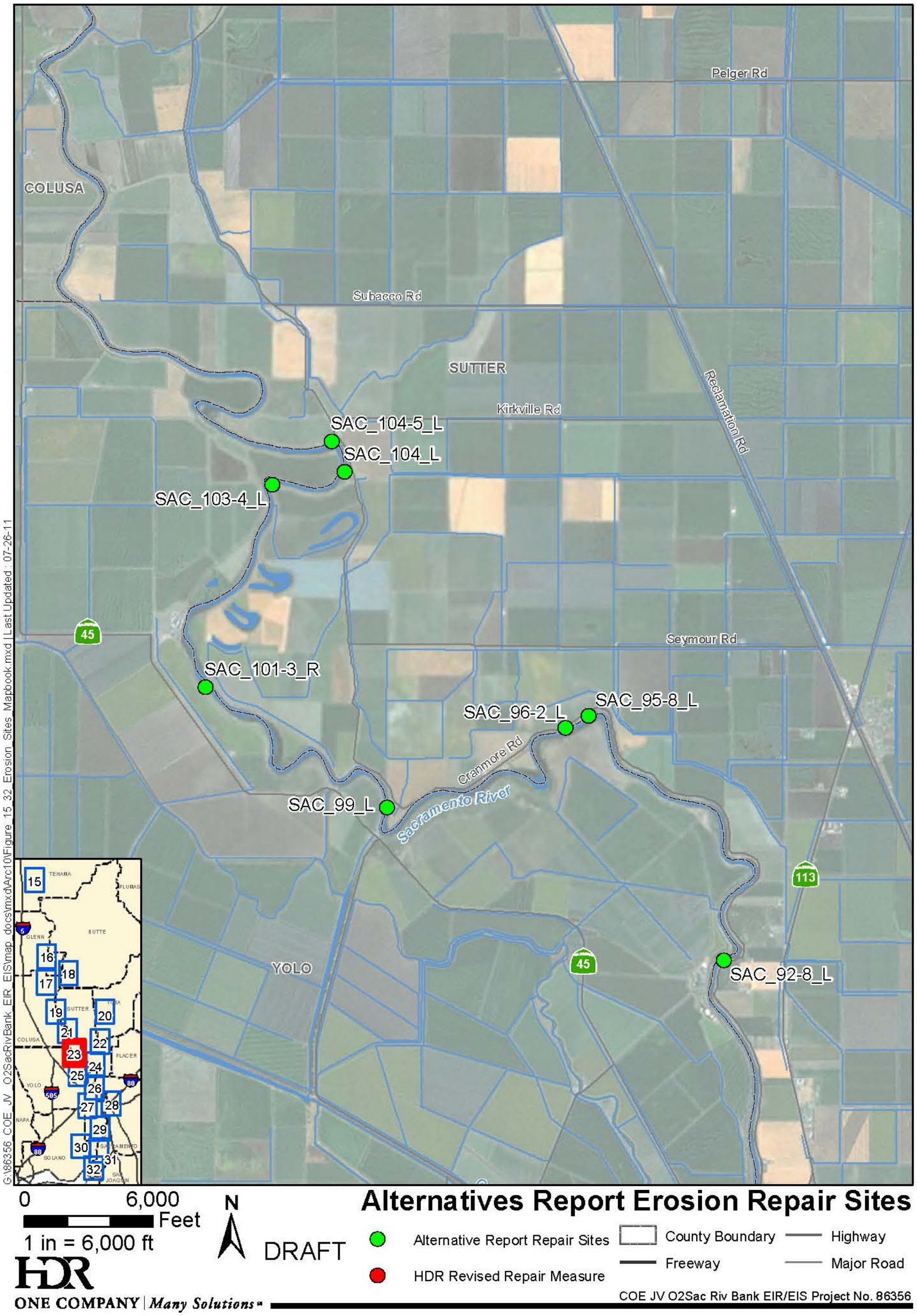


Figure 17- Alternatives Report Erosion Sites

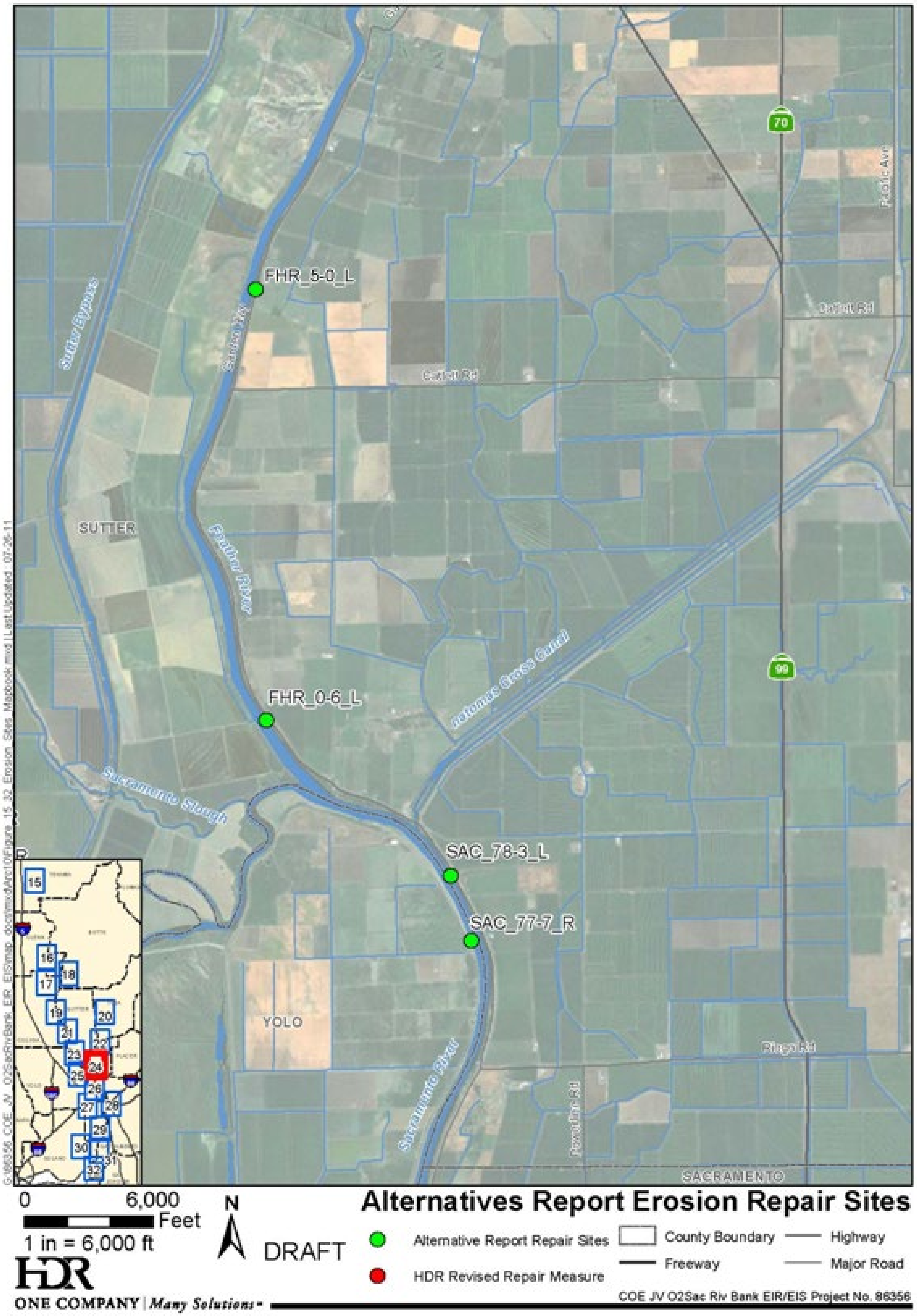


Figure 18- Alternatives Report Erosion Sites

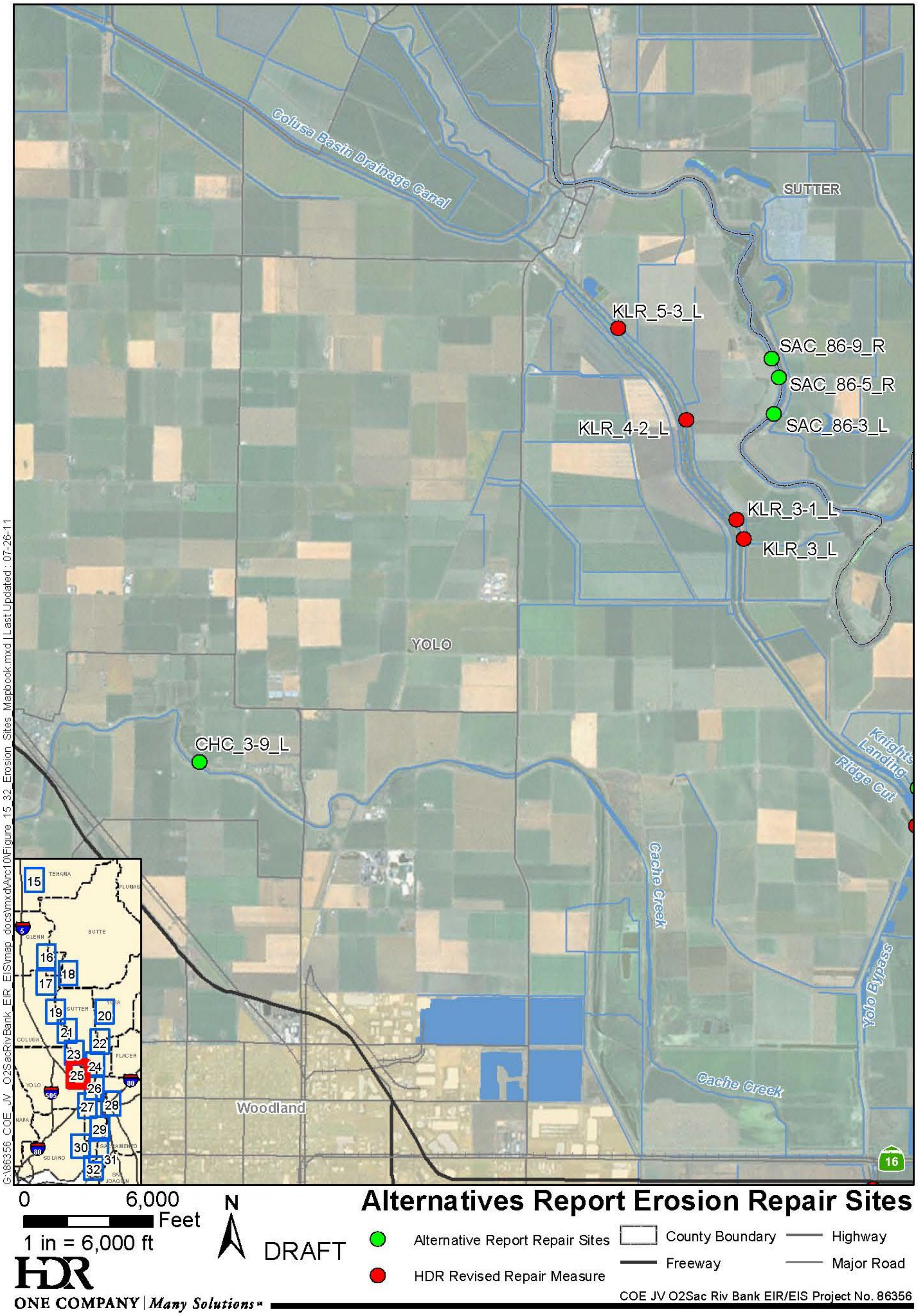


Figure 19- Alternatives Report Erosion Sites

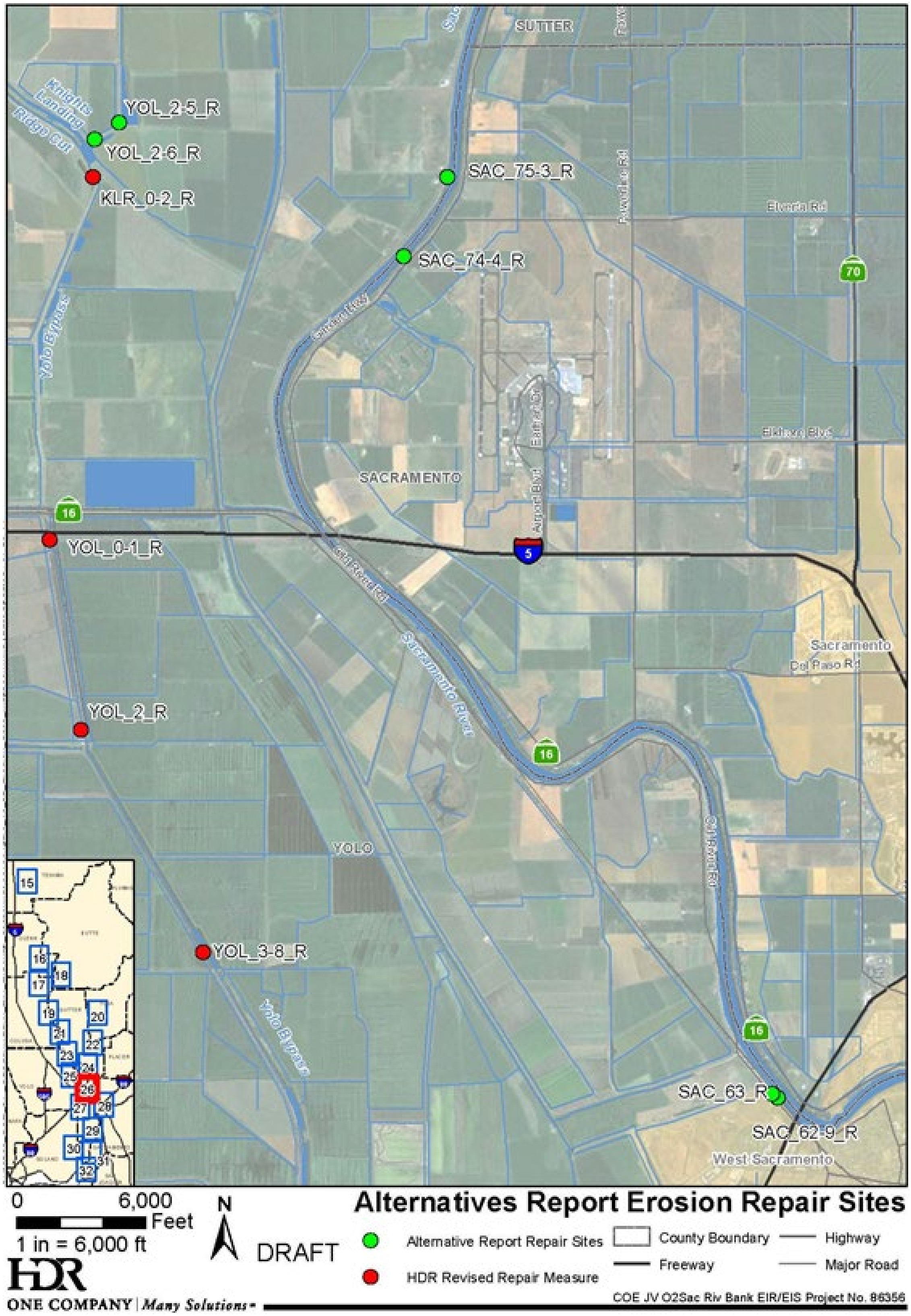


Figure 20- Alternatives Report Erosion Sites

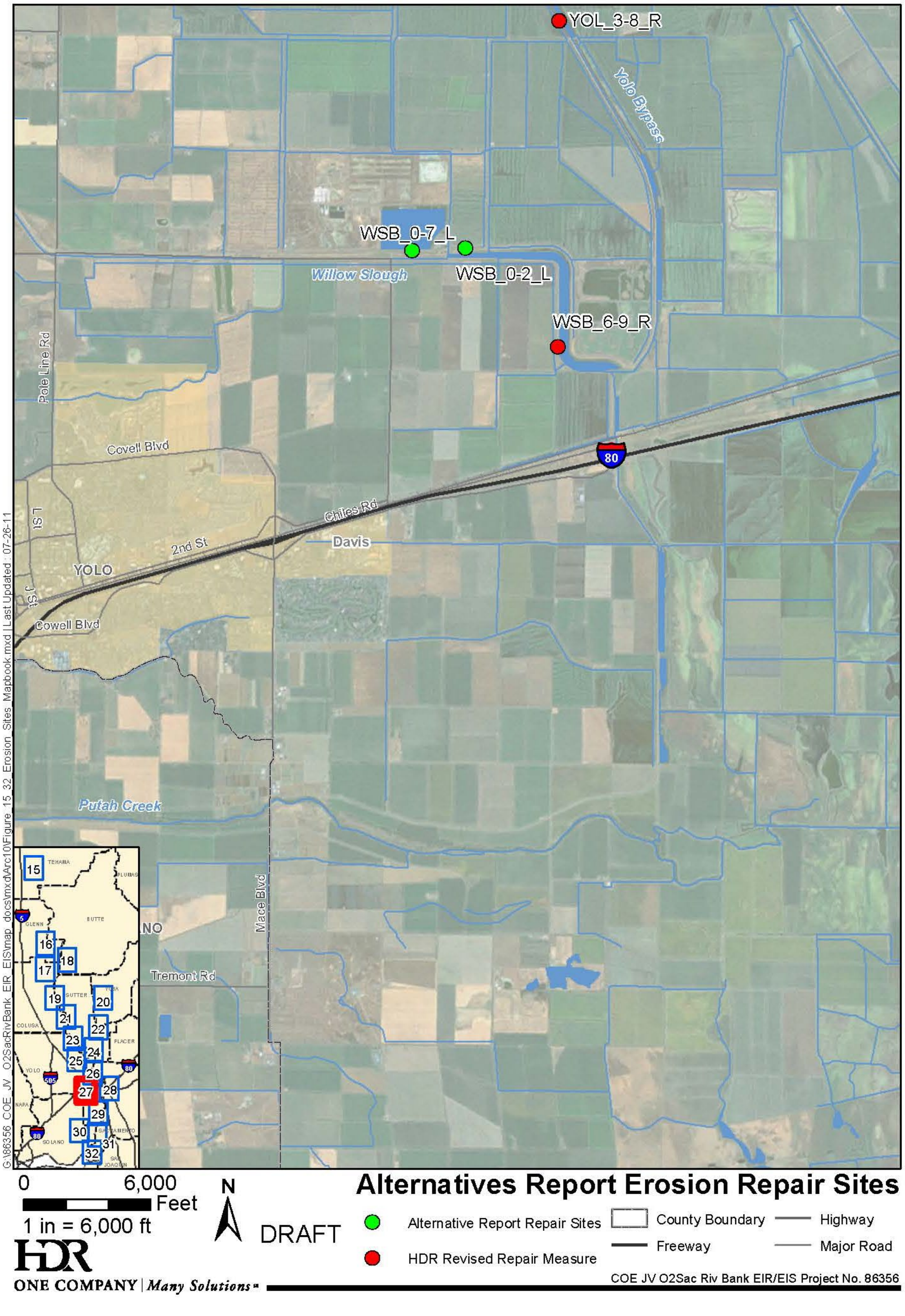


Figure 21 -Alternatives Report Erosion Sites

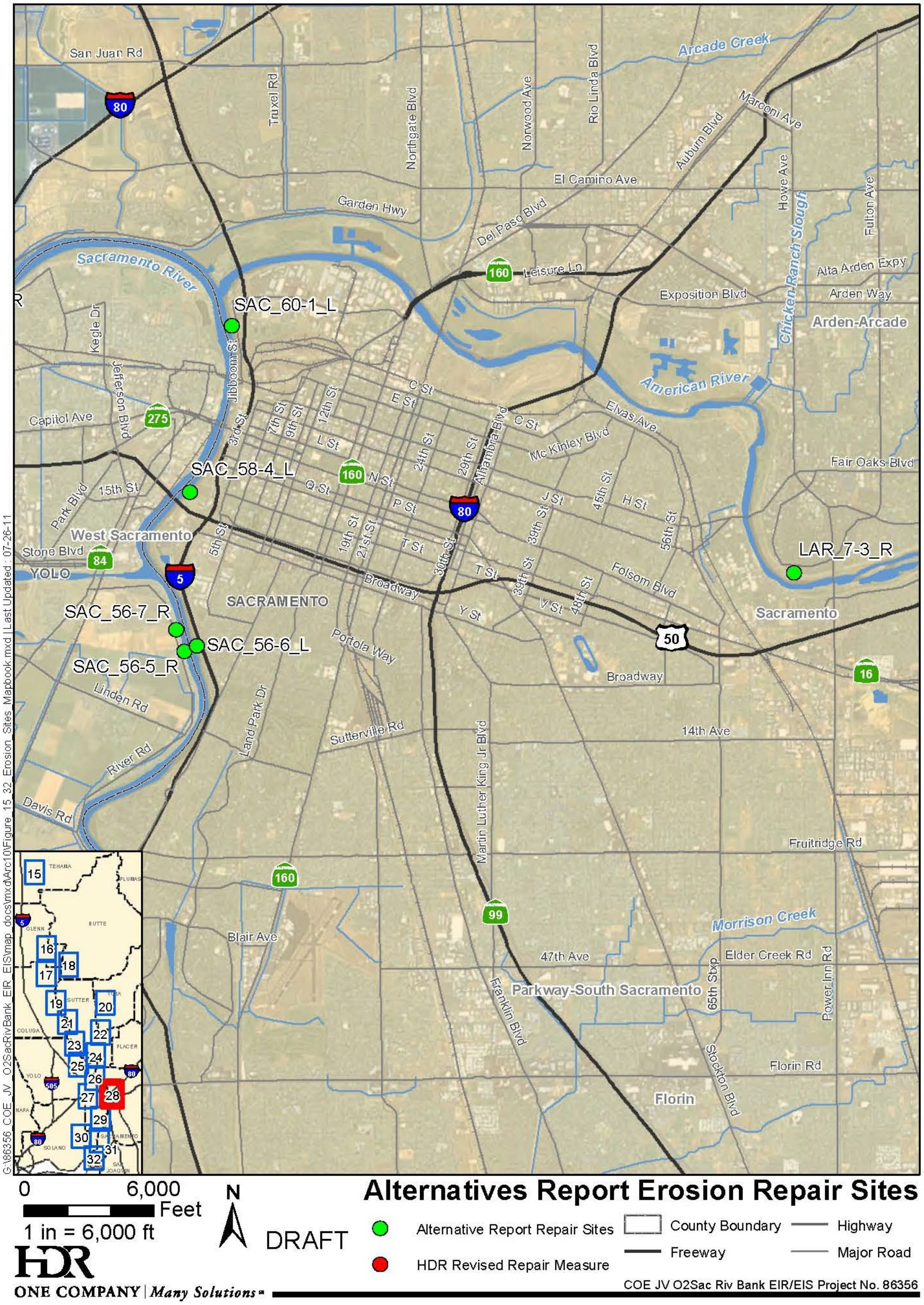


Figure 22- Alternatives Report Erosion Sites

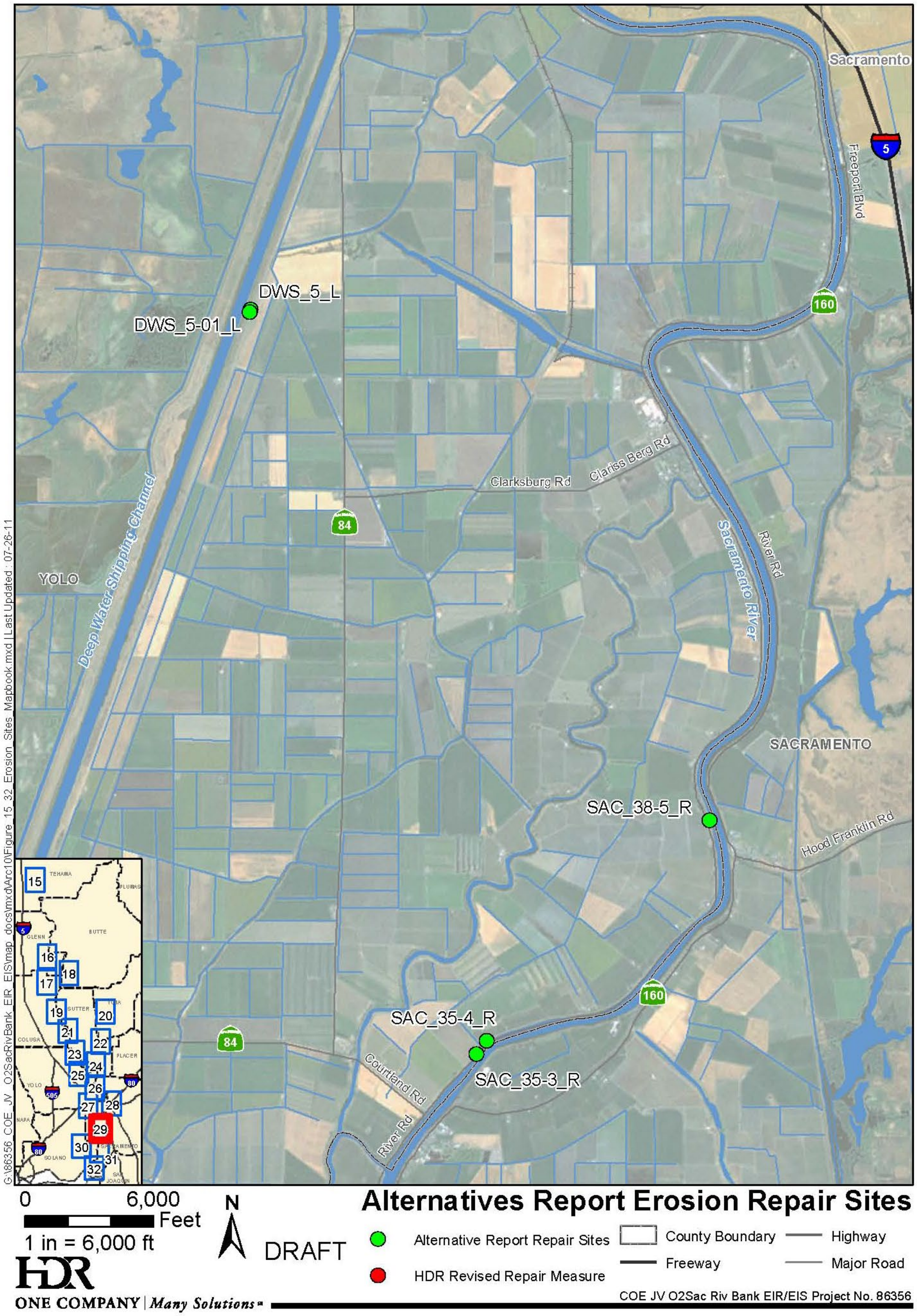


Figure 23- Alternatives Report Erosion Sites

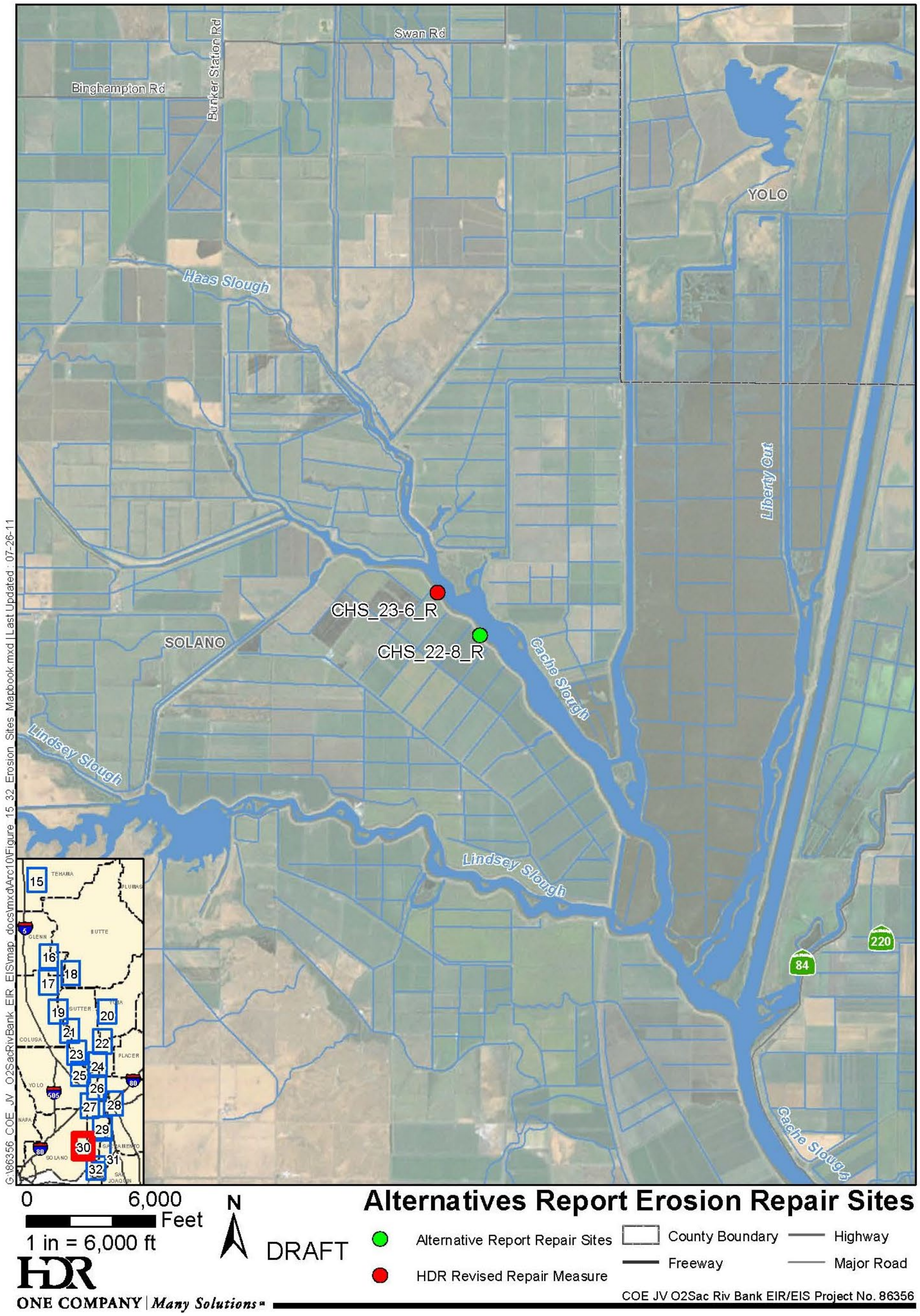


Figure 24- Alternatives Report Erosion Sites

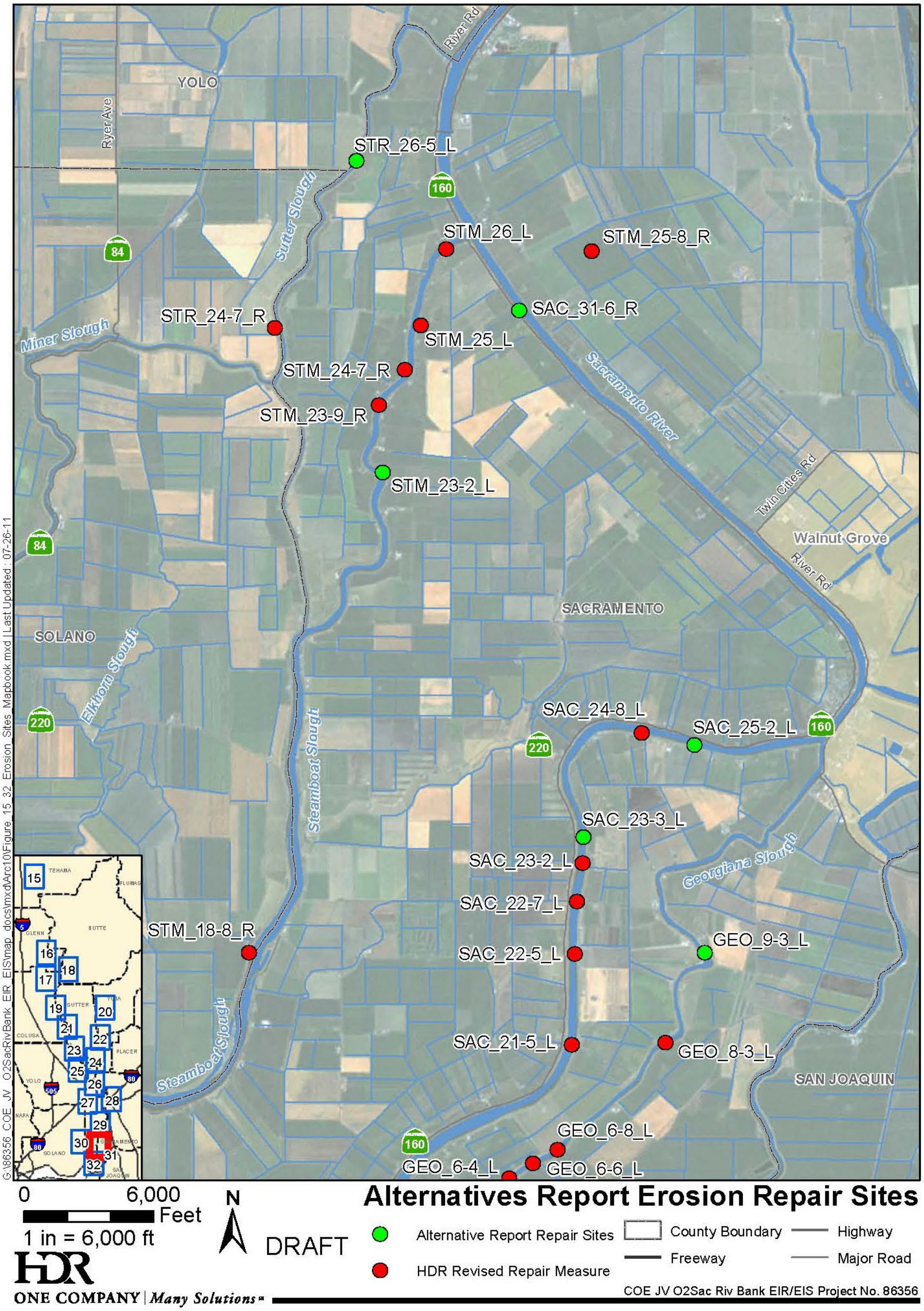


Figure 25- Alternatives Report Erosion Sites

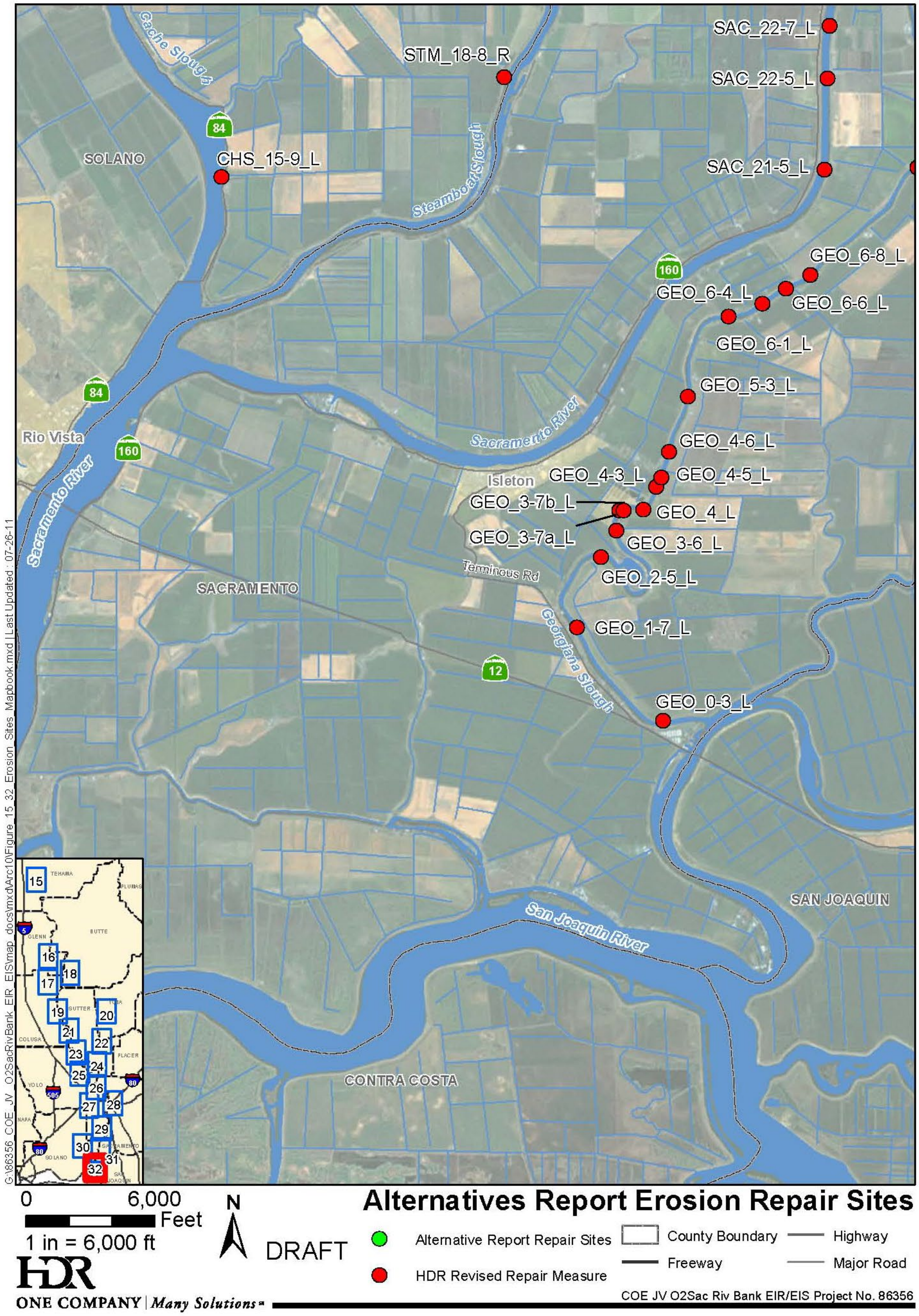
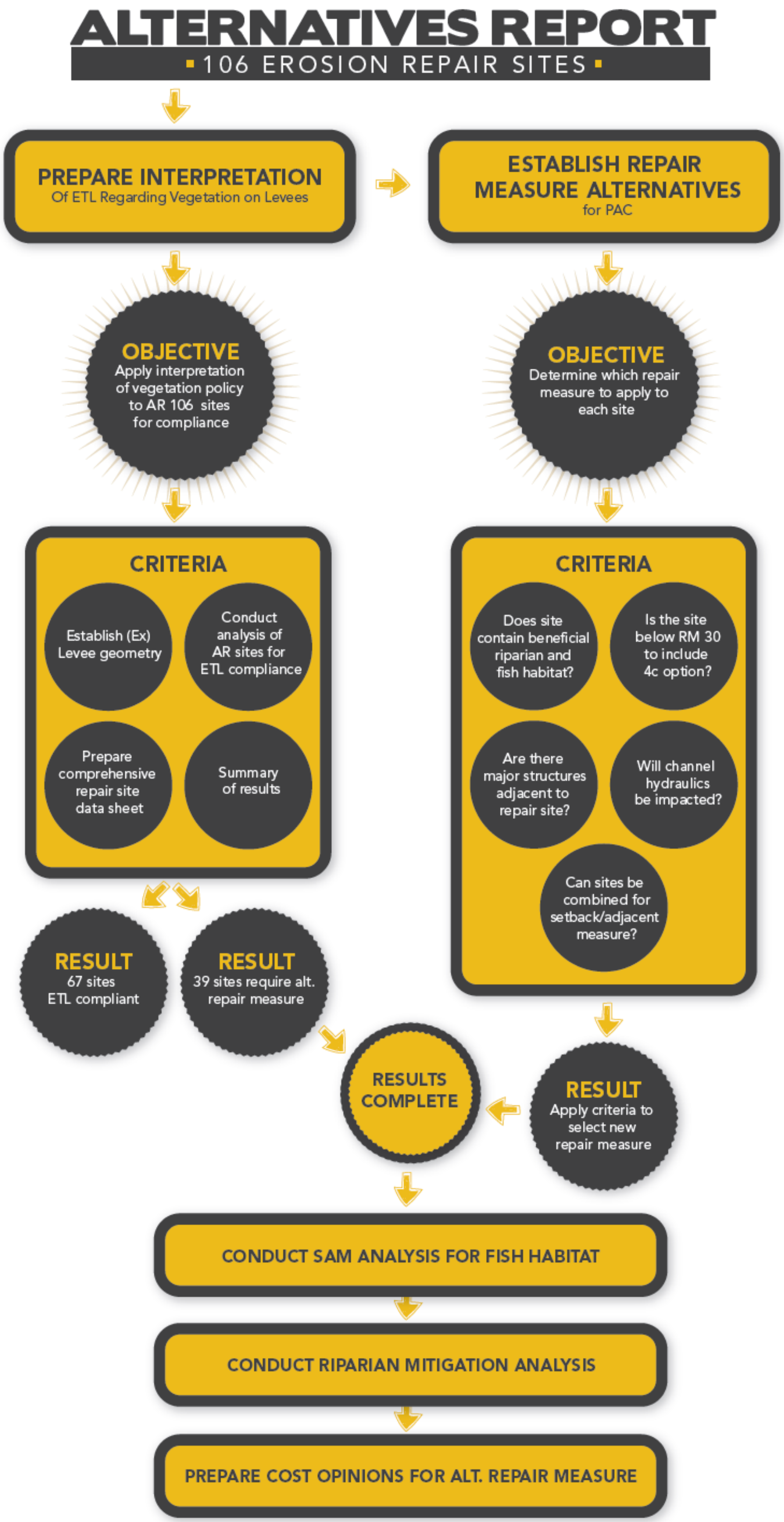


Figure 26 - Network Diagram of Process



6.0 Plan Alternatives

This Engineering Appendix describes a single bank protection programmatic plan, the PSI. However, NEPA and CEQA generally require that an EIS and EIR, respectively, consider a range of alternatives that would attain most of the basic project purpose, need, and objectives while avoiding or substantially lessening project effects. A range of reasonable alternatives is analyzed to define the issues and provide a clear basis for choice among the options. The NEPA and CEQA analysis also analyzes a no-action or no-project alternative.

In addition to a no-action alternative, five action alternatives with five sub-alternatives were analyzed. The five action alternatives apply site-specific bank protection measures (design alternatives) to each of the 106 representative erosion sites in the PSI. The site-specific bank protection measure applied to each site varies from one alternative to another.

A description of the alternatives is in the PACR and the Programmatic EIS/EIR.

6.1 Intra-Group Efficiencies

The PSI demonstrates how intra-group efficiencies may take place. By grouping geographically clustered sites, construction at one site could provide benefits to, or facilitate bank protection at a neighboring site. To realize these efficiencies, a commitment is required to view the river as a system and plan groupings of bank protection and mitigation sites, rather than designing and constructing on an individual site-by-site basis. Advantages of a systematic approach are:

- Ability to use one site as off-site mitigation for one or more other sites. Example is a setback levee that would provide ecosystem benefits that could off-set losses at another site). Other sites might be stone protection.
- Provide mitigation in advance of environmental impacts caused by bank protection.

6.2 Operations and Maintenance

Generally, operations and maintenance (O&M) for bank protection sites will include periodic inspections, repair of bank protection if there is erosion undermining or otherwise damaging the bank or levee, maintenance of vegetation on banks and in floodplains created by setback levees, and inspection and maintenance of off-site mitigation areas. Bank protection O&M is in addition to on-going SRFCP levee inspection and maintenance.

O&M requirements of bank protection generally coincide with the O&M requirements of the SRFCP. The SRFCP is divided into 65 levee maintenance units. There is an O&M manual for each unit. These are supplemental manuals to the overall Standard Operations and Maintenance Manual which covers the entire SRFCP. Upon construction of bank protection, the supplemental manual that includes that site is updated. **EA Appendix A7, Standard Procedure for Updating the Sacramento River Flood Control Project Supplemental O&M Manuals**, describes how the supplemental manuals are updated.

6.3 Construction Schedule

Construction of the Phase II 80,000 LF of bank protection is scheduled to begin in 2023. Historically, a good rule of thumb for the SRBPP was that bank protection was constructed at about 8,000 LF per year, however since 1990 that number has been about 3,300 LF on average. At this rate, construction of the 80,000 LF is estimated to be completed in 24 years from start of construction. A series of specific, supplemental DDRs will include specific Real Estate plans and specific NEPA/CEQA documents.

Repair of critical erosion sites will be expedited as much as possible. Some sites may require a more extensive design process, or longer permitting process. This could include setback levees, sites with challenging engineering or environmental considerations, or schedule delays. Repairs will continue at other sites if these critical erosion sites experience delays so as not to delay erosion repair at other critical erosion sites.

The schedule for repairing a single erosion site or constructing a setback levee will vary on a site by site basis. The schedule depends on a number of factors including the measure selected, site length, bank width, accessibility, environmental constraints, planting factors, and other factors unique to each site.

6.4 Deviations from Priority Site Inventory during Implementation

As discussed earlier, the PSI is a representation of 80,000 LF of bank protection. The actual constructed bank protection will be different. The PSI demonstrates how bank protection meets project goals, complies with Corps policy and environmental regulations, and it serves as a valuable starting point to guide implementation of the bank protection program. The program will evolve to adapt to changes in erosion, environmental, and market conditions, and revisions to policy.

Possible anticipated changes to the plan are listed below:

- As erosion problems vary year to year, the bank protection plan will adapt to changing conditions. The annual surveys may identify erosion sites as critical if erosion problems worsen at a particular site. Other sites will be removed as an erosion sites once they are repaired.
- Detailed explorations, surveys, and hydraulic modeling of sites could result in revisions to the erosion protection designs or changes to measures themselves.
- Detailed designs and real estate appraisals, and changes to market prices could revise cost estimates.
- As discussed above, the PSI complies with EP 1110-2-18, with no variances. Currently, no variances apply for the PSI. If variances were requested and granted, it could relax the extent of the vegetation removal, increase vegetation and/or in-stream woody material placement, or result in revised measures.
- Mitigation requirements could change due to revisions to bank protection measures and more detailed field surveys and analysis are completed. Supplemental Biological Opinions and NEPA-CEQA documents will be developed during the implementation design phase.
- The construction schedule is not fully determined and is subject to change. Funding, the number and extent of selected critical erosion sites, and complexity of detailed planning and design are factors that influence schedule.

Engineering Appendix Sub-Appendices

Sub-Appendix A1. Civil Design

Levee Geometry Technical Memorandum

Hydraulic Evaluation Technical Memorandum

Sub-Appendix A2. Cost Engineering

Total Project Cost Summary

Cost and Schedule Risk Analysis Report

Sub-Appendix A3. Geotechnical

Sub-Appendix A4. Hydrology

Sub-Appendix A5. Hydraulics

Sub-Appendix A6. Real Estate Maps

Sub-Appendix A7. O&M Manual Template

Sub-Appendix A1. Civil Design

Levee Geometry Technical Memorandum

Hydraulic Evaluation Technical Memorandum

Levee Geometry
Technical Memorandum

Sacramento River Bank Protection Project
Sacramento River Basin, California

U.S. Army Corps of Engineers, Sacramento District

June 2011

Prepared By:

HDR Engineering, Inc.

2365 Iron Point Road, Suite 300

Folsom, CA 9563

This page intentionally left blank.

1.0 Levee Geometry

Levee cross section geometry is critical to overlaying the vegetation free zone per the ETL and the Framework Memo. The geometry is also important for developing quantities for cost estimates. In addition, the waterside hinge point elevations relative to seasonal mean water surface elevations are critical when figuring how the treatment of the waterside of levees impacts fish habitat.

This evaluation relied on AR cross section elevation data from the AR and the Sacramento San Joaquin Basins Comprehensive Study of 2002 by USACE, and on field notes from the AR. This section only describes how elevation data for the cross sections were derived for a geometrical analysis, actual design will include an geotechnical and hydraulic analysis along with site specific conditions.

1.1 Comprehensive Repair Site Data Sheet

A Comprehensive Repair Site Data Sheet (data sheet) was prepared for each of the 107 erosion repair sites. The data sheets can be found in **Appendix A**. Each data sheet contains the information derived from the available technical resources and is presented as three individual details, labeled DETAIL1, DETAIL 2 and DETAIL3. A description of each detail is presented below.

1.1.1 DETAIL 1

This section of the data sheet presents the typical erosion repair cross section within the upstream and downstream limits of the site. The title of each data sheet describes the repair site location. For example; “Cache Creek 3.9L” describes the left bank of the Cache Creek tributary at river mile 3.9. The cross section presented is considered the worst case scenario of bank erosion along the extents of the individual repair site.

The cross section which was derived from the information provided in the AR contains the existing levee geometry modified by erosion and is denoted as a shaded dashed line, along with the AR recommended erosion repair surface which is denoted as a bold black line. Each cross section contains dimensions which denote the limits of the VFZ relative to the waterside of the levee. It is understood that the ETL establishes a VFZ across the entire levee prism which includes the landside of the levee, but this analysis, based on the Framework Memo, was limited to the portion of the levee that is being recommended for repair.

The two dots in the cross section represent the location of the landside and waterside toes. These are critical points that must be established in order to define the limits of the VFZ. The landside toe was established based on the information provided below under detail 2 while the waterside toe was established based on information provided from both detail 2 and 3.

For a more detailed explanation of the process for defining the VFZ refer to the Framework Memo.

1.1.2 DETAIL 2

Because there was no field investigation conducted by HDR as part of this analysis, and the field notes provided in the AR only present detailed information on the waterside of the levee prism, it was necessary to utilize other technical resources to establish the elevation of the landside toe. This section of the data sheet presents a summary of the method and key components used to determine the landside toe as well as establish the elevation of the waterside toe. In short, the VFZ is defined as the area between a point beginning 15 feet landward of the landside toe to a point 15 feet waterward of the waterside toe. Therefore, two critical points necessary for establishing the limits of the VFZ are the landside and waterside levee toes.

The most current and accurate technical resource available is the Comprehensive Study prepared by the USACE. This study provides a cross section at random intervals along the Sacramento River and its tributaries. These intervals range from 1000 feet to 15,000 feet. Each cross section contains an elevation point at the landside toe, the landside and waterside hinge point of the levee crown and all critical grade break elevations on the waterside and landside of the levee. A critical grade break would be characterized as an existing riparian bench or some other large waterside feature. The landside would include seepage berms or stability berms.

The first step in HDR's analysis was to establish the location of the actual repair site in relation to a known cross section provided in the Comprehensive Study. In some instances, a Comprehensive Study cross section was available at or near the actual repair site location. In these cases the data from that individual cross section was used to establish the landside toe elevation. In other instances, the repair site was not located near a known Comprehensive Study cross section which would place the repair site some incremental distance between two individual cross sections. In these cases, the landside toe elevation at the location of the repair site was interpolated based on the data provided by each upstream and downstream cross section. The result of this analysis is shown graphically in Detail 2 under the title "CROSS-SECTION FROM DWR COMPREHENSIVE STUDY". Each cross section is shown graphically and labeled as "upstream x-section", "downstream x-section" and "repair site x-section" if interpolated. If not, only the "repair site x-section" is provided.

The second step in this analysis was to determine the actual elevation of the landside toe relative to the cross section provided in the AR. A summary of this procedure is presented in Detail 2 under the title "KEY DATA FOR DETERMINING LANDSIDE TOE". When reviewing the data provided in the Comprehensive Study and the elevation information provided in the AR, it was evident that there were a minor discrepancies in the elevations at the repair site location. This discrepancy ranged from two to five feet in elevation. Because of this discrepancy the estimated Comprehensive Study toe elevation was not used. Instead, the elevation difference between the Comprehensive Study crown and landside levee toe was calculated. This elevation difference was then subtracted from the AR crown elevation to determine the elevation of the landside toe relative to the AR repair site. This revised elevation was then applied to the cross section in Detail 1 as the proposed landside toe.

1.1.3 DETAIL 3

Because of the existing geometry of the waterside slope and in most instances heavy vegetation and emergent benches that may be manmade, it is difficult, if not impossible, to establish the location of the waterside toe by observation. In addition, the existing geometry of the waterside slope has been altered by some form of bank erosion, in effect displacing the location of the pre-eroded toe location. In an effort to recreate the existing eroded waterside slope geometry as shown in Detail 1, the AR field notes were used and are presented in Detail 3 for reference.

1.1.4 Levee Geometry Summary

The critical elements necessary for conducting a comprehensive analysis and preparing an accurate representation of the existing levee geometry with regard to requirements presented in the ETL are the landside toe, waterside toe, levee crown waterside hinge point, and the geometry of the waterside slope. Each of the aforementioned elements has been established based on the preceding discussion on Details 1, 2 & 3. The final element needed to complete the geometry of the existing levee cross section is establishing the waterside toe. This point is not apparent by inspection; it is actually a point that must be established by determining the landside toe.

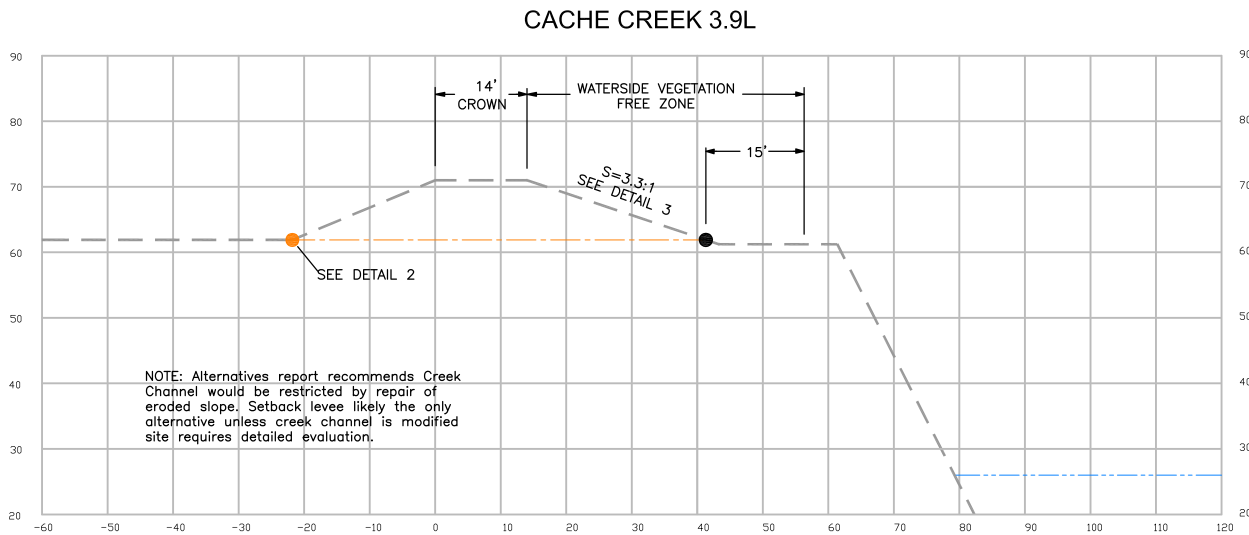
As mentioned in the preceding discussion the waterside toe has been eroded, sediment may have been deposited or soil has been placed over the waterside toe to create a waterside bench or for a previous repair. Because of this, the waterside toe must be established by identifying known points, and then assuming various projections of those points.

The first critical point to establish is the landside toe which is located at the intersection of the existing ground and the landside slope. To establish the elevation of the waterside toe, the elevation of the landside toe is projected infinitely in the waterward direction. Without borings of the repair site and conducting a soils analysis it was assumed that the elevation of this line would have been the elevation of the existing river bank prior to constructing the levee, and serves at the horizontal element needed to establish the waterside toe.

The second critical point is the waterside levee crown hinge point. This is the point of beginning for the waterside slope projection, which can either be an actual slope or an assumed slope depending on the existing condition of the waterside bank geometry. In either case, the minimum slope projection is a 2:1 ratio if the existing slope was greater; if less than a 2:1 ratio, the actual slope was projected. Once the slope ratio was defined, that line was projected from the waterside hinge point to the previously projected original ground elevation; this intersect is considered the waterside toe and is presented as a blue dot in Detail 1.

Levee Geometry Technical Memorandum Appendix A: Comprehensive Repair Site Data Sheets

DETAIL 1



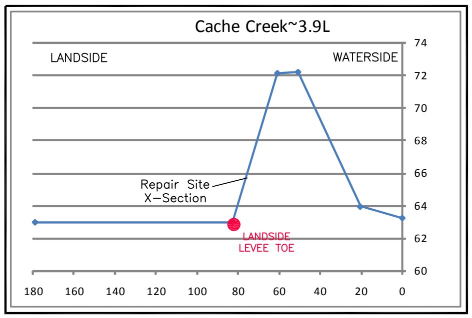
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	63.3
20.5	64.0
50.8	72.2
60.9	72.1
82.7	63.0
178.9	63.0

KEY DATA FOR DETERMINING LANDSIDE TOE

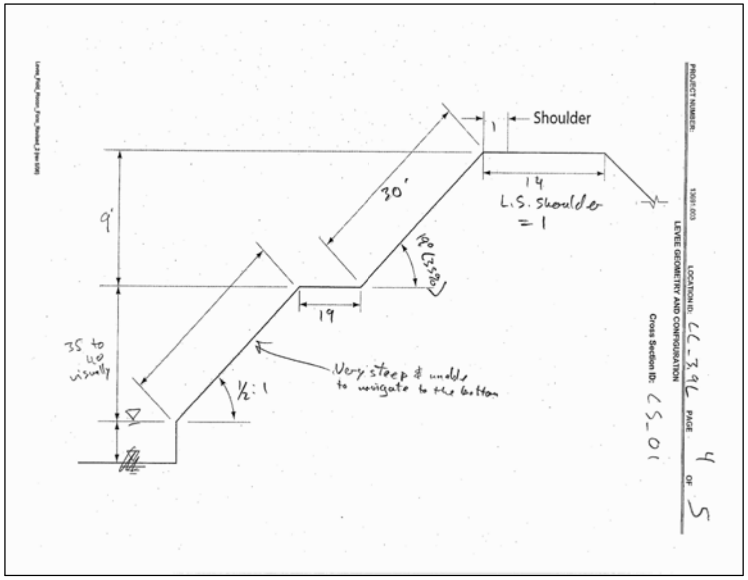
Comp Study Levee Crown Elevation	72.1
Comp Study Landside Toe Elevation	63.0
Comp Study Difference	9.1

Alternatives Report Analysis Levee Crown Elevation	71.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	61.9

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

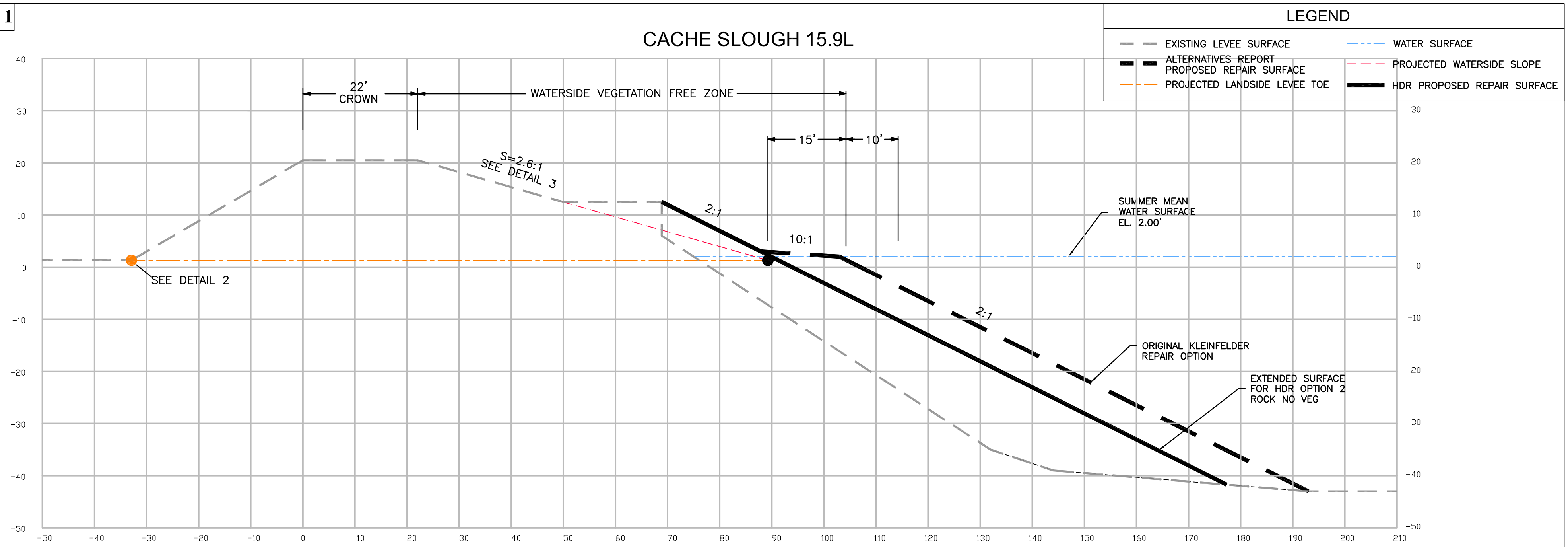
Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

CACHE CREEK ~ 3.9L

Date 3/31/11

Figure CC 3.9L

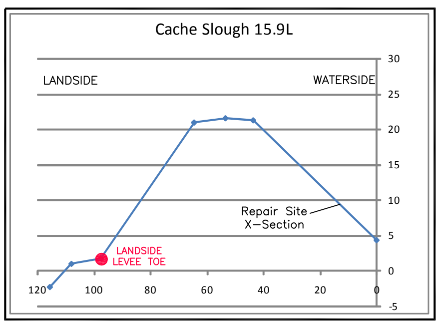
DETAIL 1



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	4.3
43.7	21.3
53.5	21.6
64.6	21.0
97.4	1.8
107.9	1.0
115.6	-2.3

KEY DATA FOR DETERMINING LANDSIDE TOE

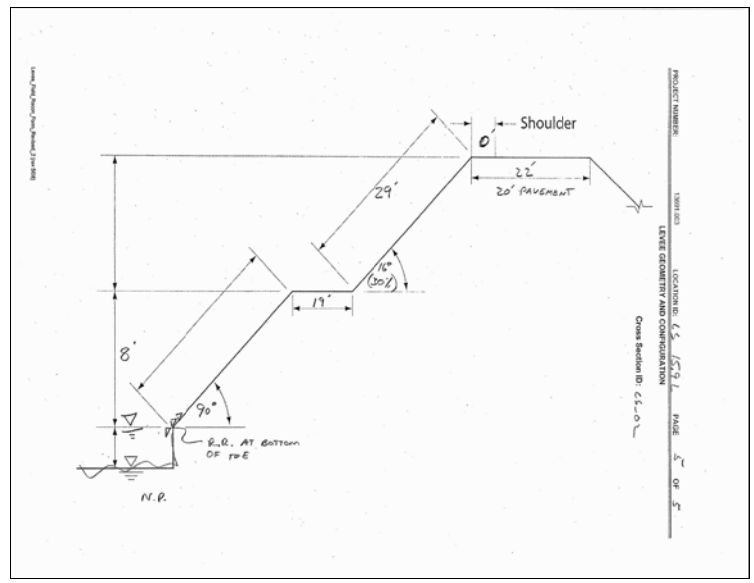
Comp Study Levee Crown Elevation	21.0
Comp Study Landside Toe Elevation	1.8
Comp Study Difference	19.2

Alternatives Report Analysis Levee Crown Elevation	20.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	1.3

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

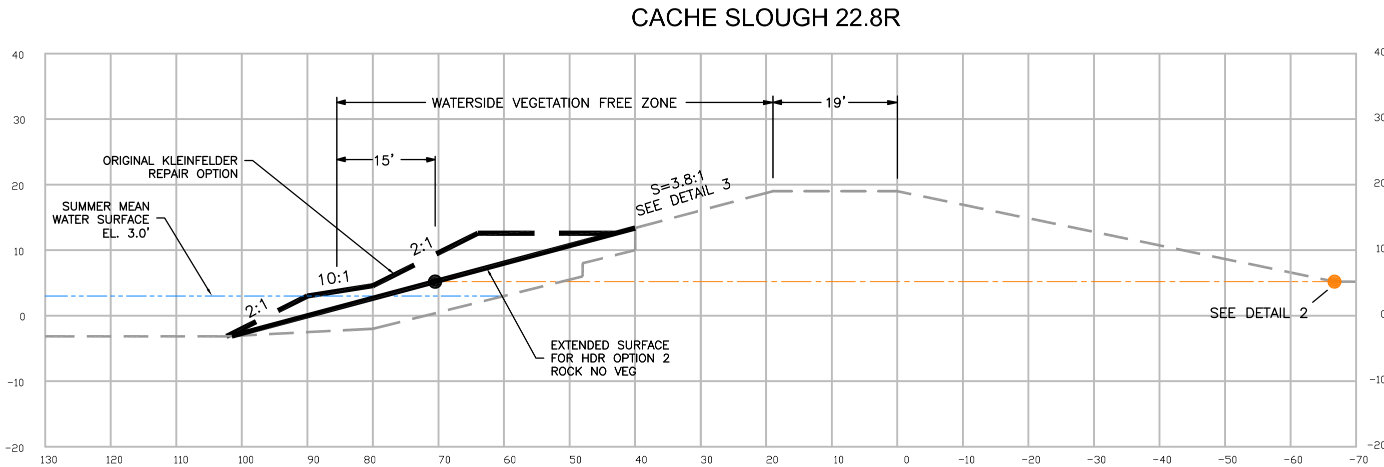
HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

CACHE SLOUGH ~ 15.9L

Date	3/31/11
Figure	CS 15.9L

DETAIL 1



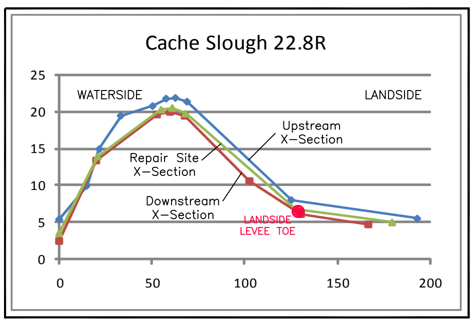
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE
- HDR PROPOSED REPAIR SURFACE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	3.5
20.8	14.0
55.1	20.4
61.1	20.6
68.2	19.9
127.8	6.8
179.8	5.0

KEY DATA FOR DETERMINING LANDSIDE TOE

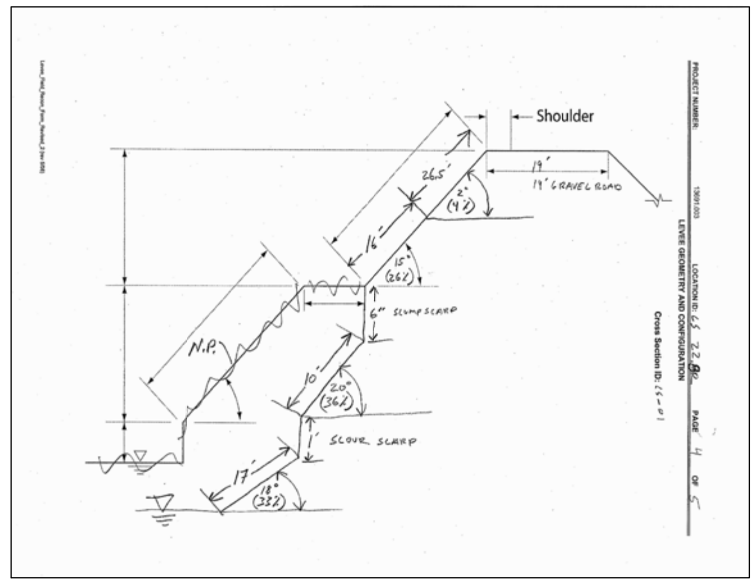
Comp Study Levee Crown Elevation	20.6
Comp Study Landside Toe Elevation	6.8
Comp Study Difference	13.8

Alternatives Report Analysis Levee Crown Elevation	19.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	5.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

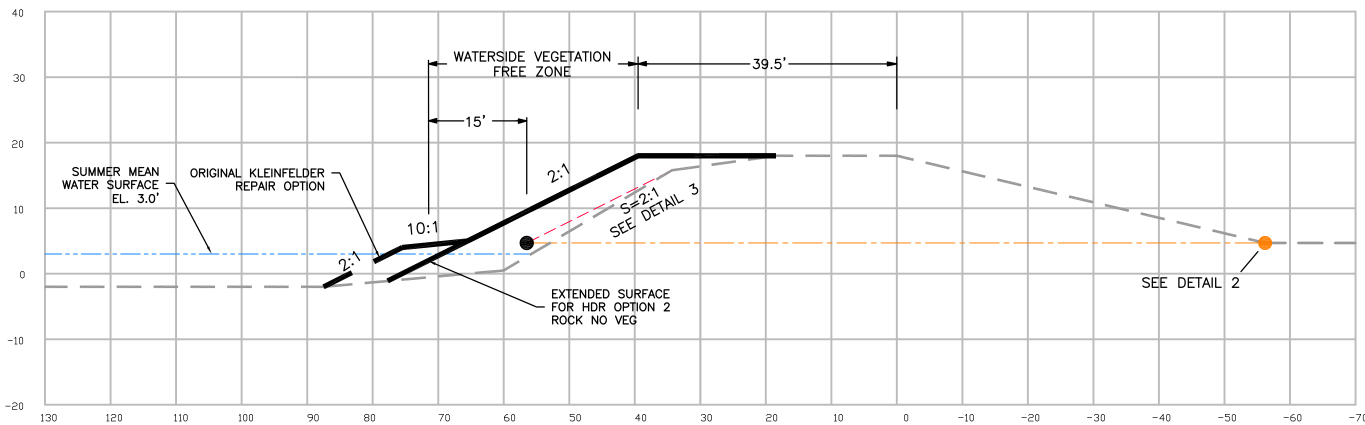
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

CACHE SLOUGH ~ 22.8R

Date	3/31/11
Figure	CS 22.8R

DETAIL 1

CACHE SLOUGH 23.6R



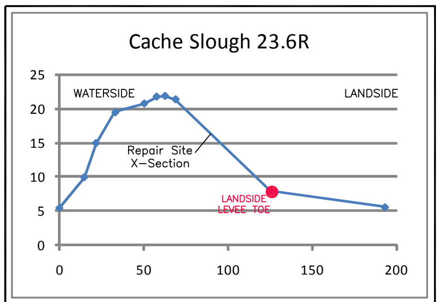
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE
- HDR PROPOSED REPAIR SURFACE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.5
14.7	10.0
21.7	15.0
33.2	19.5
50.4	20.8
57.7	21.8
62.7	21.9
69.0	21.4
125.2	8.1
193.2	5.6

KEY DATA FOR DETERMINING LANDSIDE TOE

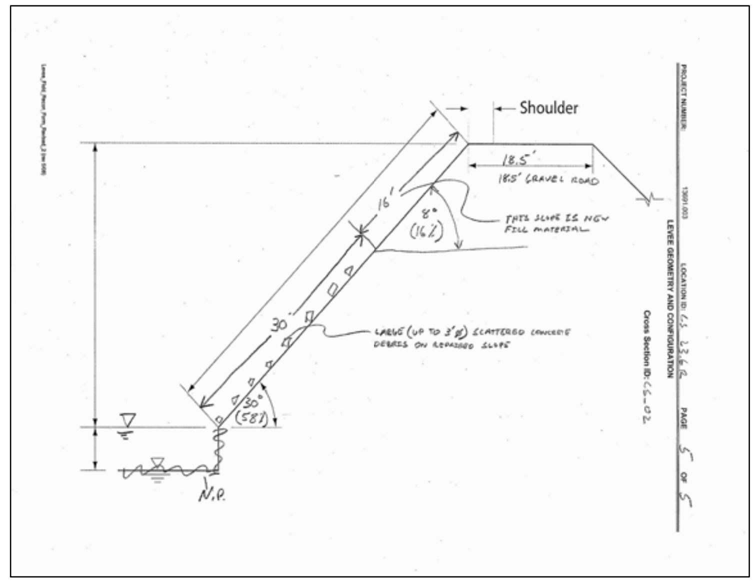
Comp Study Levee Crown Elevation	21.4
Comp Study Landside Toe Elevation	8.1
Comp Study Difference	13.3

Alternatives Report Analysis Levee Crown Elevation	18.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	4.7

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

CACHE SLOUGH ~ 23.6R

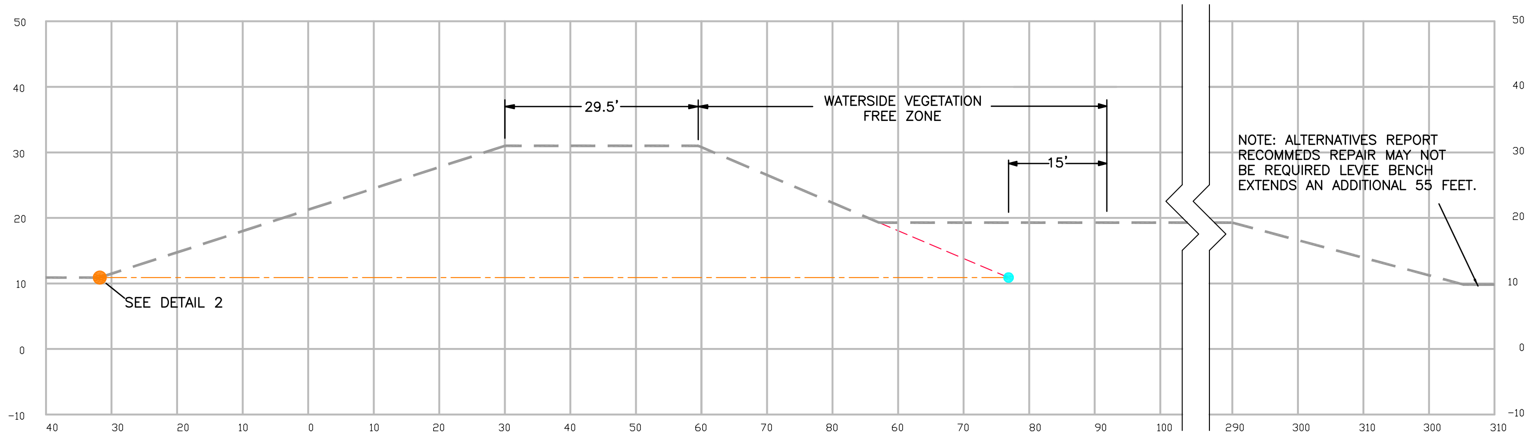
Date	3/31/11
Figure	CS 23.6R

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

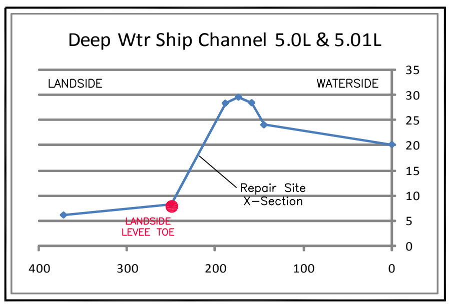
DEEP WATER SHIP CHANNEL 5.01L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	20.3
145.1	24.2
159.0	28.6
174.0	29.7
189.0	28.5
250.8	8.4
372.4	6.3

KEY DATA FOR DETERMINING LANDSIDE TOE

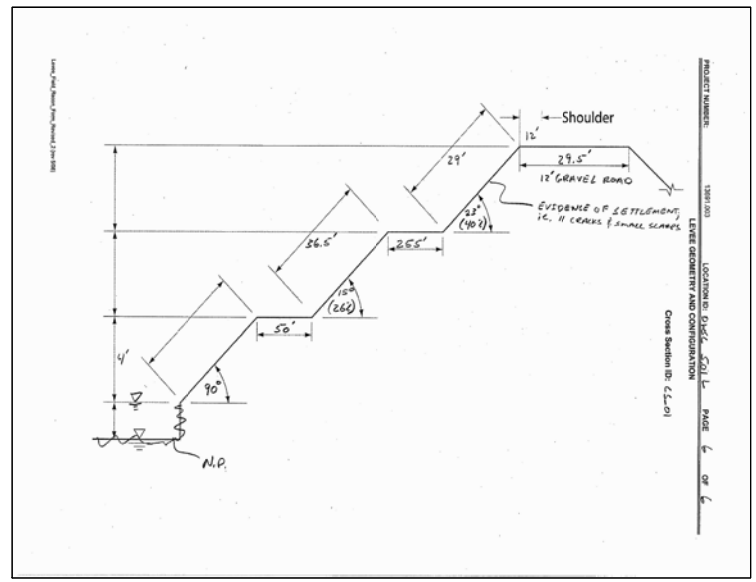
Comp Study Levee Crown Elevation	28.5
Comp Study Landside Toe Elevation	8.4
Comp Study Difference	20.1

Alternatives Report Analysis Levee Crown Elevation	26.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	5.9

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

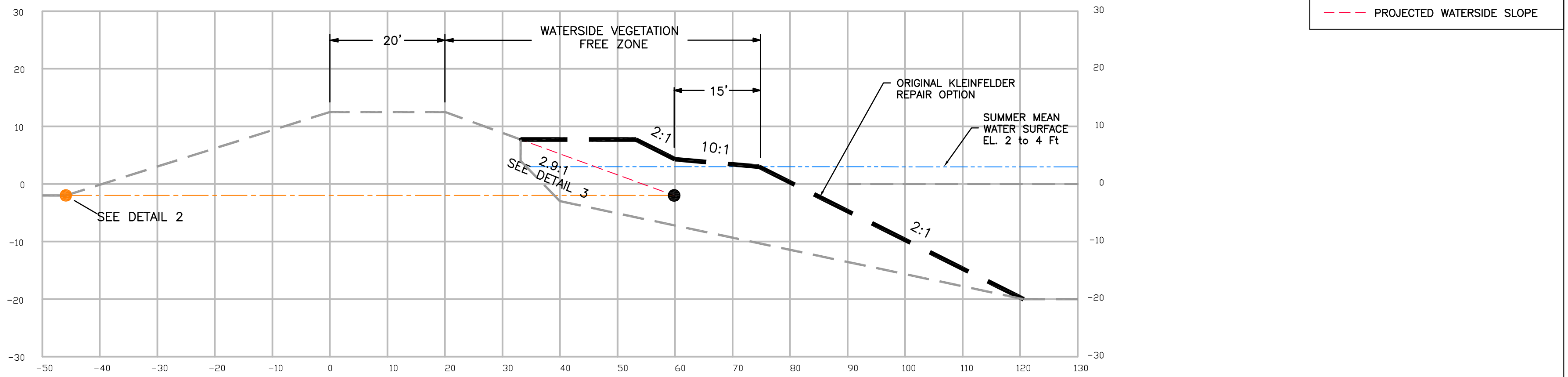
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

DEEP WTR SHIP CHANNEL ~ 5.01L

Date	3/31/11
Figure	DWSC 5.01L

DETAIL 1

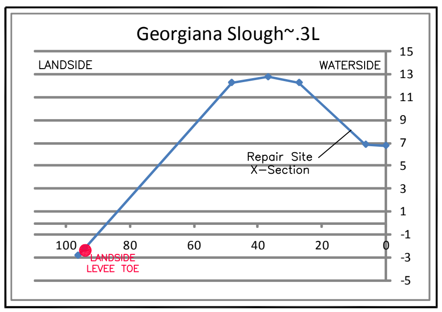
GEORGIANA SLOUGH 0.3L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	6.8
6.3	6.9
27.1	12.3
36.8	12.8
48.2	12.3
94.1	-2.2
96.2	-2.8

KEY DATA FOR DETERMINING
LANDSIDE TOE

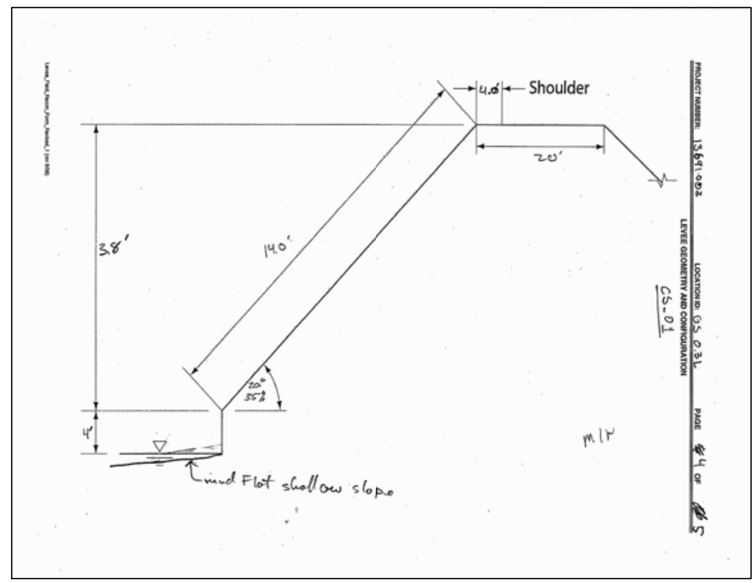
Comp Study Levee Crown Elevation	12.3
Comp Study Landside Toe Elevation	-2.2
Comp Study Difference	14.5

Alternatives Report Analysis Levee Crown Elevation	12.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	-2.0

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

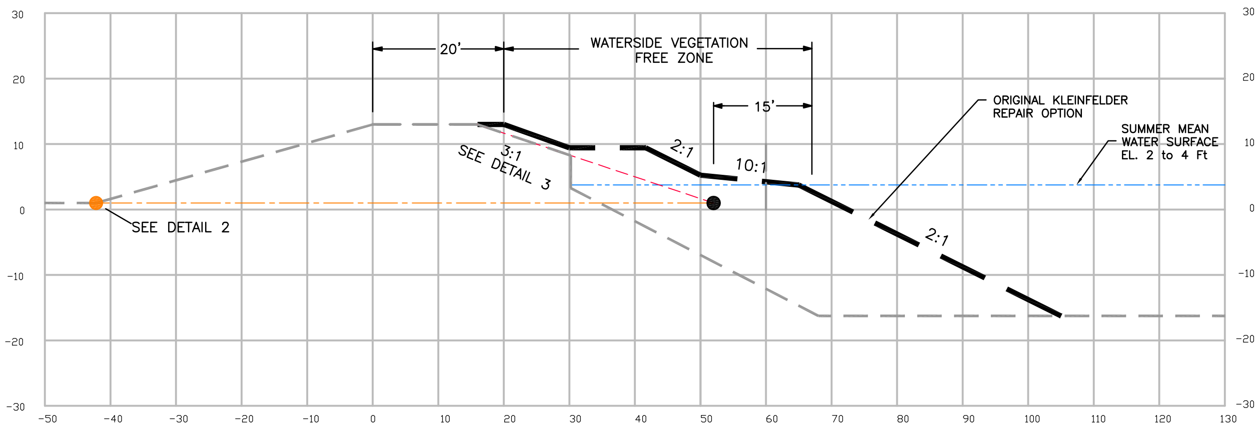
GEORGIANA SLOUGH ~ 0.3L

Date 3/31/11
Figure GS 0.3L

DETAIL 1

LEGEND
EXISTING LEVEE SURFACE
ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
PROJECTED LANDSIDE LEVEE TOE
WATER SURFACE
PROJECTED WATERSIDE SLOPE

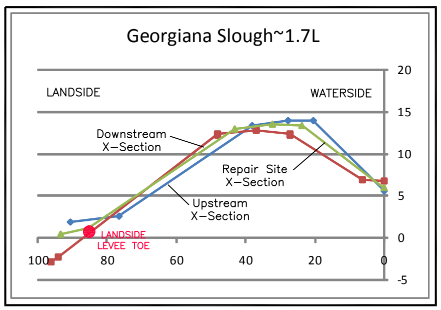
GEORGIANA SLOUGH 1.7L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	6.0
23.8	13.5
32.3	13.6
43.2	13.0
85.4	1.1
93.5	0.4

KEY DATA FOR DETERMINING LANDSIDE TOE

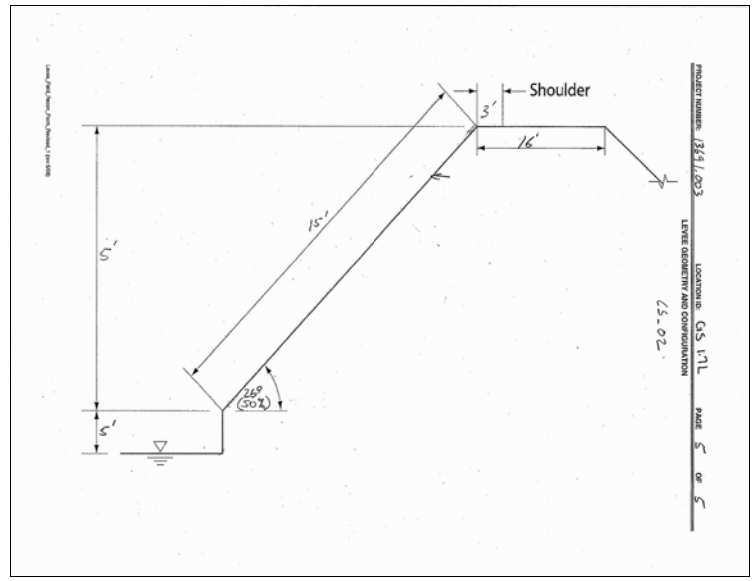
Comp Study Levee Crown Elevation	13.0
Comp Study Landside Toe Elevation	1.1
Comp Study Difference	12.0

Alternatives Report Analysis Levee Crown Elevation	13.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	1.0

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

GEORGIANA SLOUGH ~1.7L

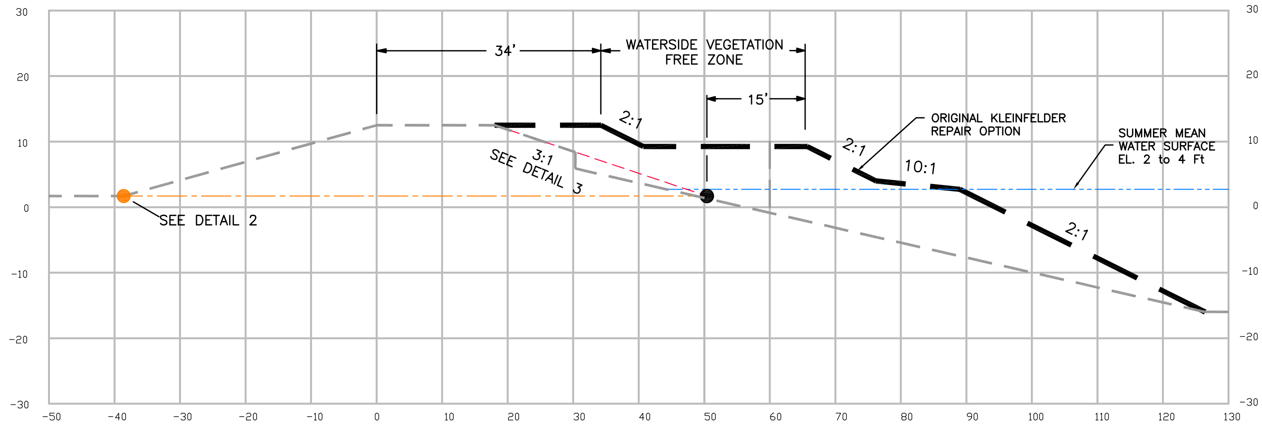
Date	3/31/11
Figure	GS 1.7L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

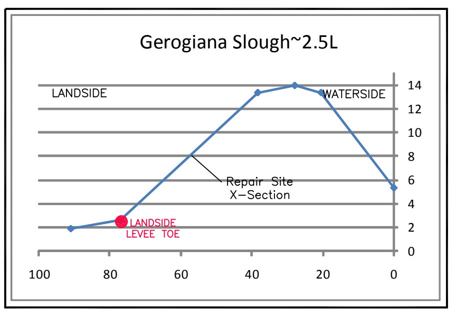
GEORGIANA SLOUGH 2.5L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.4
20.5	13.4
27.9	14.0
38.3	13.4
76.9	2.6
90.9	1.9

KEY DATA FOR DETERMINING LANDSIDE TOE

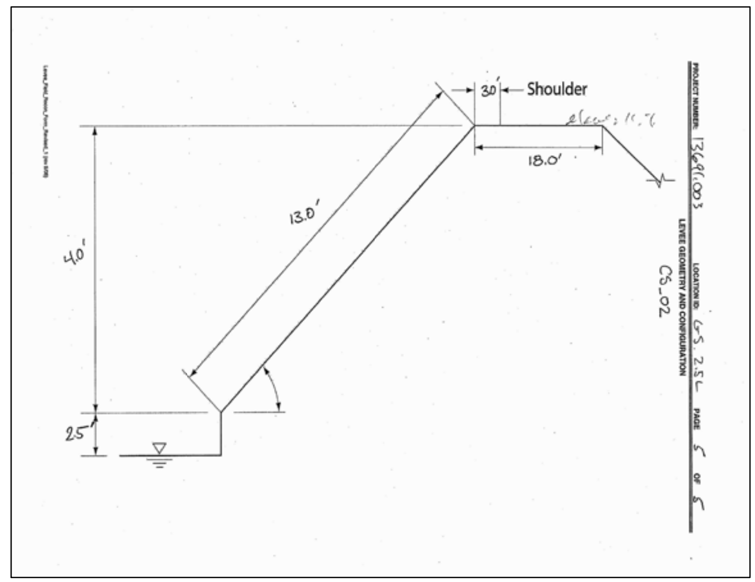
Comp Study Levee Crown Elevation	13.4
Comp Study Landside Toe Elevation	2.6
Comp Study Difference	10.8

Alternatives Report Analysis Levee Crown Elevation	12.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	1.7

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

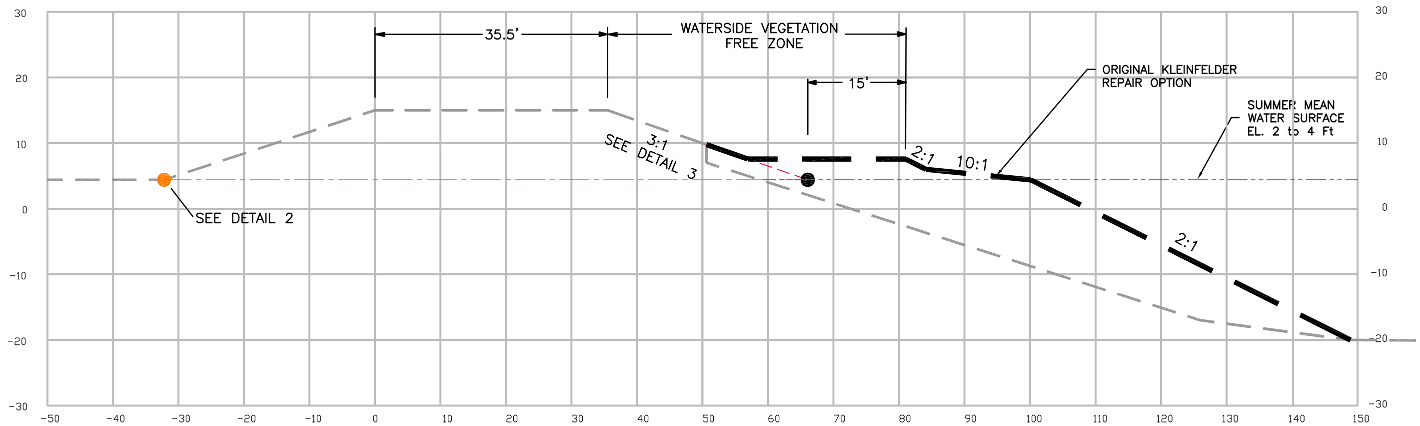
GEORGIANA SLOUGH ~2.5L

Date 3/31/11

Figure GS 2.5L

DETAIL 1

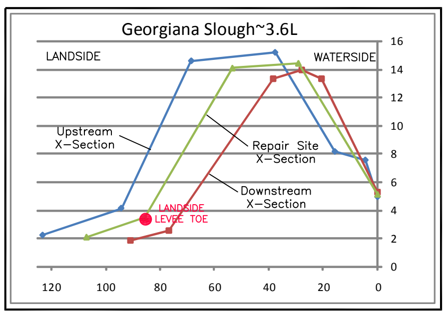
GEORGIANA SLOUGH 3.6L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.2
29.1	14.4
53.4	14.1
85.6	3.5
107.1	2.1

KEY DATA FOR DETERMINING LANDSIDE TOE

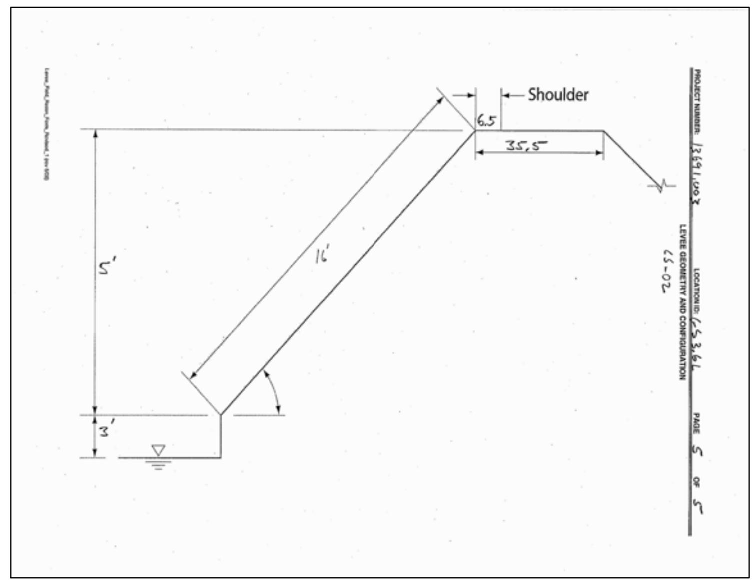
Comp Study Levee Crown Elevation	14.1
Comp Study Landside Toe Elevation	3.5
Comp Study Difference	10.6

Alternatives Report Analysis Levee Crown Elevation	15.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	4.4

DETAIL 3

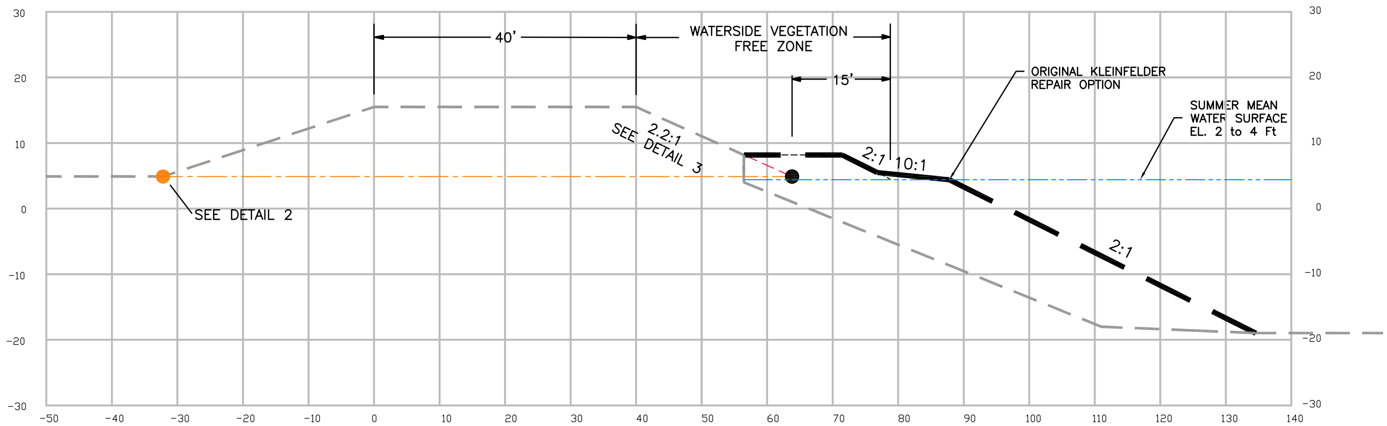
ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR HDR Engineering, Inc.	Drawn By: E. Smith Checked By: R. Little QA/QC Review: D. Teak	GEORGIANA SLOUGH ~3.6L	Date 3/31/11 Figure GS 3.6L

DETAIL 1

GEORGIANA SLOUGH 3.7aL



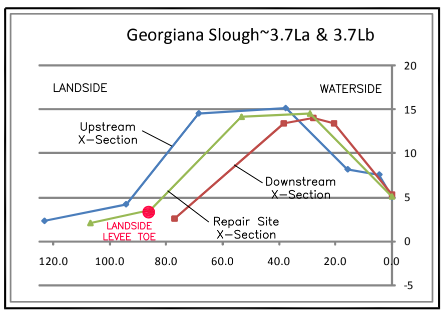
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.1
29.1	14.5
53.4	14.1
85.6	3.6
107.1	2.1

KEY DATA FOR DETERMINING LANDSIDE TOE

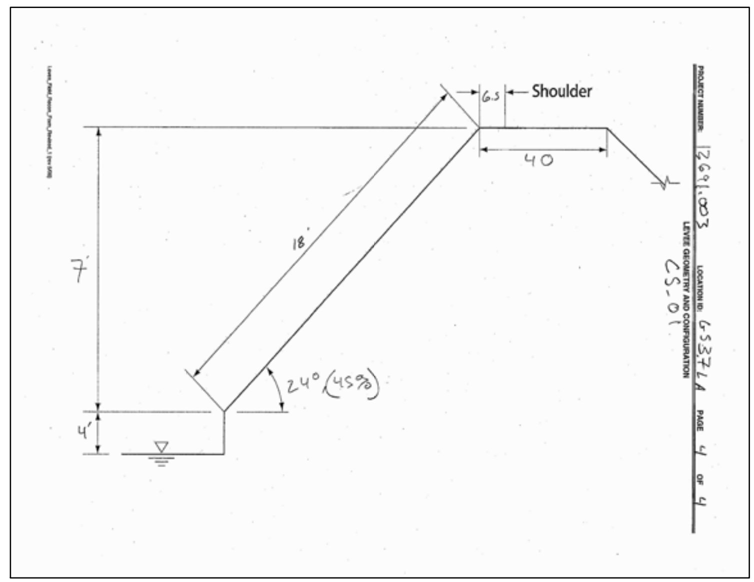
Comp Study Levee Crown Elevation	14.1
Comp Study Landside Toe Elevation	3.6
Comp Study Difference	10.6

Alternatives Report Analysis Levee Crown Elevation	15.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	4.9

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



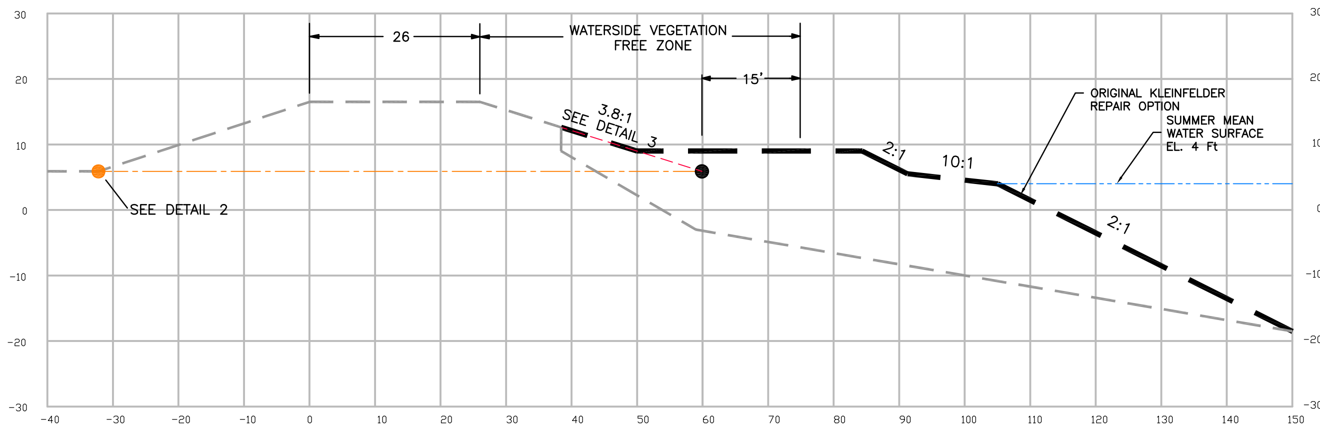
Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR Engineering, Inc.
Drawn By: E. Smith	GEORGIANA SLOUGH ~3.7aL
Checked By: R. Little	Date 3/31/11
QA/QC Review: D. Teak	Figure GS 3.7aL

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

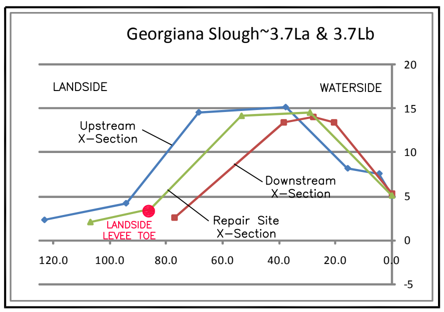
GEORGIANA SLOUGH 3.7bL



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.1
29.1	14.5
53.4	14.1
85.6	3.6
107.1	2.1

KEY DATA FOR DETERMINING LANDSIDE TOE

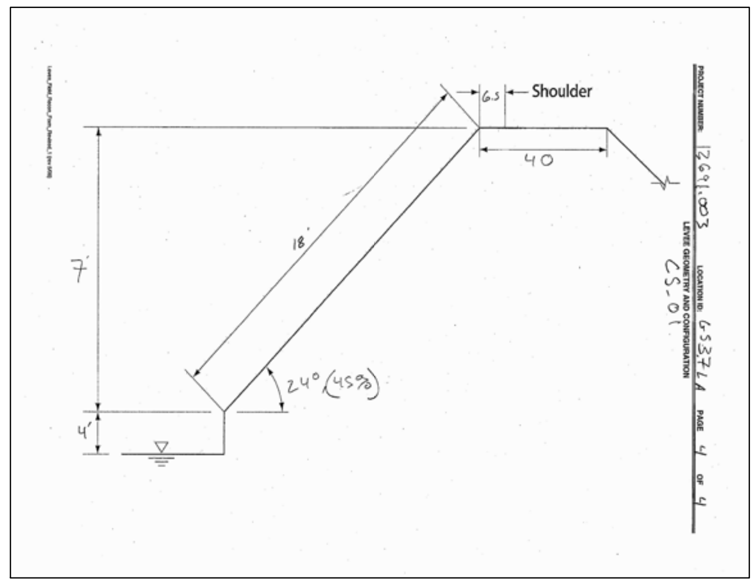
Comp Study Levee Crown Elevation	14.1
Comp Study Landside Toe Elevation	3.6
Comp Study Difference	10.6

Alternatives Report Analysis Levee Crown Elevation	15.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	4.9

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



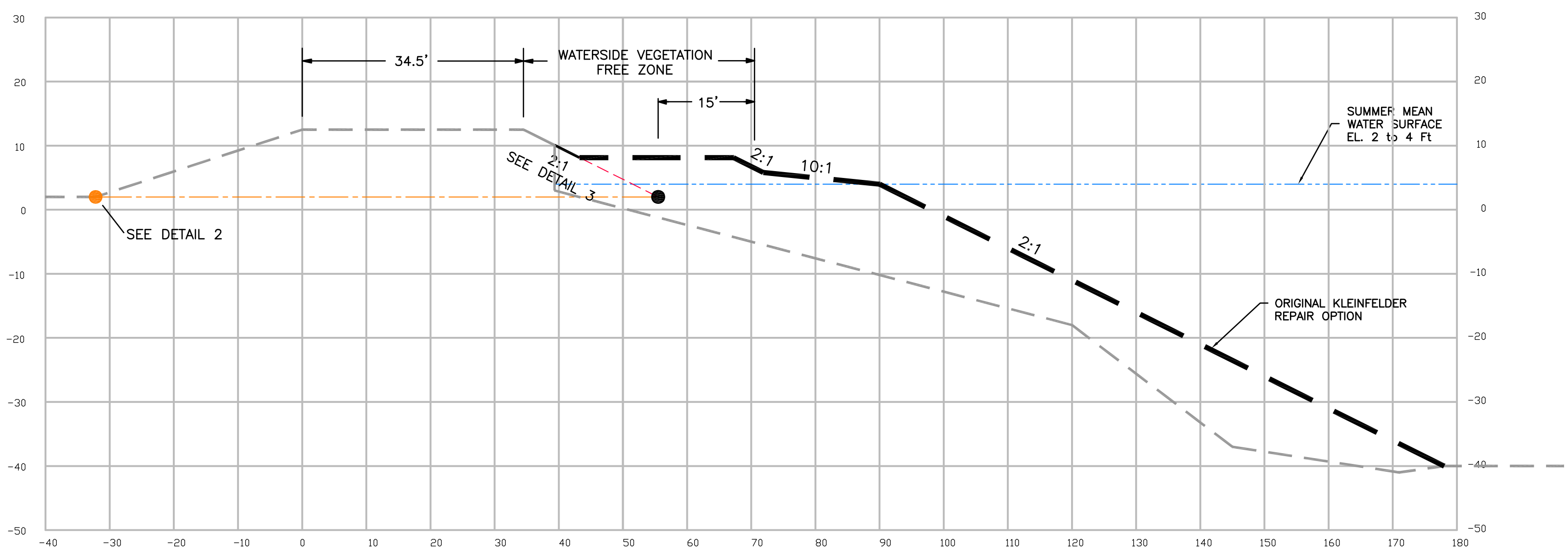
Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith		GEORGIANA SLOUGH ~3.7bL	Date	3/31/11
Checked By: R. Little			Figure	GS 3.7bL
QA/QC Review: D. Teak				

DETAIL 1

GEORGIANA SLOUGH 4.0L



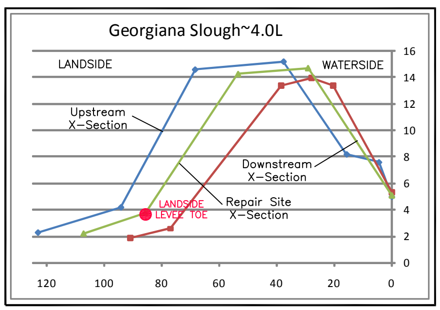
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.1
29.1	14.7
53.4	14.3
85.6	3.8
107.1	2.2

KEY DATA FOR DETERMINING LANDSIDE TOE

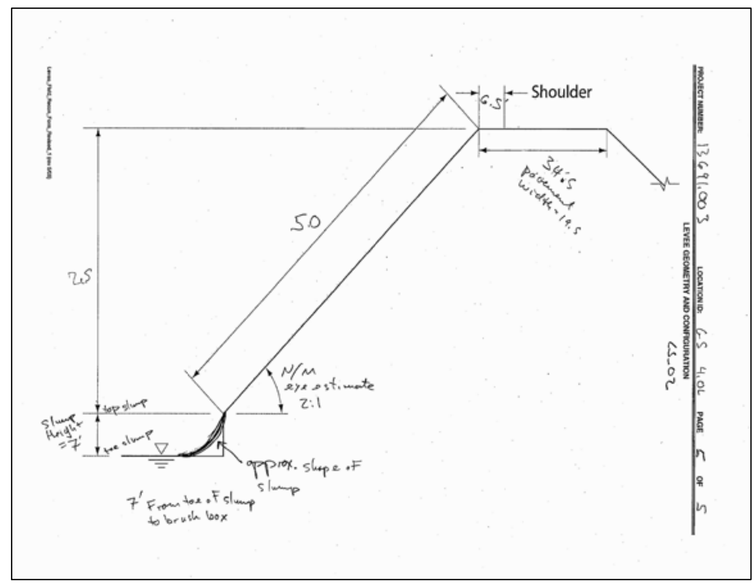
Comp Study Levee Crown Elevation	14.3
Comp Study Landside Toe Elevation	3.8
Comp Study Difference	10.5

Alternatives Report Analysis Levee Crown Elevation	12.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	2.0

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



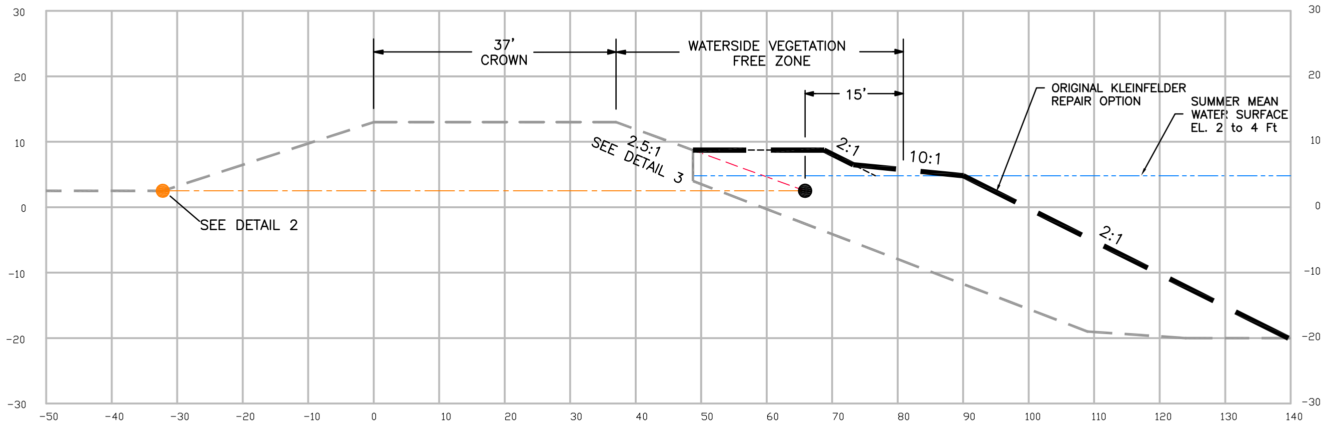
Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR HDR Engineering, Inc.	Drawn By: E. Smith Checked By: R. Little QA/QC Review: D. Teak	GEORGIANA SLOUGH ~ 4.0L
			Date 3/31/11
			Figure GS 4.0L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

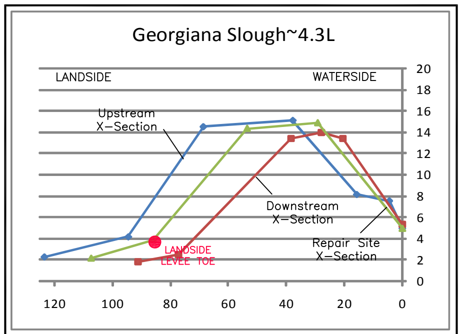
GEORGIANA SLOUGH 4.3L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.0
29.1	15.0
53.4	14.4
85.6	4.0
107.1	2.2

KEY DATA FOR DETERMINING LANDSIDE TOE

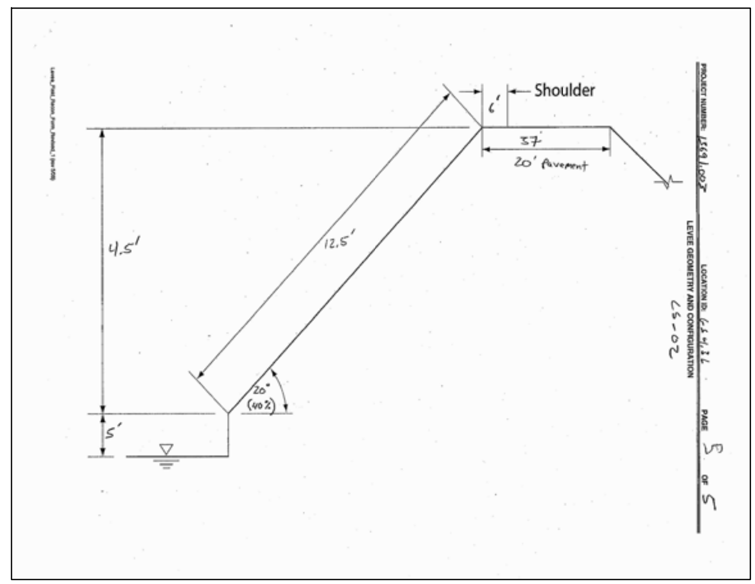
Comp Study Levee Crown Elevation	14.4
Comp Study Landside Toe Elevation	4.0
Comp Study Difference	10.5

Alternatives Report Analysis Levee Crown Elevation	12.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	2.0

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR HDR Engineering, Inc.	Drawn By: E. Smith Checked By: R. Little QA/QC Review: D. Teak	GEORGIANA SLOUGH ~ 4.3L

Date 3/31/11

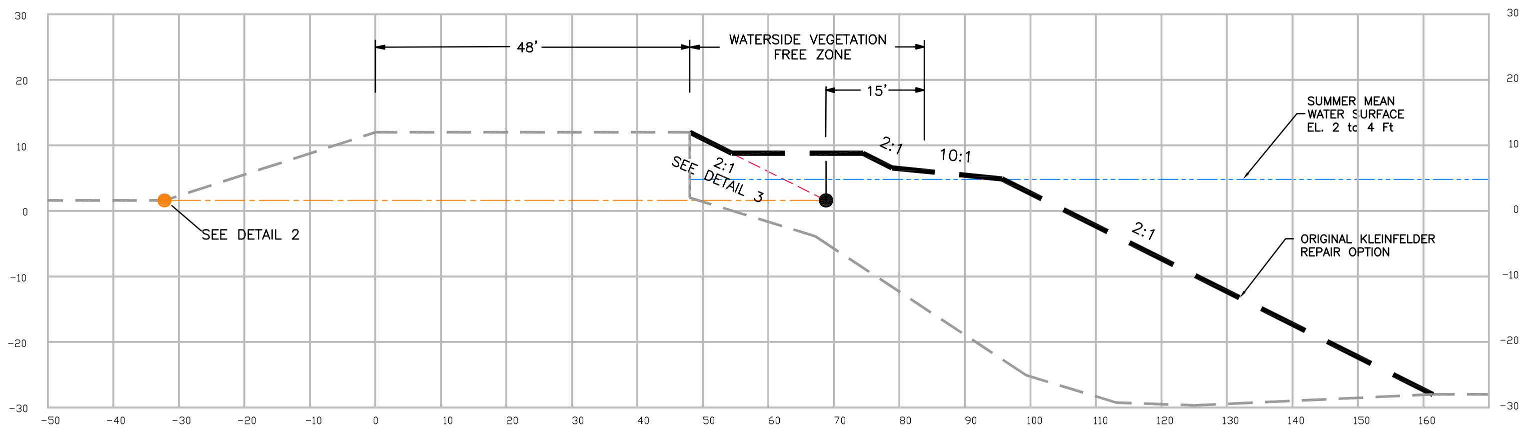
Figure GS 4.3L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

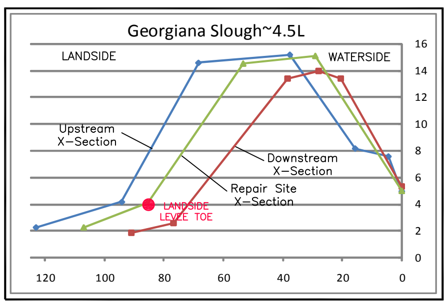
GEORGIANA SLOUGH 4.5L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.0
29.1	15.1
53.4	14.5
85.6	4.1
107.1	2.3

KEY DATA FOR DETERMINING LANDSIDE TOE

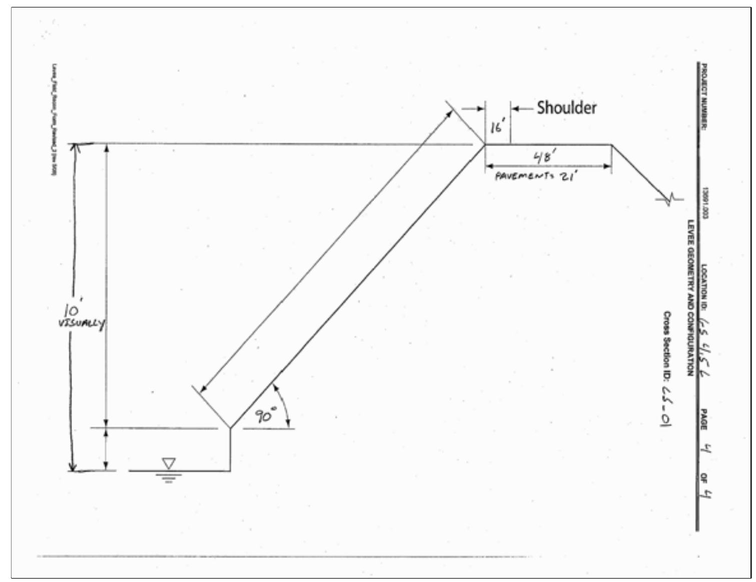
Comp Study Levee Crown Elevation	14.5
Comp Study Landside Toe Elevation	4.1
Comp Study Difference	10.4

Alternatives Report Analysis Levee Crown Elevation	12.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	1.6

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

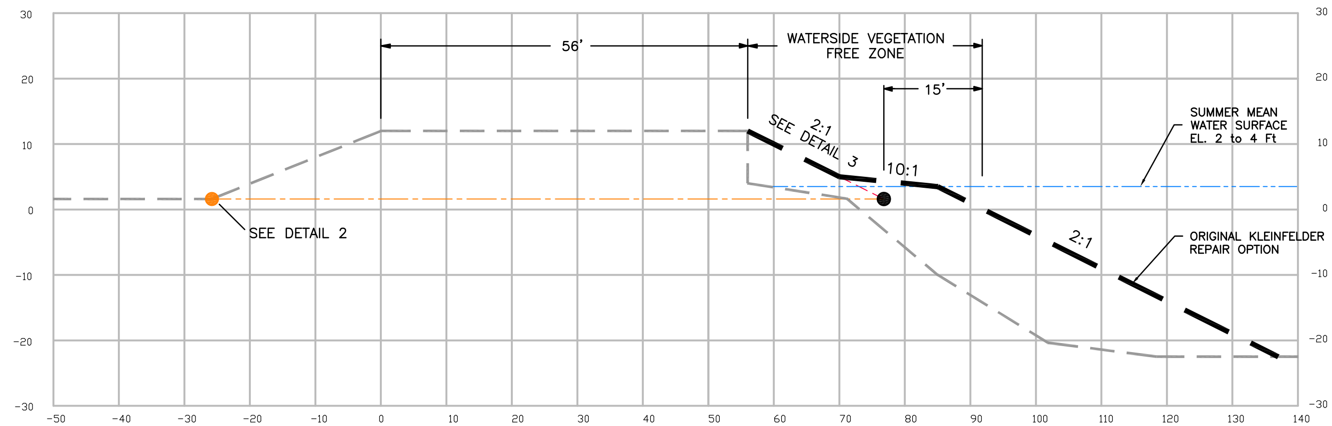
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

GEORGIANA SLOUGH ~ 4.5L

Date	3/31/11
Figure	GS 4.5L

DETAIL 1

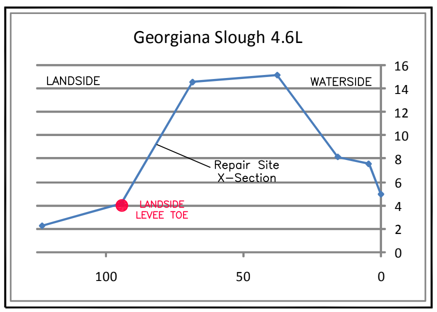
LEGEND
EXISTING LEVEE SURFACE
ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
PROJECTED LANDSIDE LEVEE TOE
WATER SURFACE
PROJECTED WATERSIDE SLOPE



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.0
4.5	7.6
15.7	8.2
37.7	15.2
68.5	14.6
94.3	4.2
123.2	2.3

KEY DATA FOR DETERMINING
LANDSIDE TOE

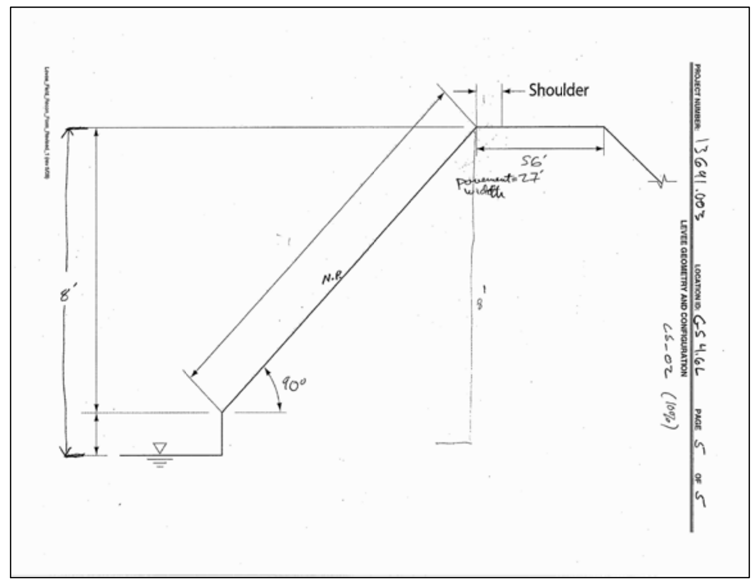
Comp Study Levee Crown Elevation	14.6
Comp Study Landside Toe Elevation	4.2
Comp Study Difference	10.4

Alternatives Report Analysis Levee Crown Elevation	12.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	1.6

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

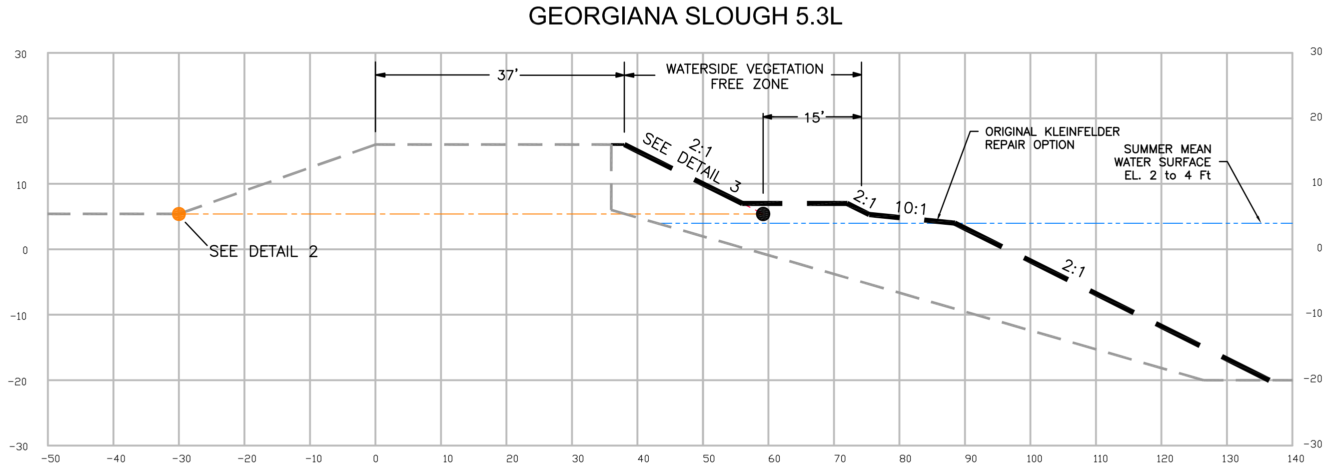
GEORGIANA SLOUGH ~ 4.6L

Date	3/31/11
Figure	GS 4.6L

DETAIL 1

LEGEND

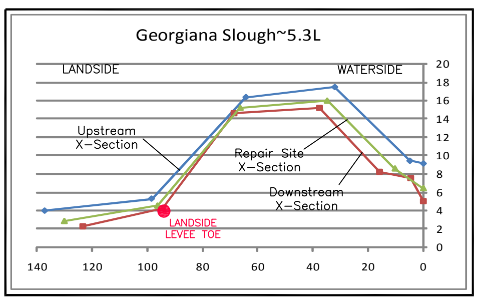
- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	6.5
10.3	8.7
34.9	16.0
66.4	15.3
96.4	4.6
130.2	3.0

KEY DATA FOR DETERMINING LANDSIDE TOE

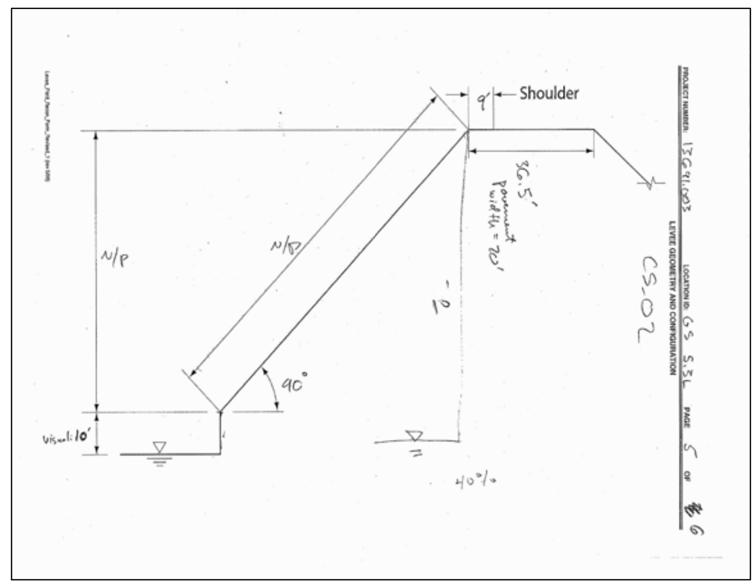
Comp Study Levee Crown Elevation	15.3
Comp Study Landside Toe Elevation	4.6
Comp Study Difference	10.6

Alternatives Report Analysis Levee Crown Elevation	16.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	5.4

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

GEORGIANA SLOUGH ~ 5.3L

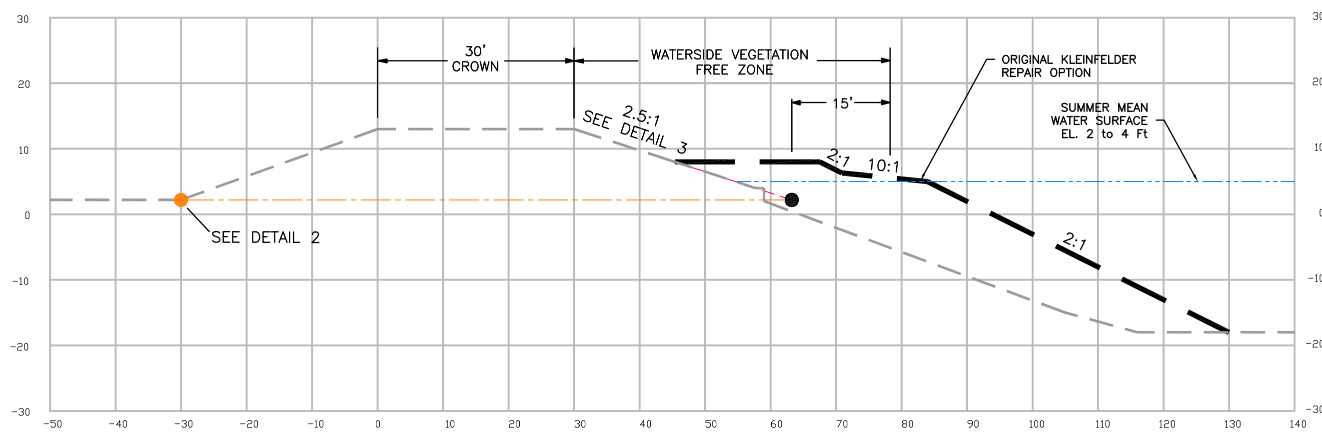
Date	3/31/11
Figure	GS 5.3L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

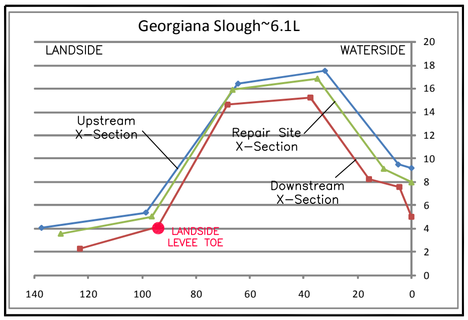
GEORGIANA SLOUGH 6.1L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	7.9
10.3	9.1
34.9	16.8
66.4	15.9
96.4	5.0
130.2	3.6

KEY DATA FOR DETERMINING LANDSIDE TOE

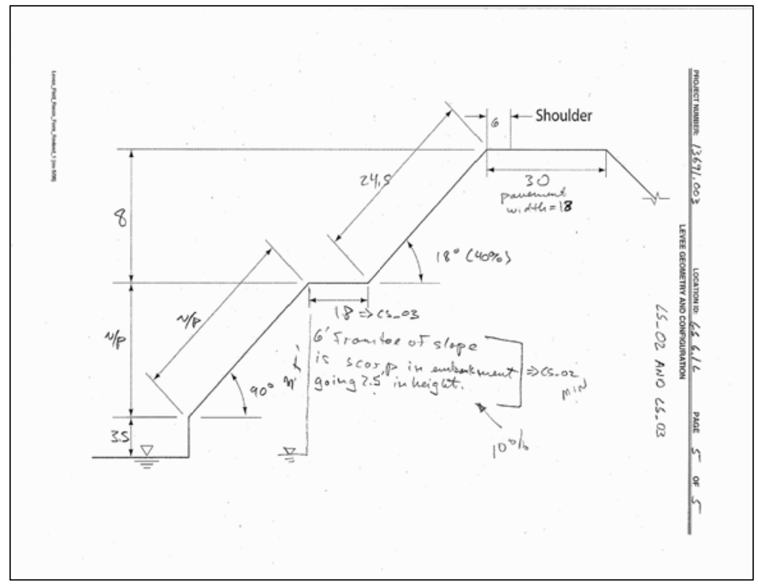
Comp Study Levee Crown Elevation	15.9
Comp Study Landside Toe Elevation	5.0
Comp Study Difference	10.8

Alternatives Report Analysis Levee Crown Elevation	13.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	2.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

GEORGIANA SLOUGH ~ 6.1L

Date 3/31/11

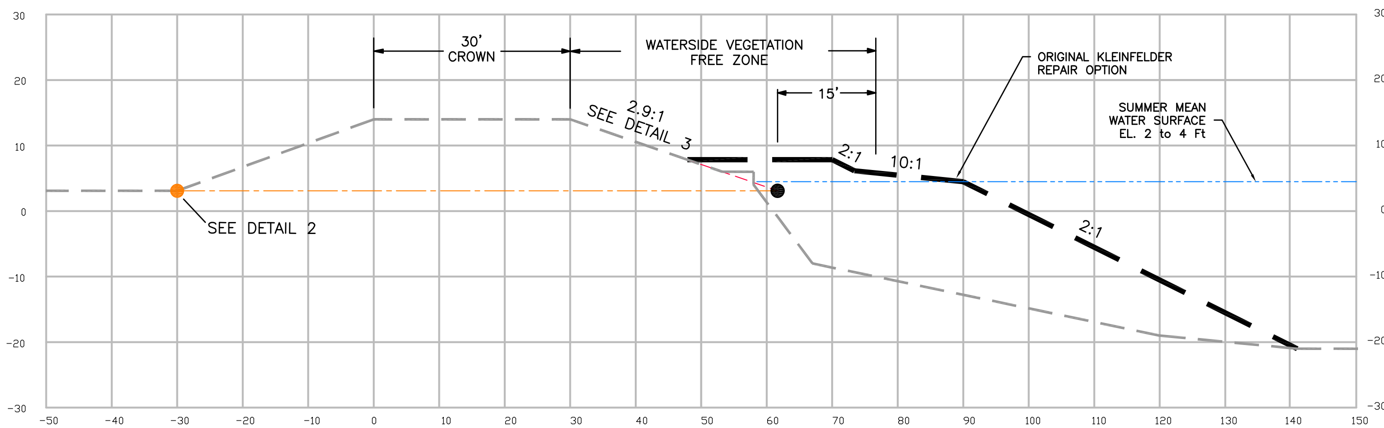
Figure GS 6.1L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

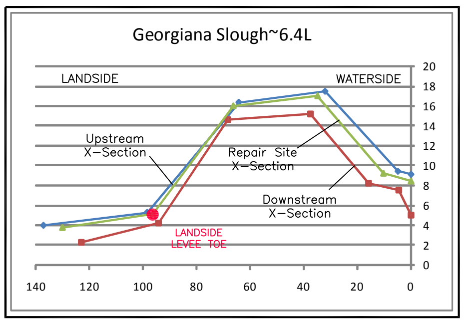
GEORGIANA SLOUGH 6.4L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	8.5
10.3	9.3
34.9	17.1
66.4	16.1
96.4	5.2
130.2	3.8

KEY DATA FOR DETERMINING LANDSIDE TOE

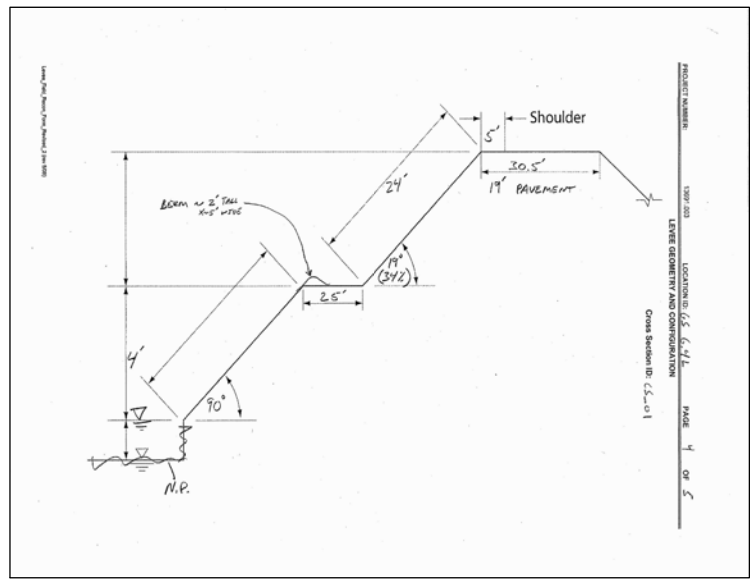
Comp Study Levee Crown Elevation	16.1
Comp Study Landside Toe Elevation	5.2
Comp Study Difference	10.9

Alternatives Report Analysis Levee Crown Elevation	14.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	3.1

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

GEORGIANA SLOUGH ~ 6.4L

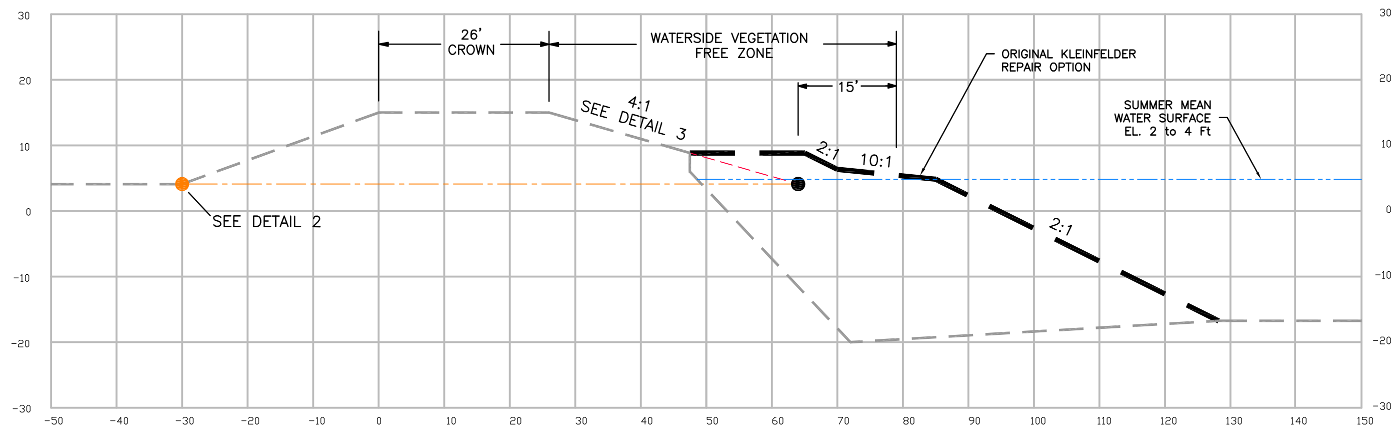
Date	3/31/11
Figure	GS 6.4L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

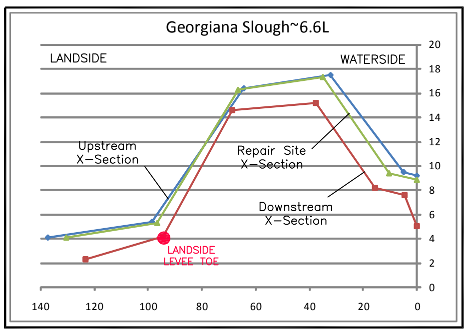
GEORGIANA SLOUGH 6.6L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	8.8
10.3	9.4
34.9	17.3
66.4	16.2
96.4	5.3
130.2	4.1

KEY DATA FOR DETERMINING LANDSIDE TOE

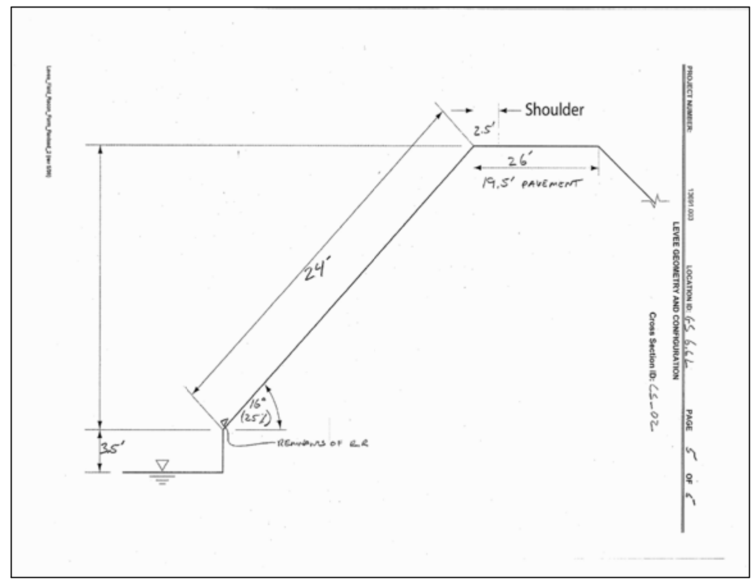
Comp Study Levee Crown Elevation	16.2
Comp Study Landside Toe Elevation	5.3
Comp Study Difference	10.9

Alternatives Report Analysis Levee Crown Elevation	15.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	4.1

DETAIL 3

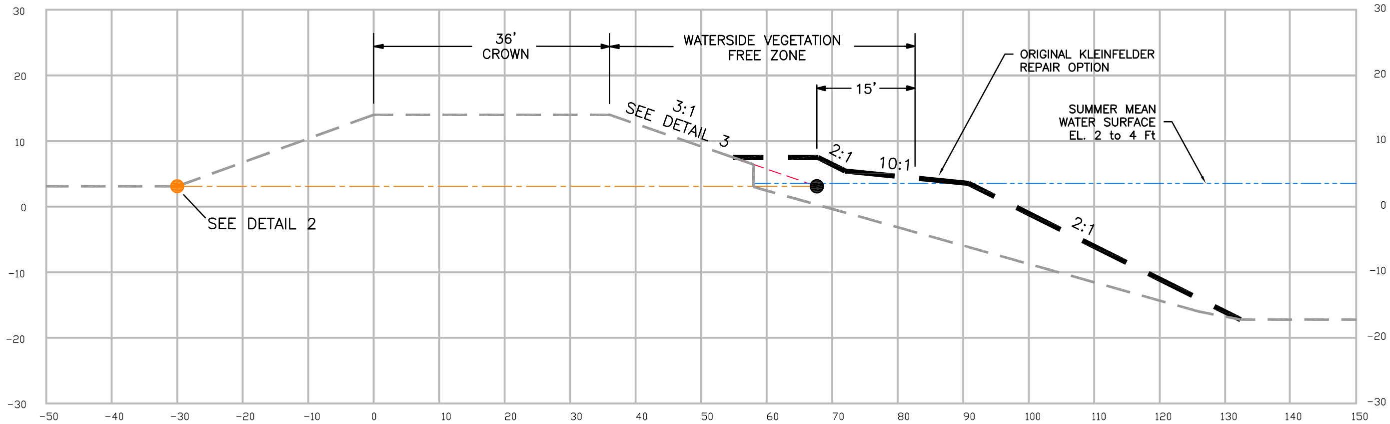
ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR HDR Engineering, Inc.	Drawn By: E. Smith Checked By: R. Little QA/QC Review: D. Teak	GEORGIANA SLOUGH 6.6L
		Date: 3/31/11	Figure: GS 6.6L

DETAIL 1

GEORGIANA SLOUGH 6.8L



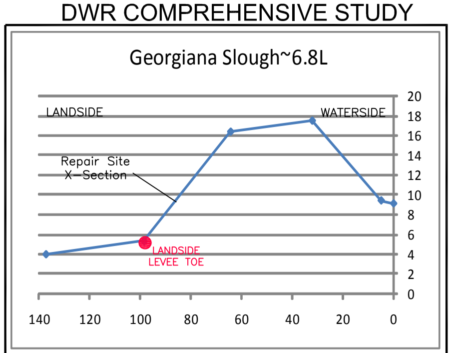
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	9.2
4.9	9.5
32.1	17.5
64.3	16.4
98.5	5.4
137.2	4.1

KEY DATA FOR DETERMINING LANDSIDE TOE

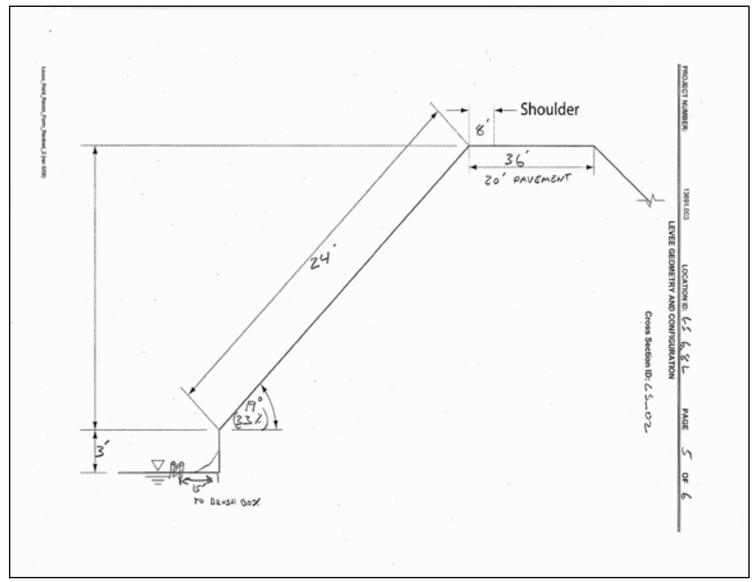
Comp Study Levee Crown Elevation	16.4
Comp Study Landside Toe Elevation	5.4
Comp Study Difference	11.0

Alternatives Report Analysis Levee Crown Elevation	14.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	3.5

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



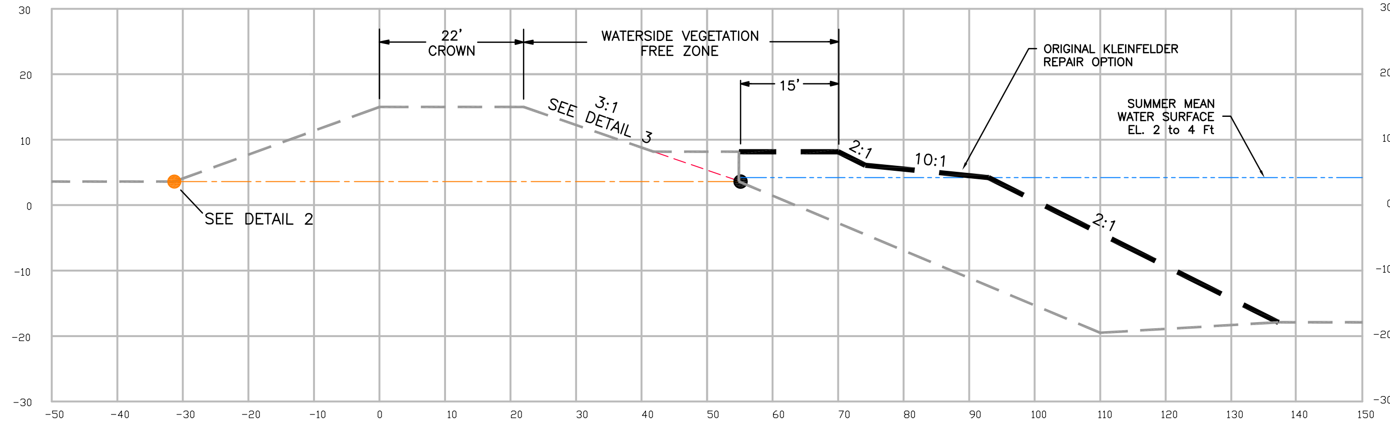
Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR HDR Engineering, Inc.	Drawn By: E. Smith Checked By: R. Little QA/QC Review: D. Teak
GEORGIANA SLOUGH ~ 6.8L	Date 3/31/11	Figure GS 6.8L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

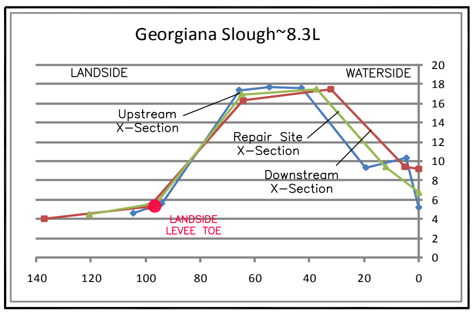
GEORGIANA SLOUGH 8.3L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	6.8
12.1	9.4
37.5	17.6
65.0	17.0
96.3	5.6
120.8	4.5

KEY DATA FOR DETERMINING
LANDSIDE TOE

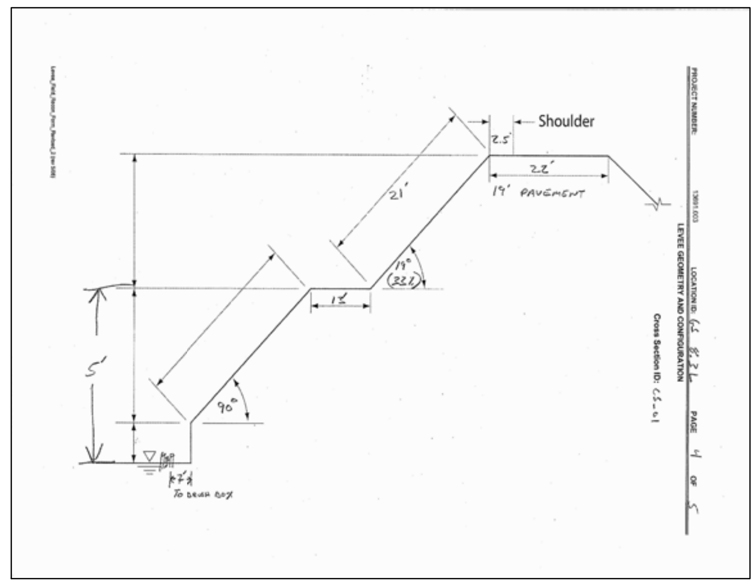
Comp Study Levee Crown Elevation	17.0
Comp Study Landside Toe Elevation	5.6
Comp Study Difference	11.4

Alternatives Report Analysis Levee Crown Elevation	15.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	3.6

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

GEORGIANA SLOUGH ~ 8.3L

Date 3/31/11

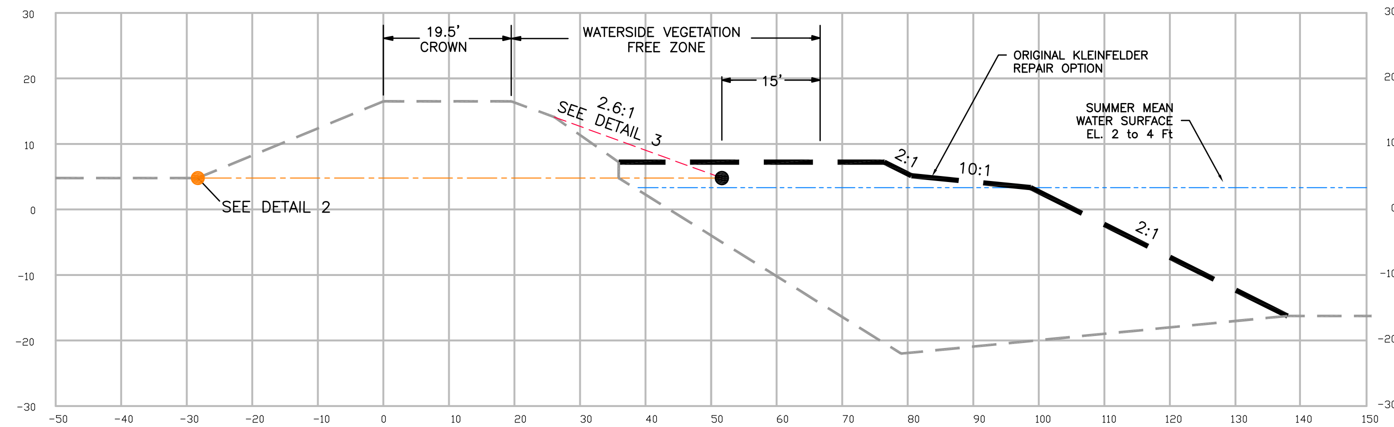
Figure GS 8.3L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

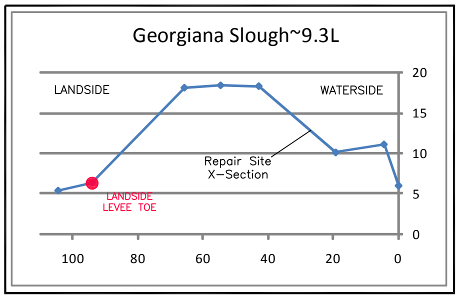
GEORGIANA SLOUGH 9.3L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	6.1
4.5	11.2
19.3	10.2
42.9	18.4
54.6	18.5
65.7	18.2
94.0	6.5
104.4	5.5

KEY DATA FOR DETERMINING LANDSIDE TOE

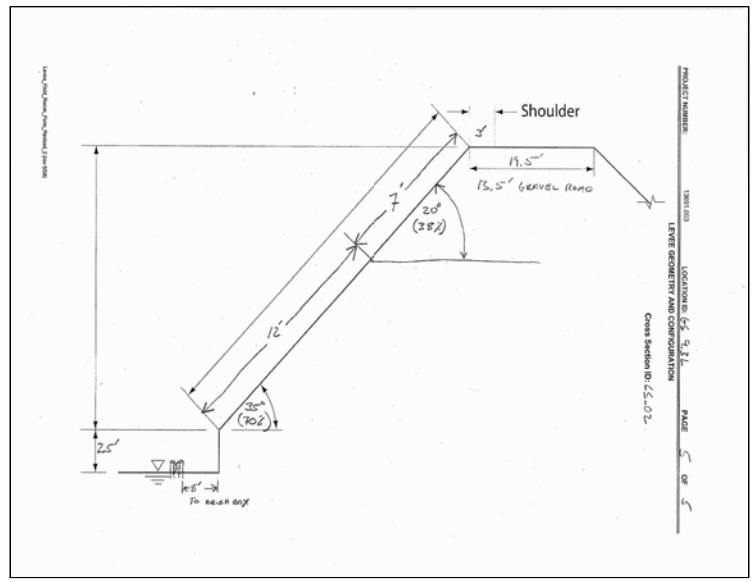
Comp Study Levee Crown Elevation	18.2
Comp Study Landside Toe Elevation	6.5
Comp Study Difference	11.7

Alternatives Report Analysis Levee Crown Elevation	16.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	4.8

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

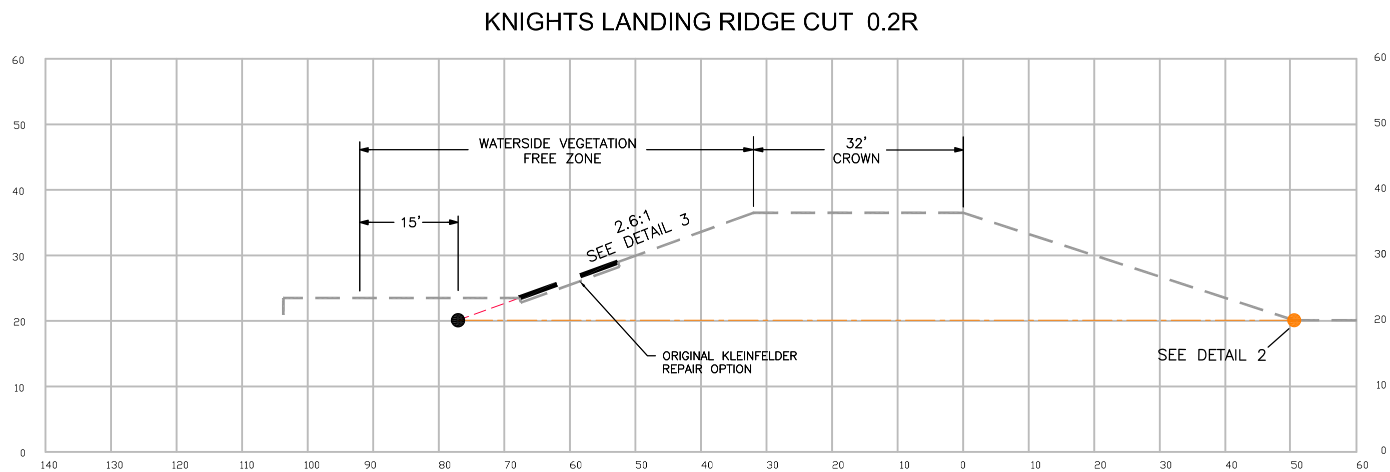
GEORGIANA SLOUGH ~ 9.3L

Date	3/31/11
Figure	GS 9.3L

DETAIL 1

LEGEND

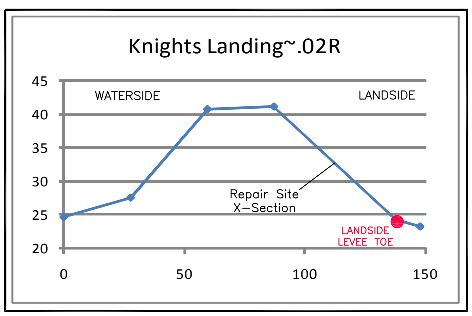
- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	24.8
27.8	27.7
59.7	40.9
87.3	41.3
137.8	24.5
148.0	23.4

KEY DATA FOR DETERMINING LANDSIDE TOE

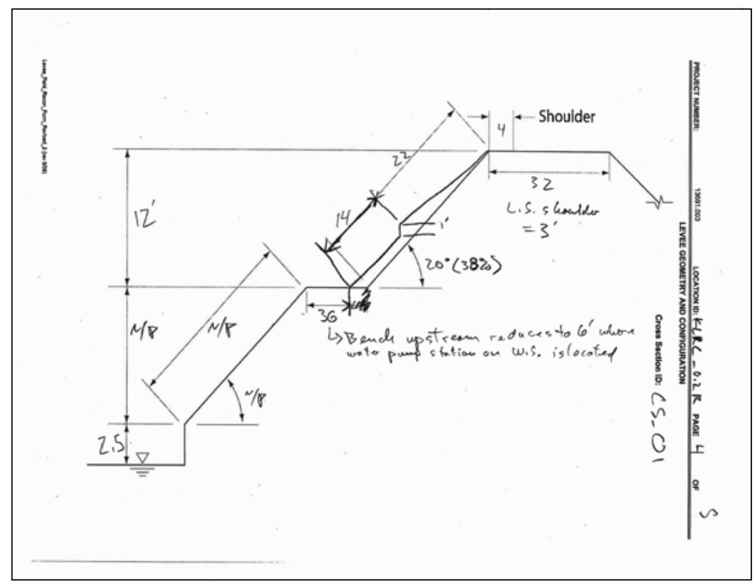
Comp Study Levee Crown Elevation	40.9
Comp Study Landside Toe Elevation	24.5
Comp Study Difference	16.4

Alternatives Report Analysis Levee Crown Elevation	36.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	20.1

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

KNIGHTS LANDING RIDGE CUT ~ 0.2R

Date 3/31/11

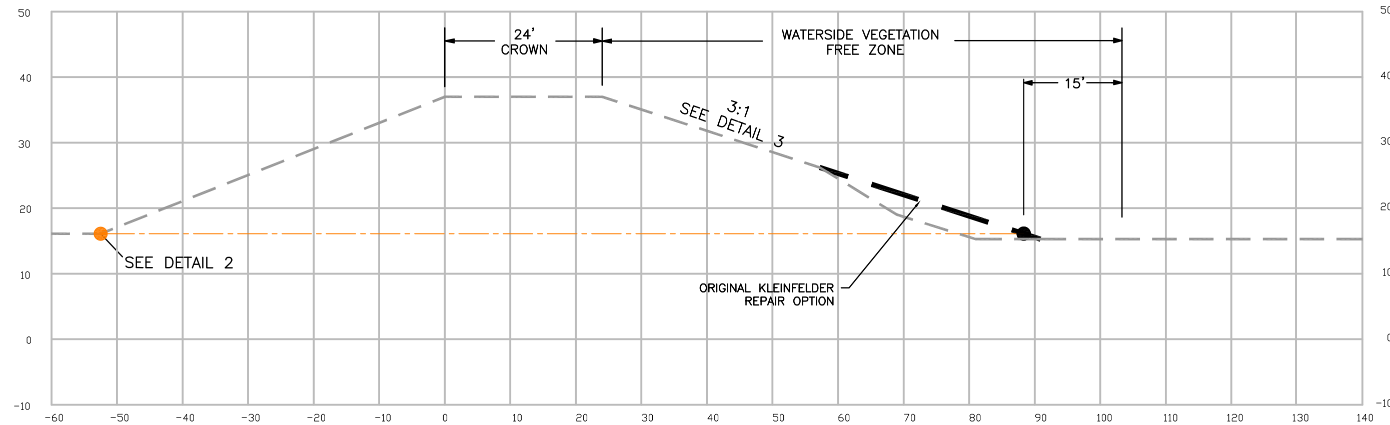
Figure KLRC 0.2R

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

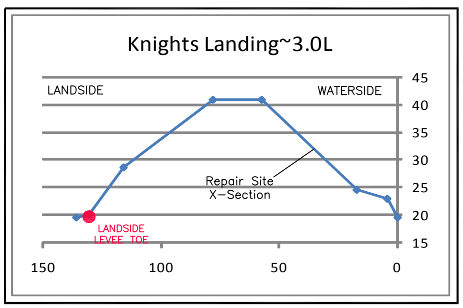
KNIGHTS LANDING RIDGE CUT 3.0L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	19.6
4.4	23.0
17.3	24.6
57.5	41.0
78.1	41.0
115.9	28.7
130.6	20.1
135.9	19.6

KEY DATA FOR DETERMINING
LANDSIDE TOE

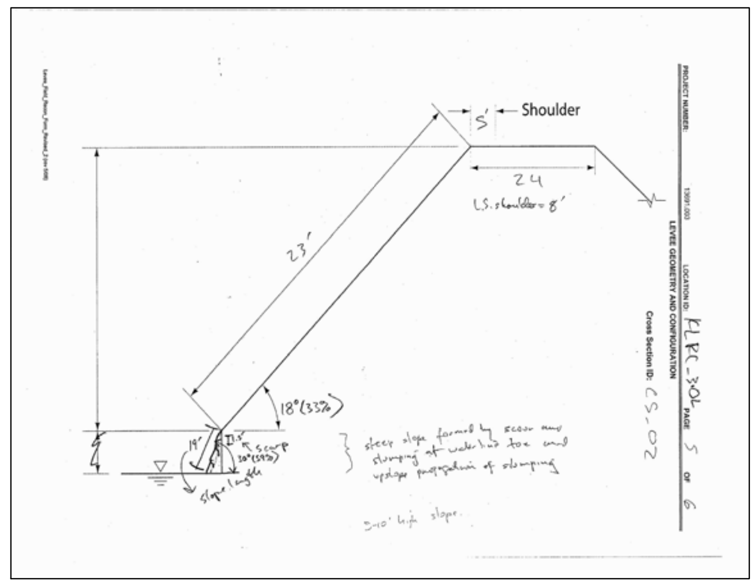
Comp Study Levee Crown Elevation	41.0
Comp Study Landside Toe Elevation	20.1
Comp Study Difference	20.9

Alternatives Report Analysis Levee Crown Elevation	37.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	16.1

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

KNIGHTS LANDING
RIDGE CUT ~ 3.0L

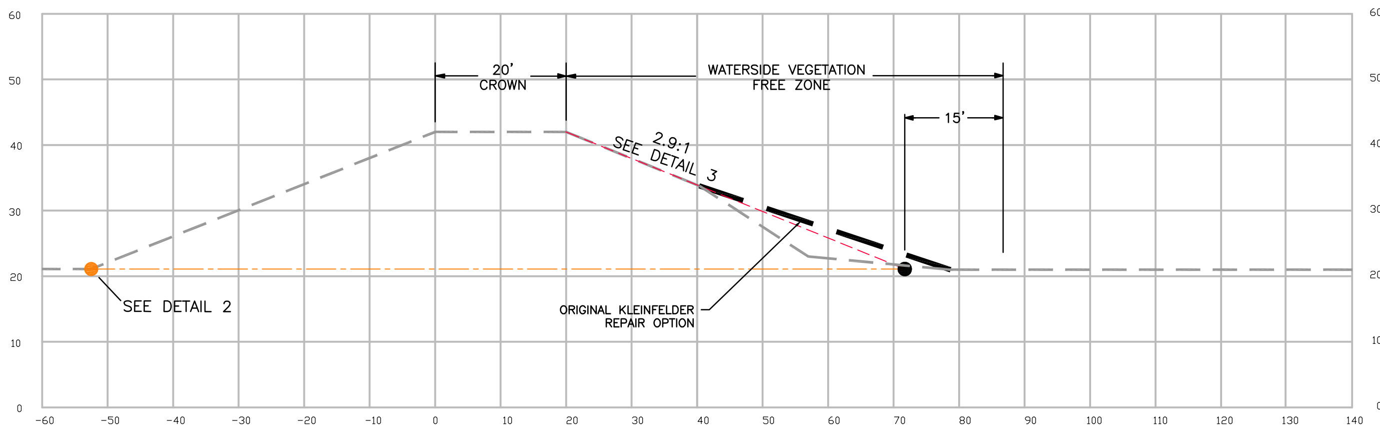
Date	3/31/11
Figure	KLRC 3.0L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

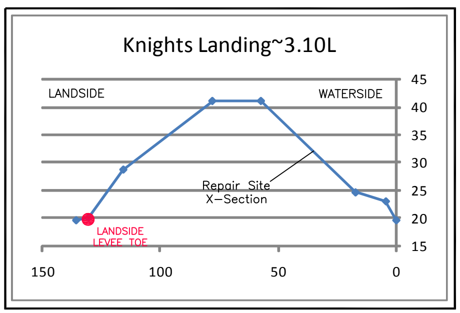
KNIGHTS LANDING RIDGE CUT 3.1L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	19.7
4.4	23.1
17.3	24.7
57.5	41.1
78.1	41.1
115.9	28.8
130.6	20.2
135.9	19.7

KEY DATA FOR DETERMINING LANDSIDE TOE

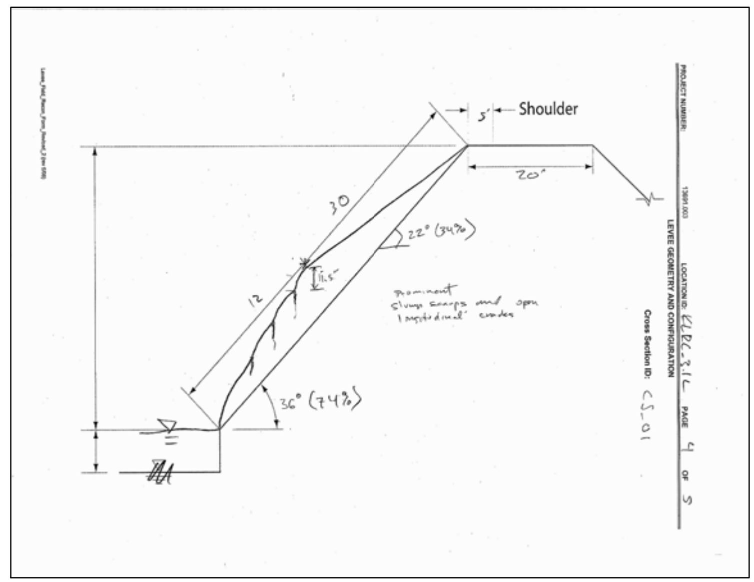
Comp Study Levee Crown Elevation	41.1
Comp Study Landside Toe Elevation	20.2
Comp Study Difference	20.9

Alternatives Report Analysis Levee Crown Elevation	42.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	21.1

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



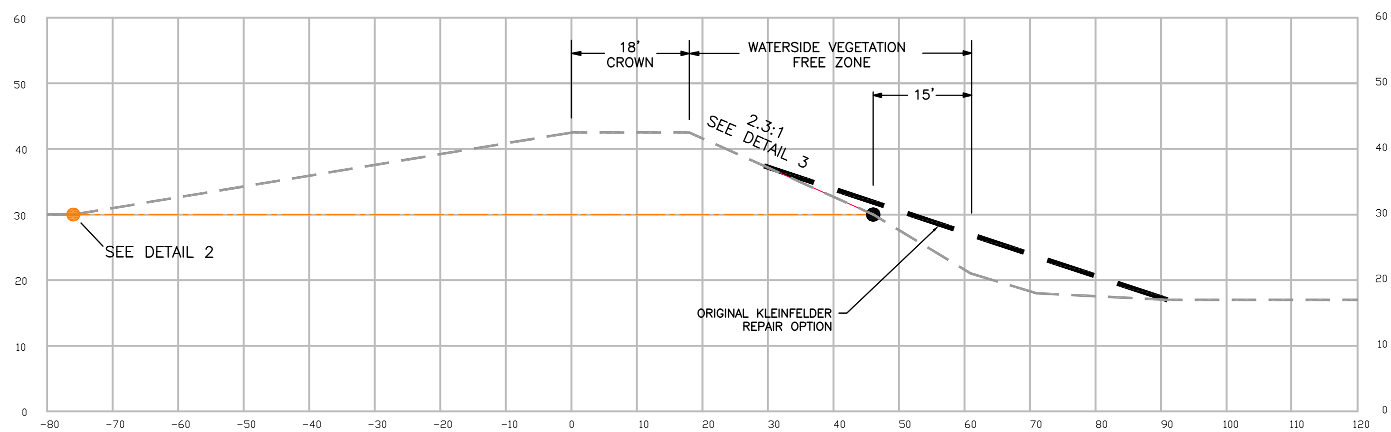
Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR HDR Engineering, Inc.	Drawn By: E. Smith Checked By: R. Little QA/QC Review: D. Teak	KNIGHTS LANDING RIDGE CUT ~ 3.1L
		Date: 3/31/11	Figure: KLRC 3.1L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

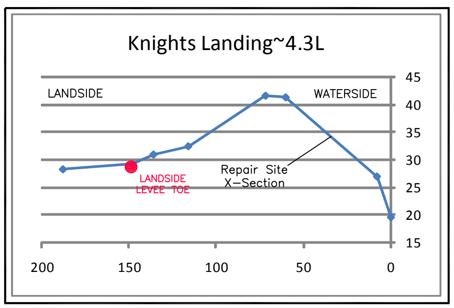
KNIGHTS LANDING RIDGE CUT 4.3L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	19.6
8.0	27.0
60.2	41.4
71.7	41.7
115.9	32.5
135.8	31.0
147.7	29.2
187.6	28.3

KEY DATA FOR DETERMINING
LANDSIDE TOE

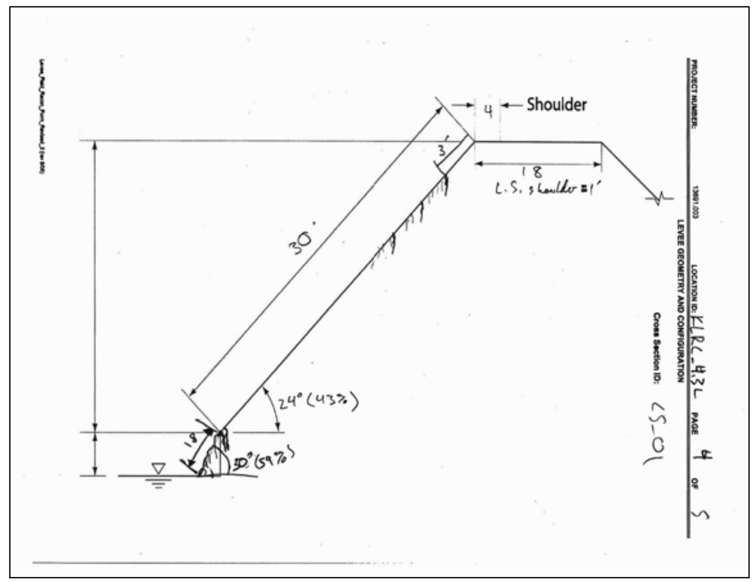
Comp Study Levee Crown Elevation	41.7
Comp Study Landside Toe Elevation	29.2
Comp Study Difference	12.5

Alternatives Report Analysis Levee Crown Elevation	42.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	30.0

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

KNIGHTS LANDING
RIDGE CUT ~ 4.3L

Date 3/31/11

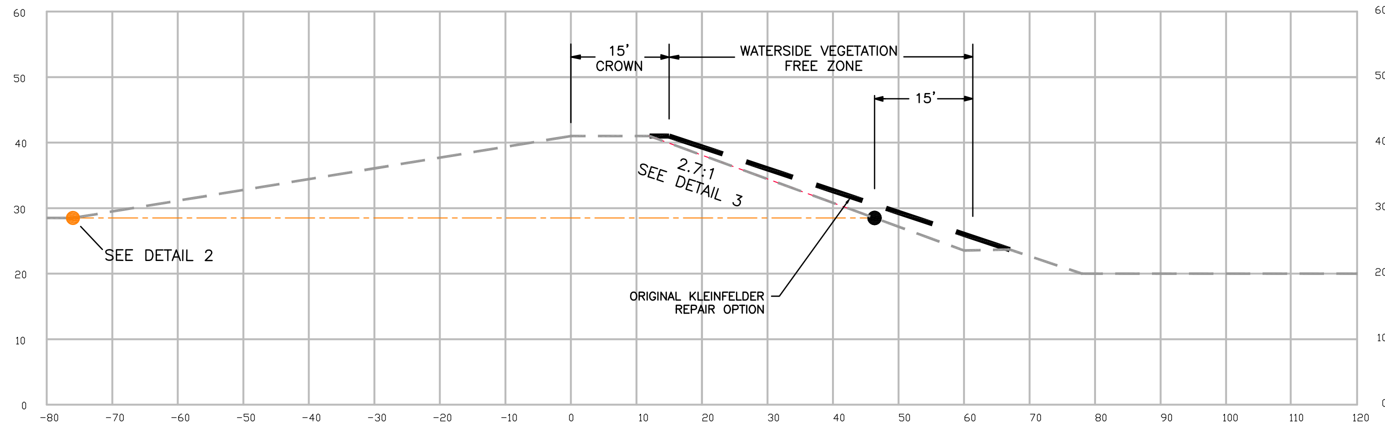
Figure KLRC 4.3L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

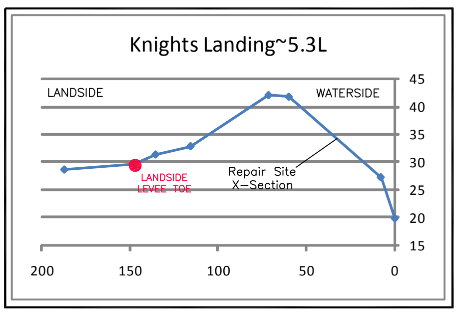
KNIGHTS LANDING RIDGE CUT 5.3L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	20.0
8.0	27.4
60.2	41.9
71.7	42.2
115.9	33.0
135.8	31.5
147.7	29.7
187.6	28.8

KEY DATA FOR DETERMINING LANDSIDE TOE

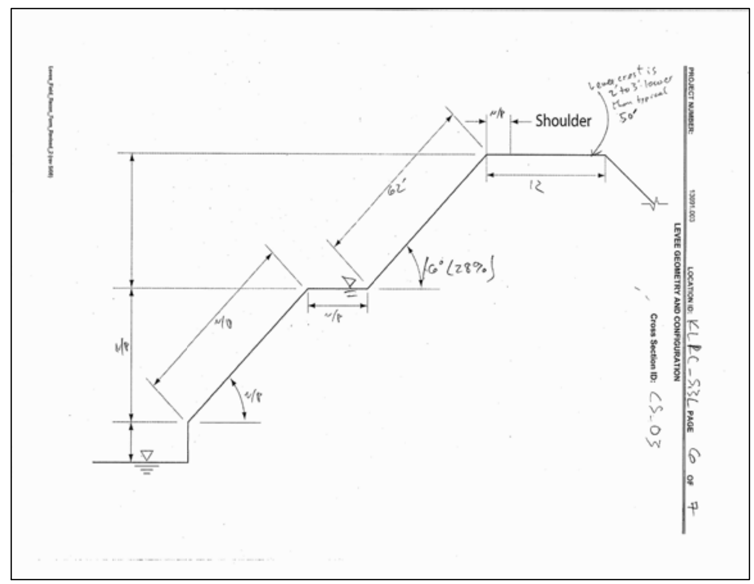
Comp Study Levee Crown Elevation	42.2
Comp Study Landside Toe Elevation	29.7
Comp Study Difference	12.5

Alternatives Report Analysis Levee Crown Elevation	41.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	28.5

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

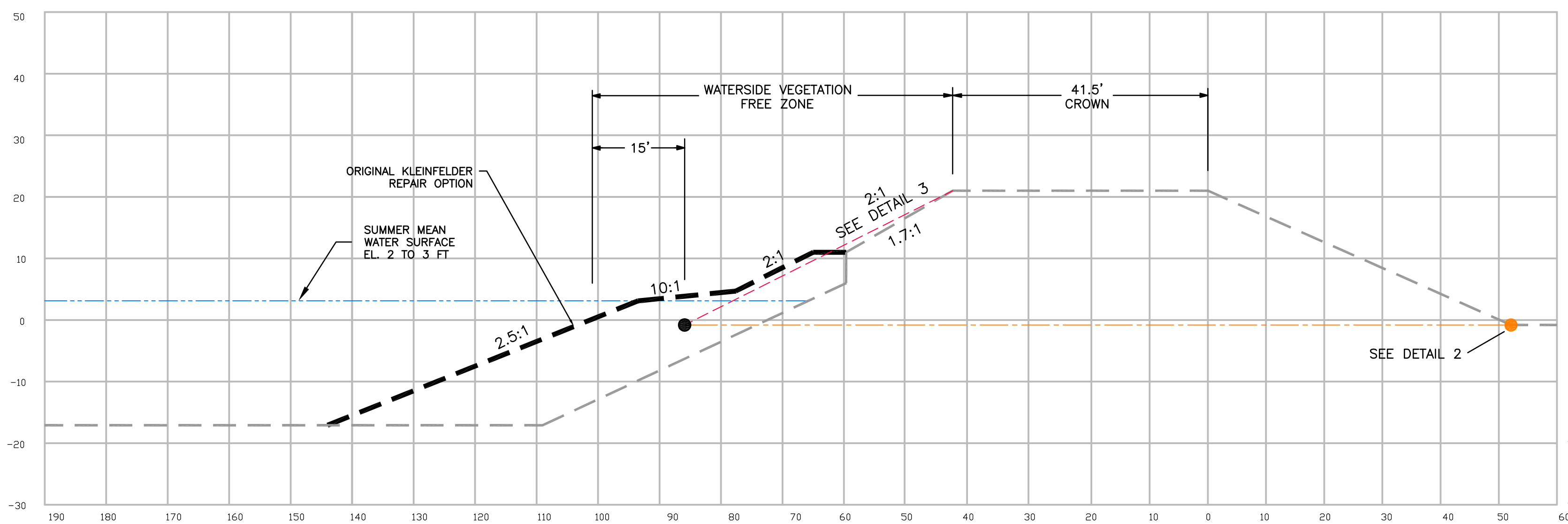
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

KNIGHTS LANDING RIDGE CUT ~ 5.3L

Date	3/31/11
Figure	KLRC 5.3L

DETAIL 1

STEAMBOAT SLOUGH 18.8R



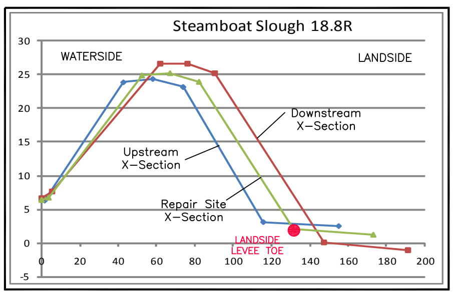
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	6.4
3.7	6.8
52.3	24.8
67.0	25.2
82.1	23.9
131.4	2.1
173.0	1.3

KEY DATA FOR DETERMINING
LANDSIDE TOE

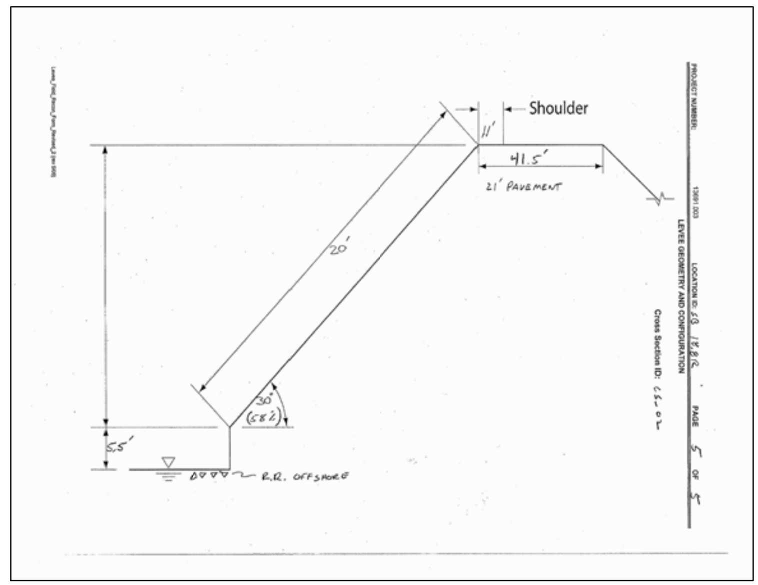
Comp Study Levee Crown Elevation	23.9
Comp Study Landside Toe Elevation	2.1
Comp Study Difference	21.8

Alternatives Report Analysis Levee Crown Elevation	21.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	-0.8

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

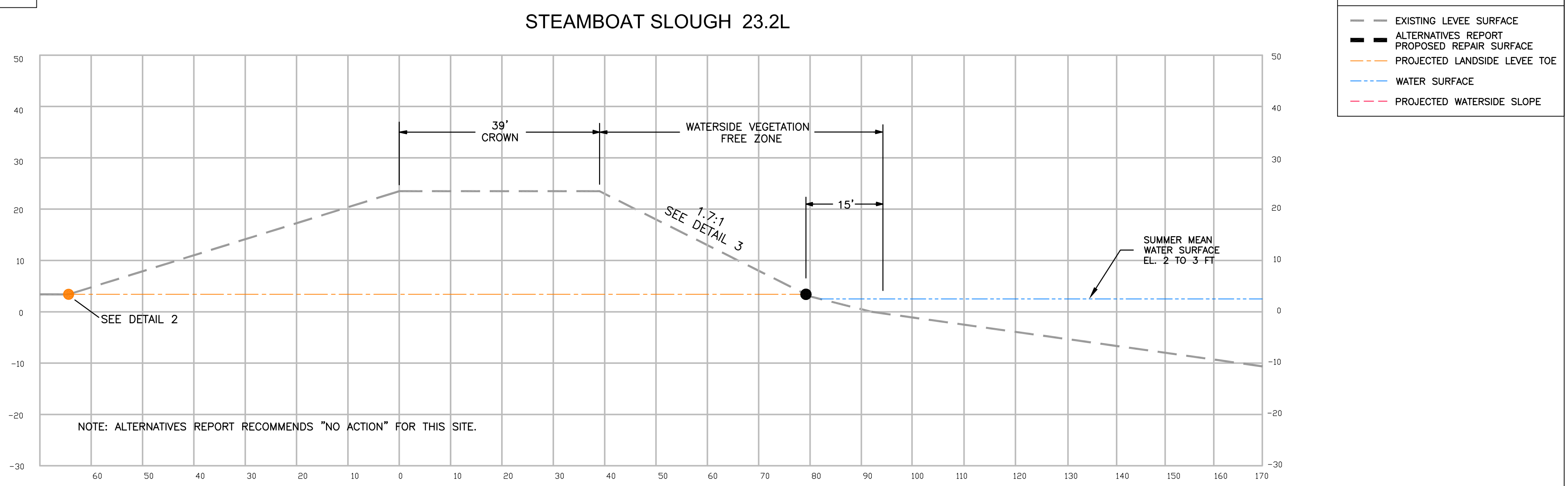
Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

STEAMBOAT SLOUGH ~ 18.8R

Date 3/31/11

Figure StS 18.8R

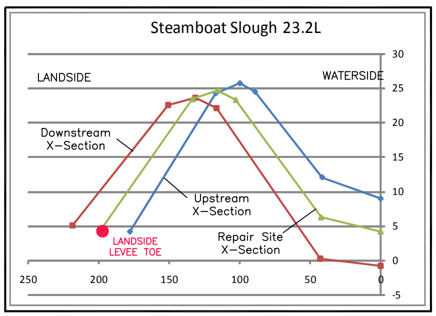
DETAIL 1



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	4.3
42.3	6.3
102.8	23.4
115.5	24.7
133.6	23.5
198.0	4.7

KEY DATA FOR DETERMINING LANDSIDE TOE

Comp Study Levee Crown Elevation	24.7
Comp Study Landside Toe Elevation	4.7
Comp Study Difference	20.1

Alternatives Report Analysis Levee Crown Elevation	23.5

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	3.4

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES

NO FIELD NOTES EXISTS

Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

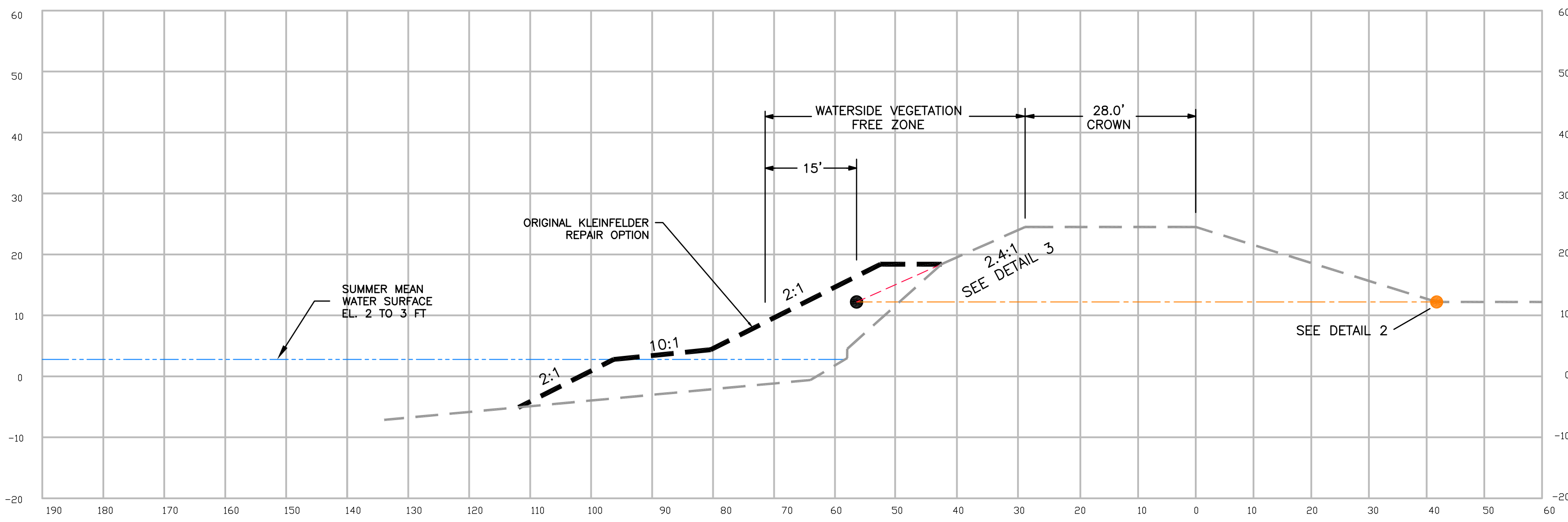
STEAMBOAT SLOUGH ~ 23.2R

Date 3/31/11

Figure StS 23.2R

DETAIL 1

STEAMBOAT SLOUGH 23.9R



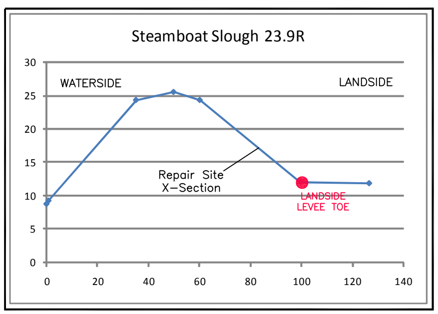
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	8.8
0.8	9.3
35.1	24.4
49.8	25.6
60.1	24.4
99.6	12.1
126.5	11.9

KEY DATA FOR DETERMINING
LANDSIDE TOE

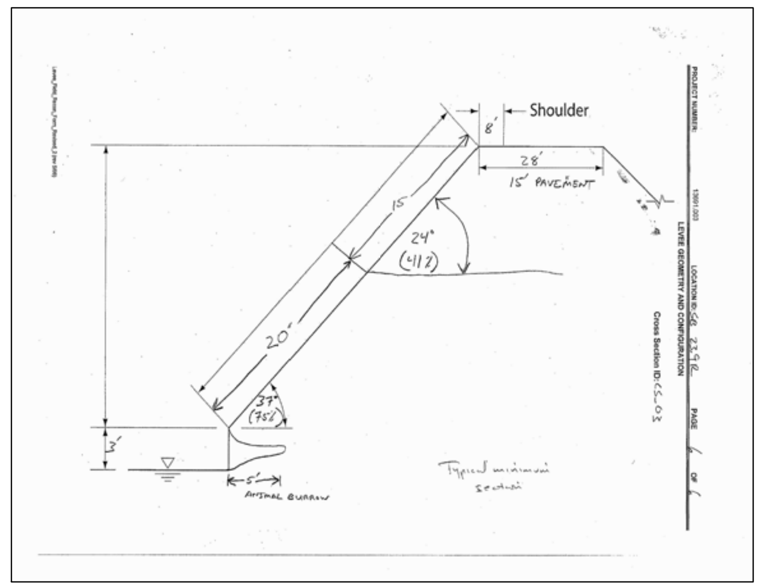
Comp Study Levee Crown Elevation	24.4
Comp Study Landside Toe Elevation	12.1
Comp Study Difference	12.3

Alternatives Report Analysis Levee Crown Elevation	24.5

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	12.2

DETAIL 3

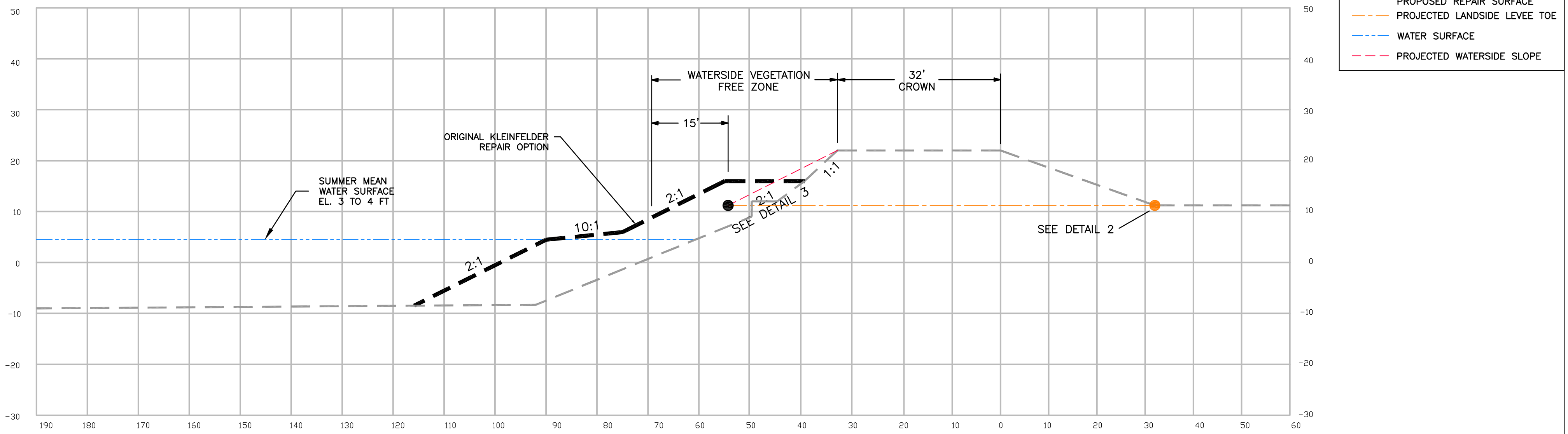
ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR Engineering, Inc.
Drawn By: E. Smith	Date 3/31/11
Checked By: R. Little	STEAMBOAT SLOUGH ~ 23.9R
QA/QC Review: D. Teak	Figure StS 23.9R

DETAIL 1

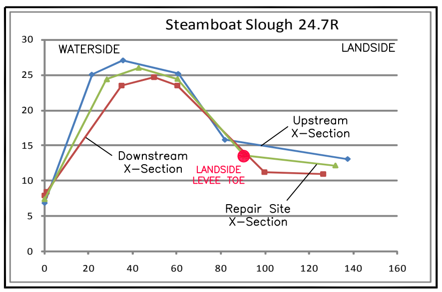
STEAMBOAT SLOUGH 24.7R



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	7.4
28.3	24.3
42.7	25.9
60.4	24.4
90.7	13.6
132.0	12.1

KEY DATA FOR DETERMINING LANDSIDE TOE

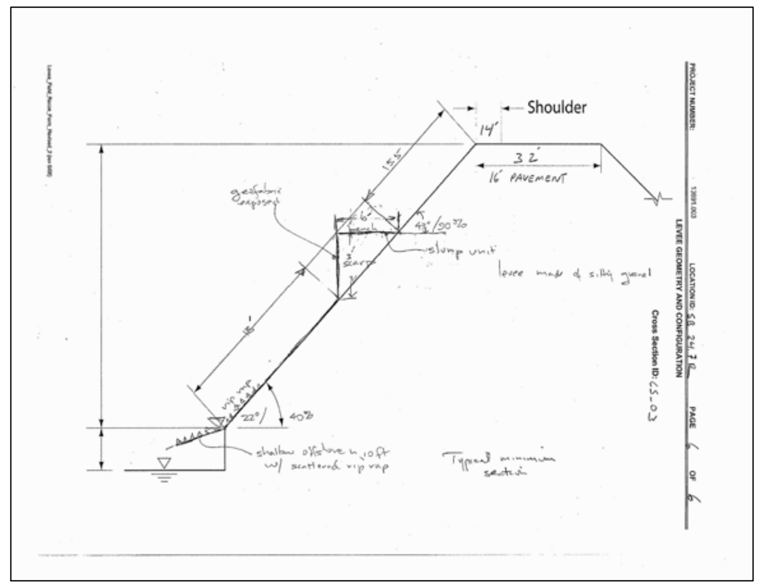
Comp Study Levee Crown Elevation	24.4
Comp Study Landside Toe Elevation	13.6
Comp Study Difference	10.8

Alternatives Report Analysis Levee Crown Elevation	22.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	11.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

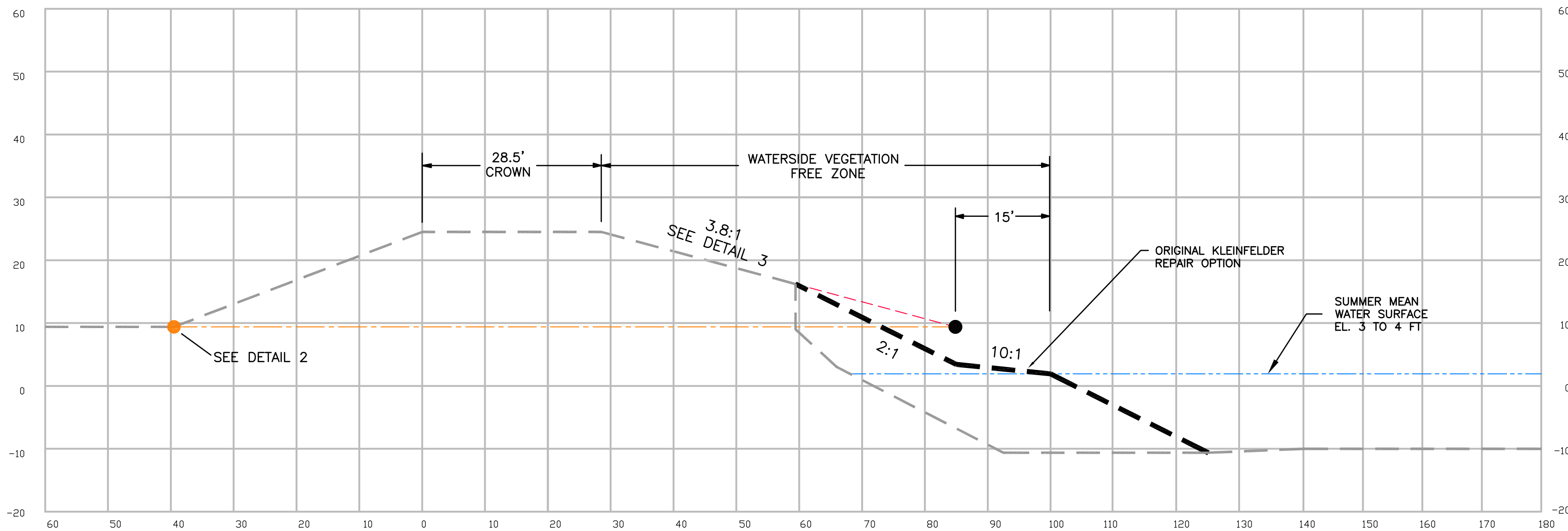
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

STEAMBOAT SLOUGH ~ 24.7R

Date	3/31/11
Figure	StS 24.7R

DETAIL 1

STEAMBOAT SLOUGH 25.0L



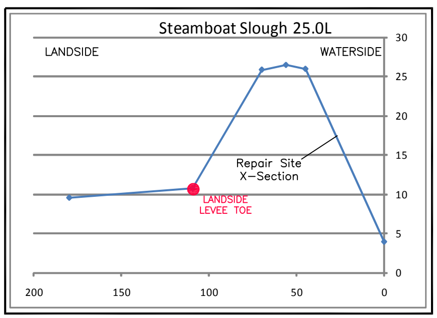
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	4.0
45.0	26.0
56.1	26.5
69.9	25.9
109.4	10.8
179.9	9.6

KEY DATA FOR DETERMINING LANDSIDE TOE

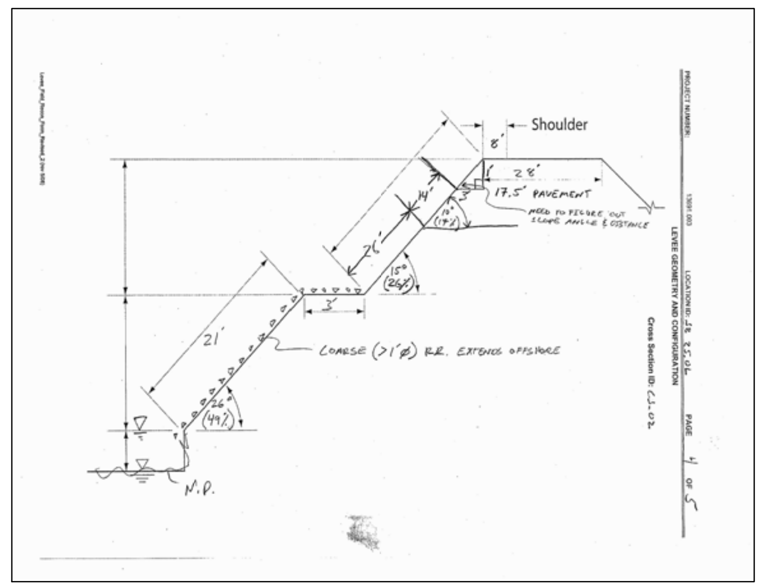
Comp Study Levee Crown Elevation	25.9
Comp Study Landside Toe Elevation	10.8
Comp Study Difference	15.1

Alternatives Report Analysis Levee Crown Elevation	24.5

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	9.4

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

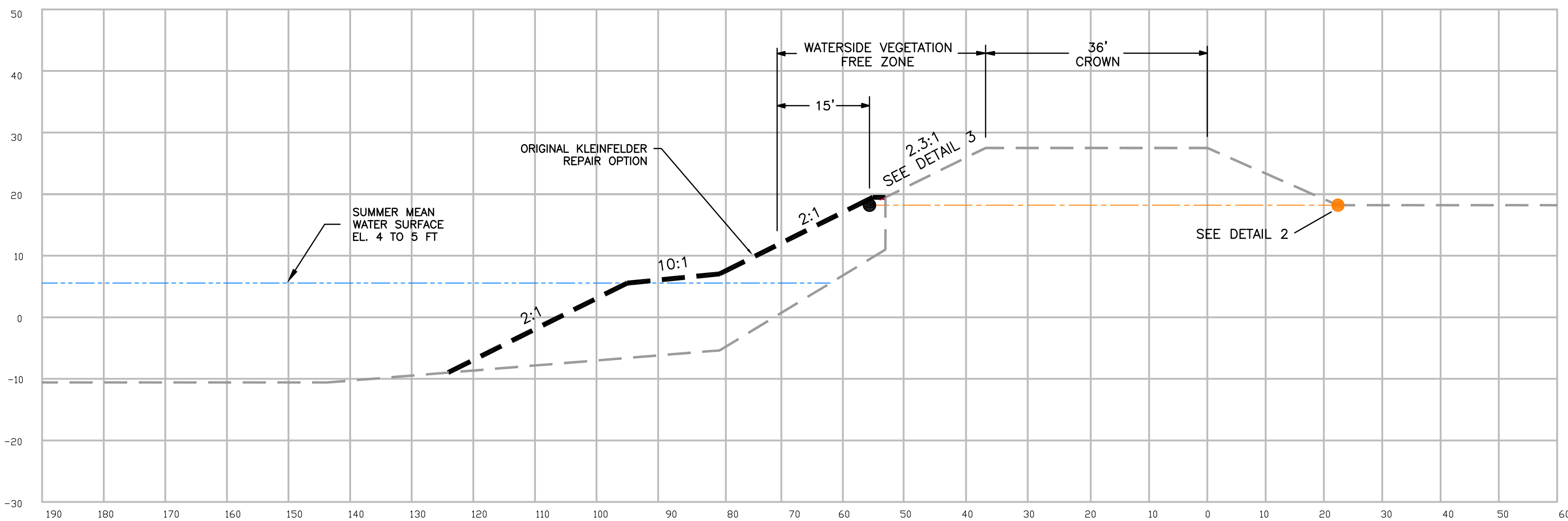
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

STEAMBOAT SLOUGH ~ 25.0L

Date	3/31/11
Figure	StS 25.0L

DETAIL 1

STEAMBOAT SLOUGH 25.8R



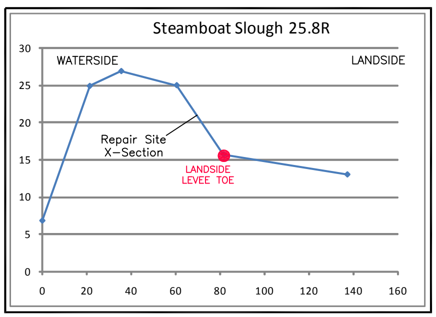
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	6.9
21.4	25.0
35.6	27.0
60.6	25.1
81.8	15.8
137.5	13.1

KEY DATA FOR DETERMINING LANDSIDE TOE

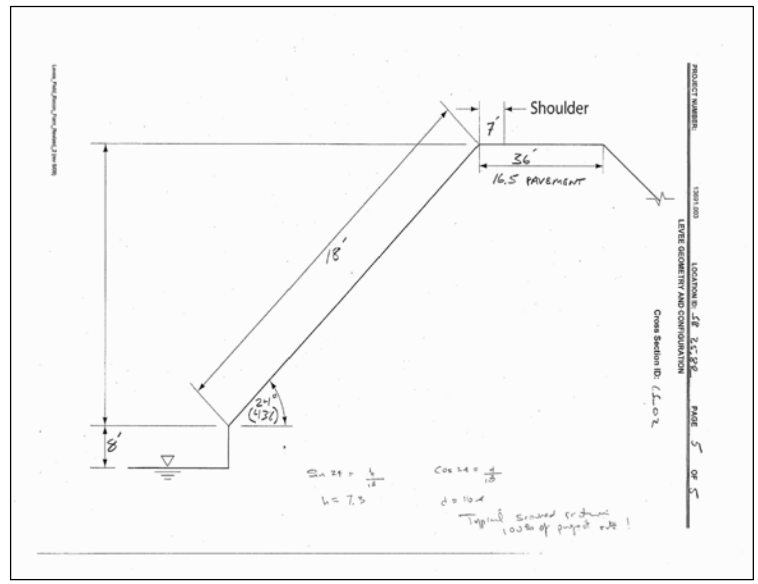
Comp Study Levee Crown Elevation	25.1
Comp Study Landside Toe Elevation	15.8
Comp Study Difference	9.3

Alternatives Report Analysis Levee Crown Elevation	27.5

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	18.2

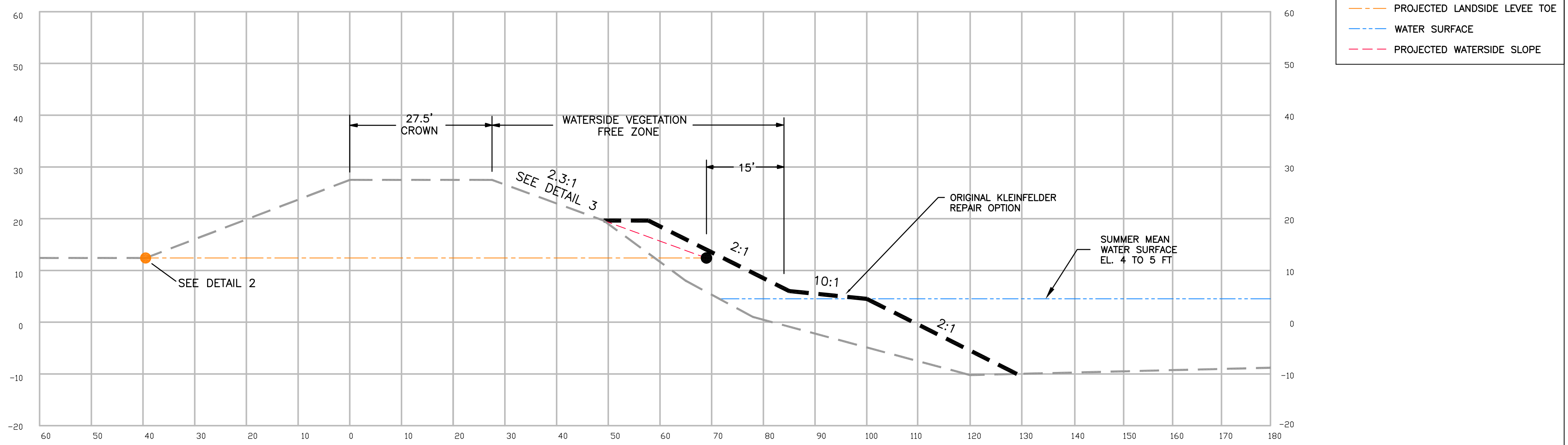
DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



DETAIL 1

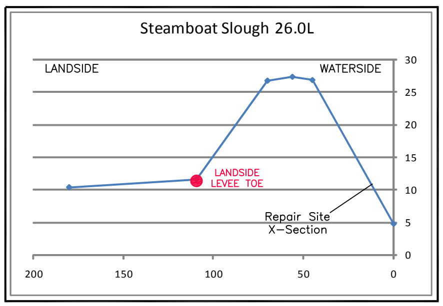
STEAMBOAT SLOUGH RIVER 26.0L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	4.9
45.0	26.9
56.1	27.4
69.9	26.8
109.4	11.7
179.9	10.5

KEY DATA FOR DETERMINING LANDSIDE TOE

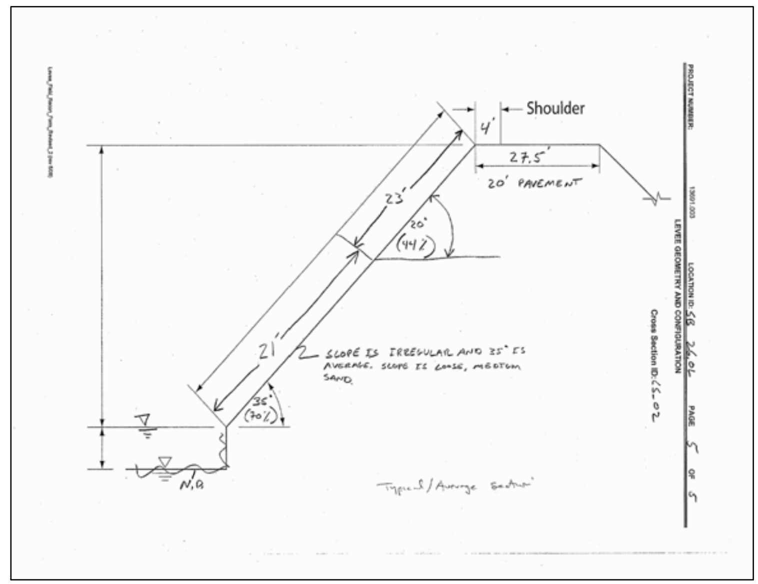
Comp Study Levee Crown Elevation	26.8
Comp Study Landside Toe Elevation	11.7
Comp Study Difference	15.1

Alternatives Report Analysis Levee Crown Elevation	27.5

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	12.4

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

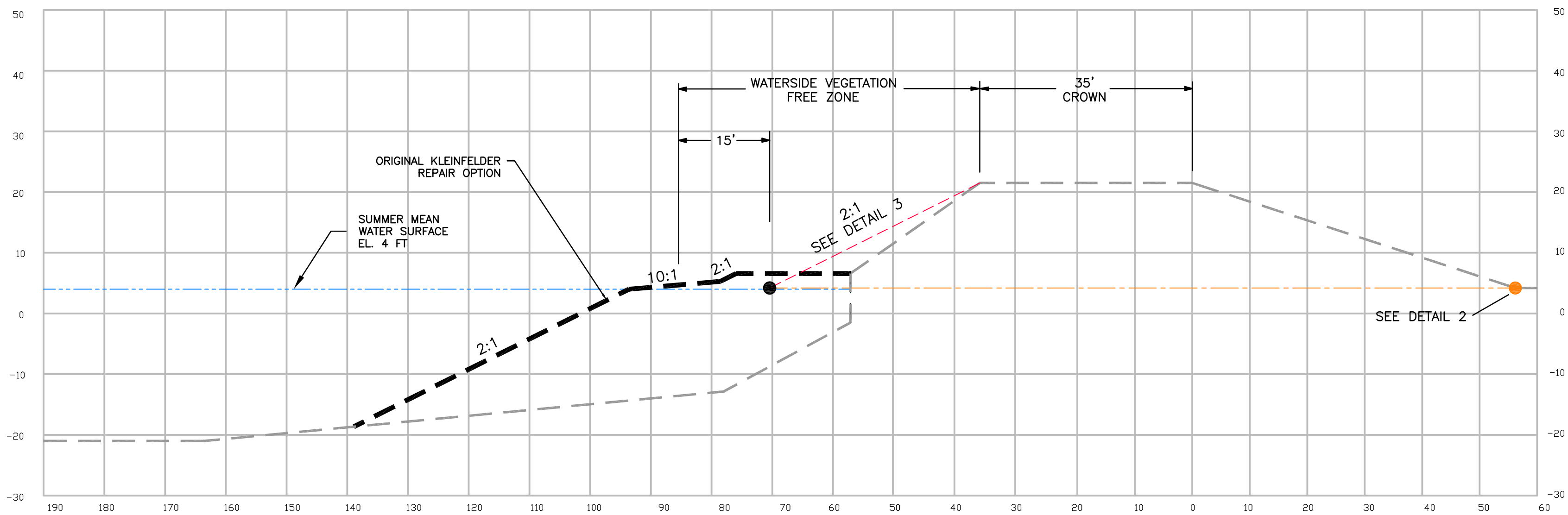
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

STEAMBOAT SLOUGH ~ 26.0L

Date	3/31/11
Figure	StS 26.0L

DETAIL 1

SUTTER SLOUGH RIVER 24.7R



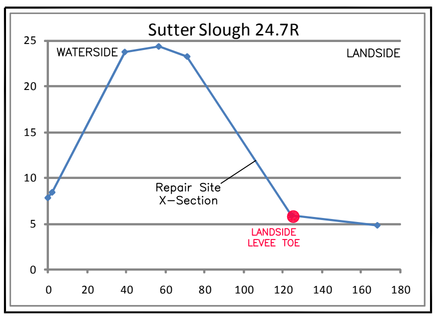
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	7.8
2.2	8.4
39.2	23.7
56.5	24.3
71.0	23.2
124.2	5.9
167.9	4.8

KEY DATA FOR DETERMINING LANDSIDE TOE

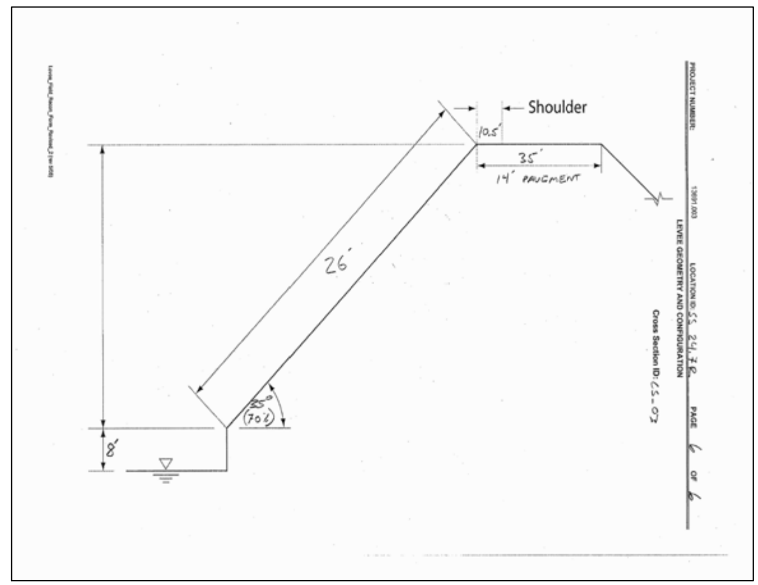
Comp Study Levee Crown Elevation	23.2
Comp Study Landside Toe Elevation	5.9
Comp Study Difference	17.3

Alternatives Report Analysis Levee Crown Elevation	21.5

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	4.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

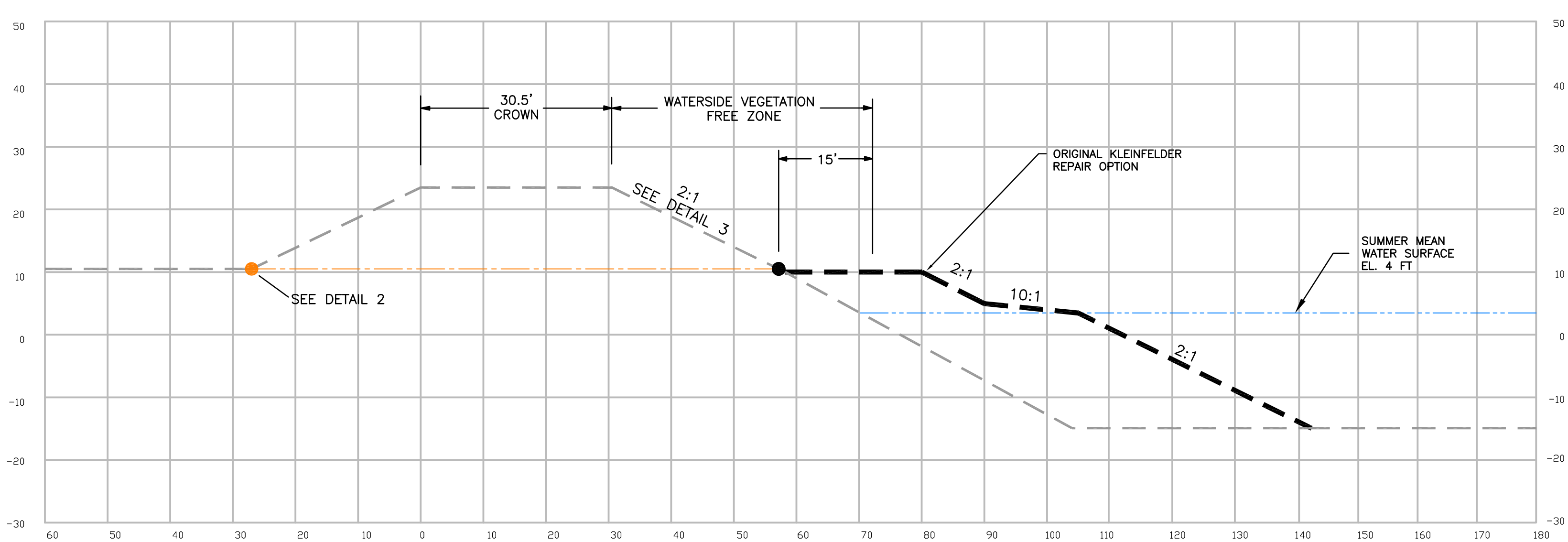
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SUTTER SLOUGH ~ 24.7R

Date	3/31/11
Figure	SS 24.7R

DETAIL 1

SUTTER SLOUGH RIVER 26.5L



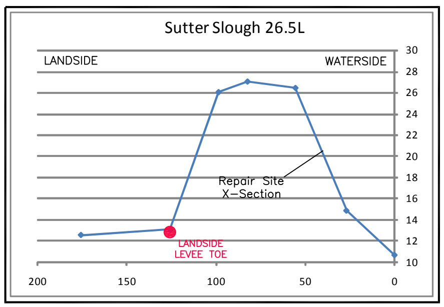
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	10.7
26.9	14.9
55.4	26.5
82.2	27.1
98.4	26.1
125.4	13.1
175.4	12.6

KEY DATA FOR DETERMINING
LANDSIDE TOE

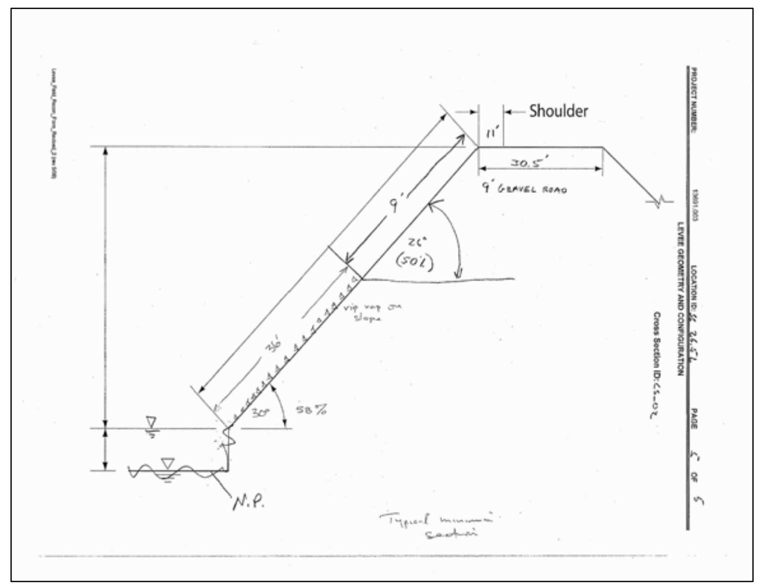
Comp Study Levee Crown Elevation	26.1
Comp Study Landside Toe Elevation	13.1
Comp Study Difference	13.0

Alternatives Report Analysis Levee Crown Elevation	23.5

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	10.5

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

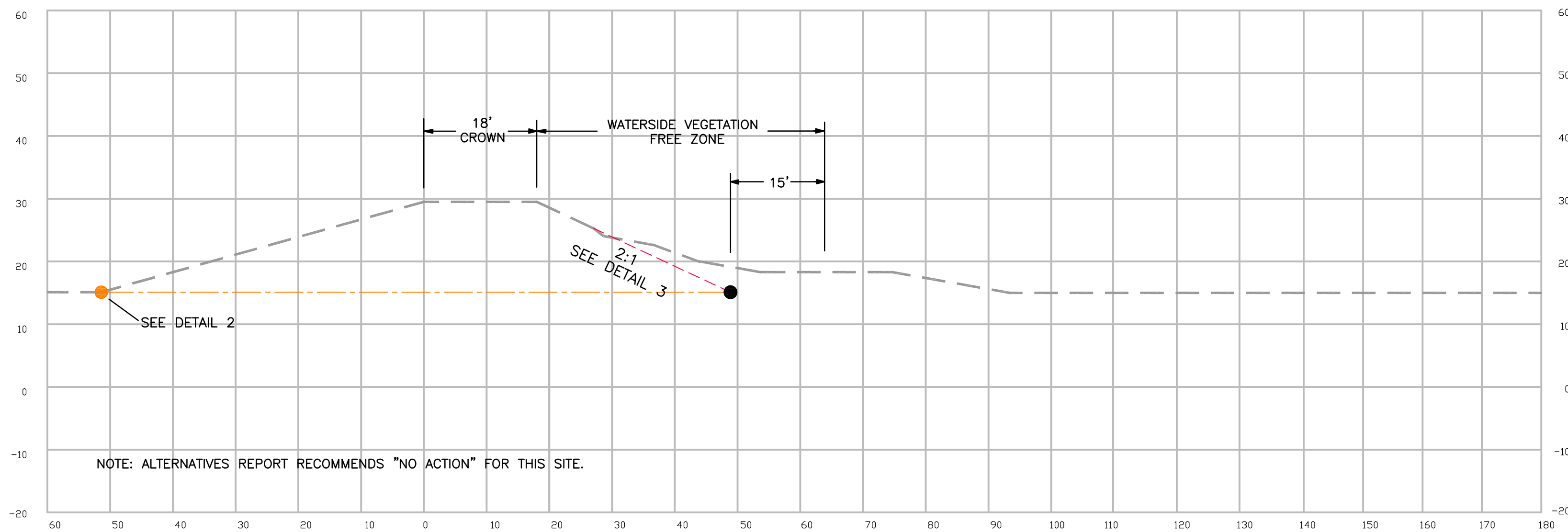
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SUTTER SLOUGH ~ 26.5L

Date	3/31/11
Figure	SS 26.5L

DETAIL 1

WILLOW SLOUGH BYPASS LEVEE 0.2L



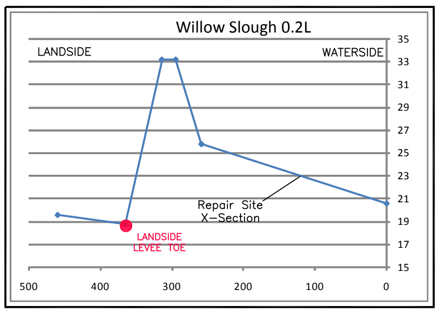
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	20.6
259.0	25.8
295.0	33.2
314.4	33.2
365.8	18.8
460.0	19.6

KEY DATA FOR DETERMINING
LANDSIDE TOE

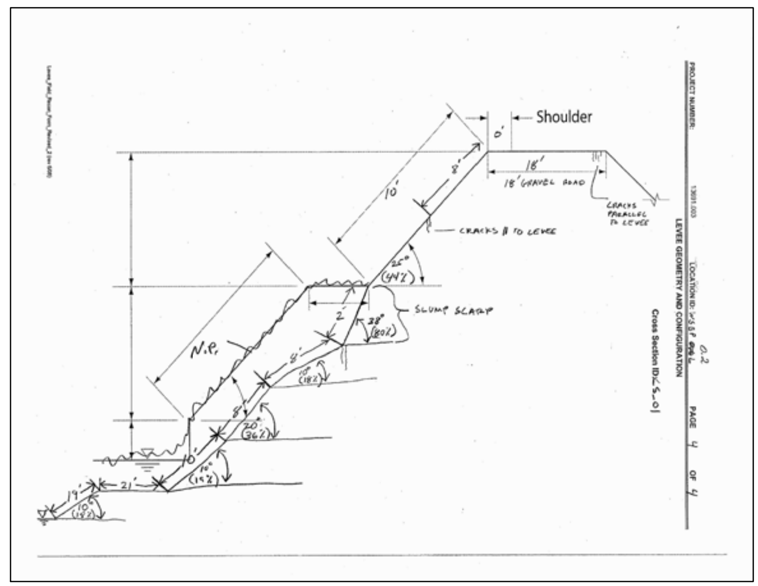
Comp Study Levee Crown Elevation	33.2
Comp Study Landside Toe Elevation	18.8
Comp Study Difference	14.4

Alternatives Report Analysis Levee Crown Elevation	29.5

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	15.1

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



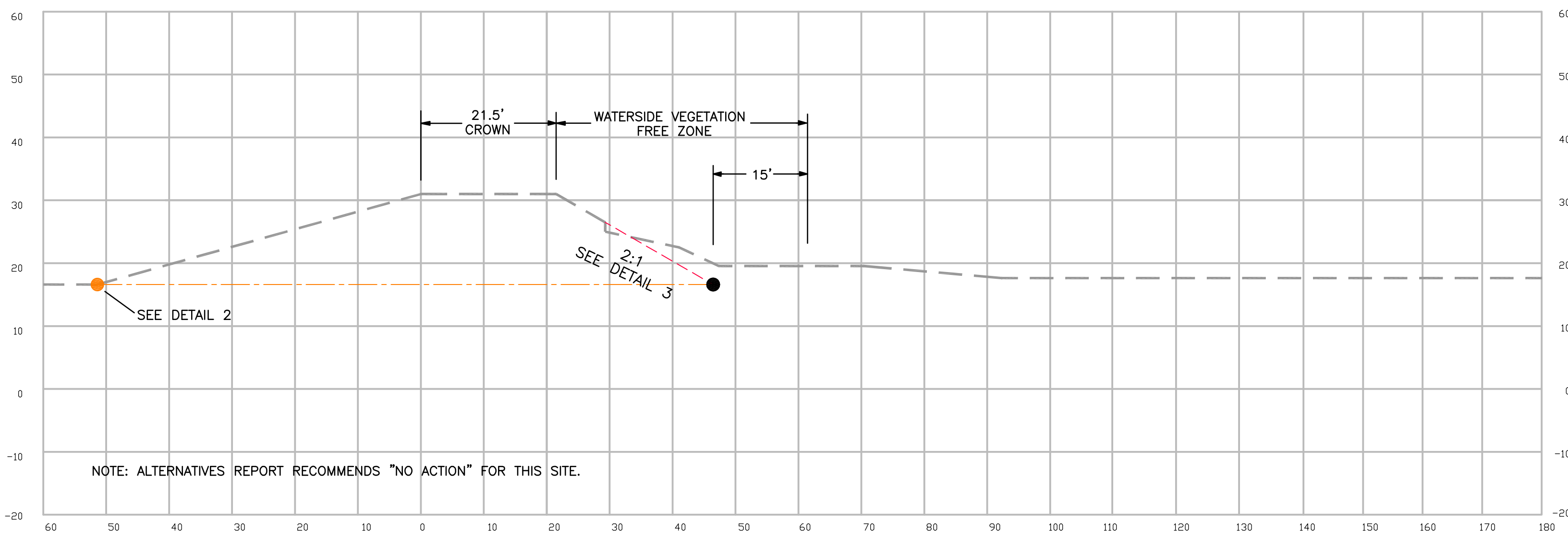
Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith	WILLOW SLOUGH BYPASS 0.2L	Date	3/31/11
Checked By: R. Little		Figure	WS 0.2L
QA/QC Review: D. Teak			

DETAIL 1

WILLOW SLOUGH BYPASS LEVEE 0.7L



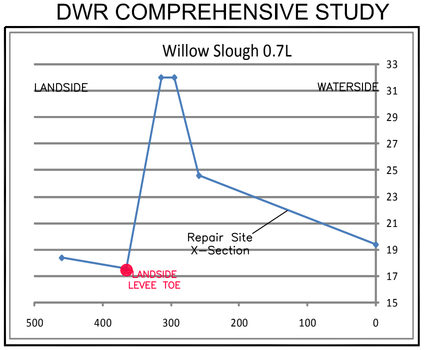
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	19.4
259.0	24.6
295.0	32.0
314.4	33.2
365.8	17.6
460.0	18.4

KEY DATA FOR DETERMINING
LANDSIDE TOE

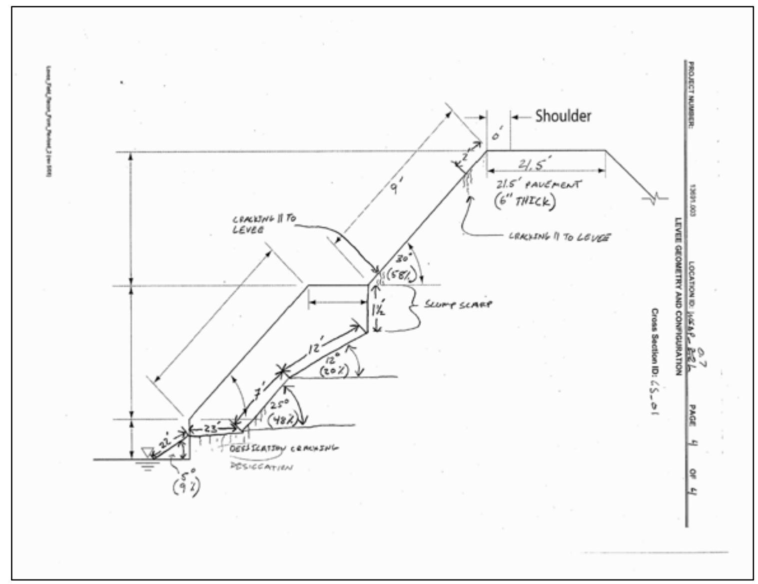
Comp Study Levee Crown Elevation	32.0
Comp Study Landside Toe Elevation	17.6
Comp Study Difference	14.4

Alternatives Report Analysis Levee Crown Elevation	31.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	16.6

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

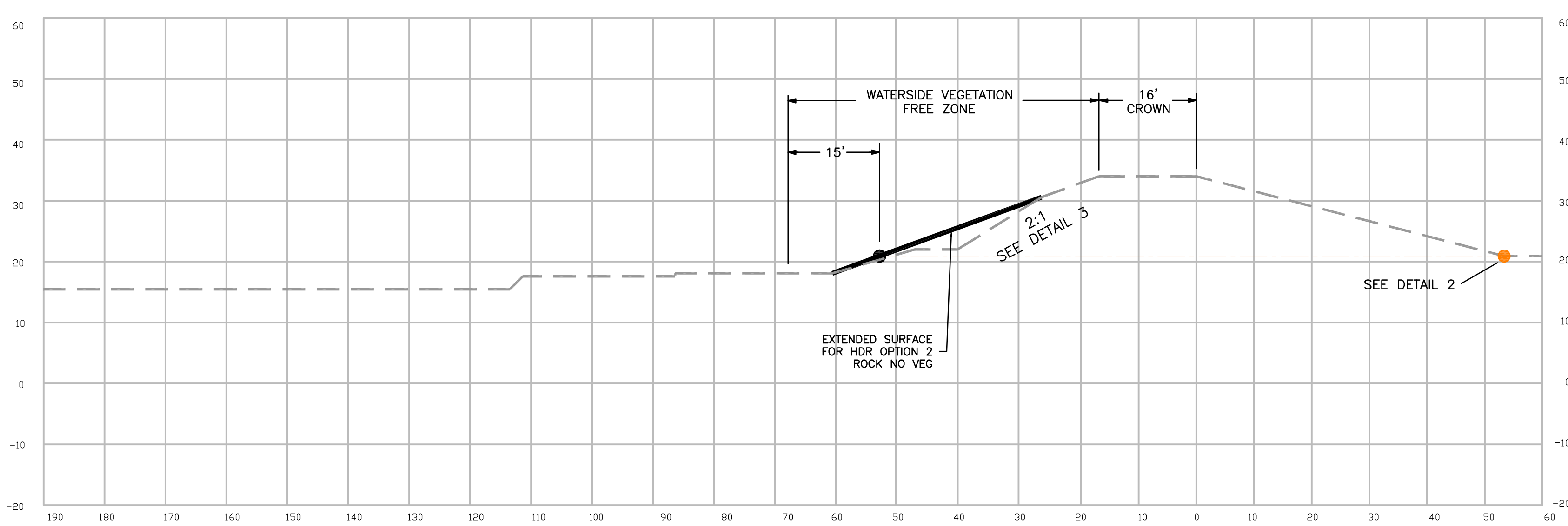
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

WILLOW SLOUGH BYPASS 0.7L

Date	3/31/11
Figure	WS 0.7L

DETAIL 1

WILLOW SLOUGH BYPASS LEVEE 6.9R



LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE
- PROPOSED HDR REPAIR SURFACE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY

Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	19.5
7.0	24.5
29.4	24.4
65.9	34.7
80.2	35.0
104.0	28.3
116.3	28.2
130.7	21.9

KEY DATA FOR DETERMINING LANDSIDE TOE

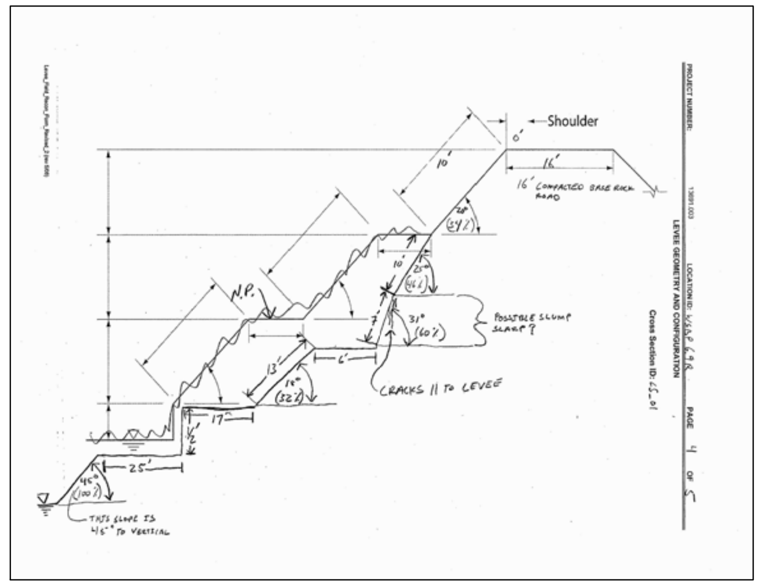
Comp Study Levee Crown Elevation	35.0
Comp Study Landside Toe Elevation	21.9
Comp Study Difference	13.1

Alternatives Report Analysis Levee Crown Elevation	34.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	20.9

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

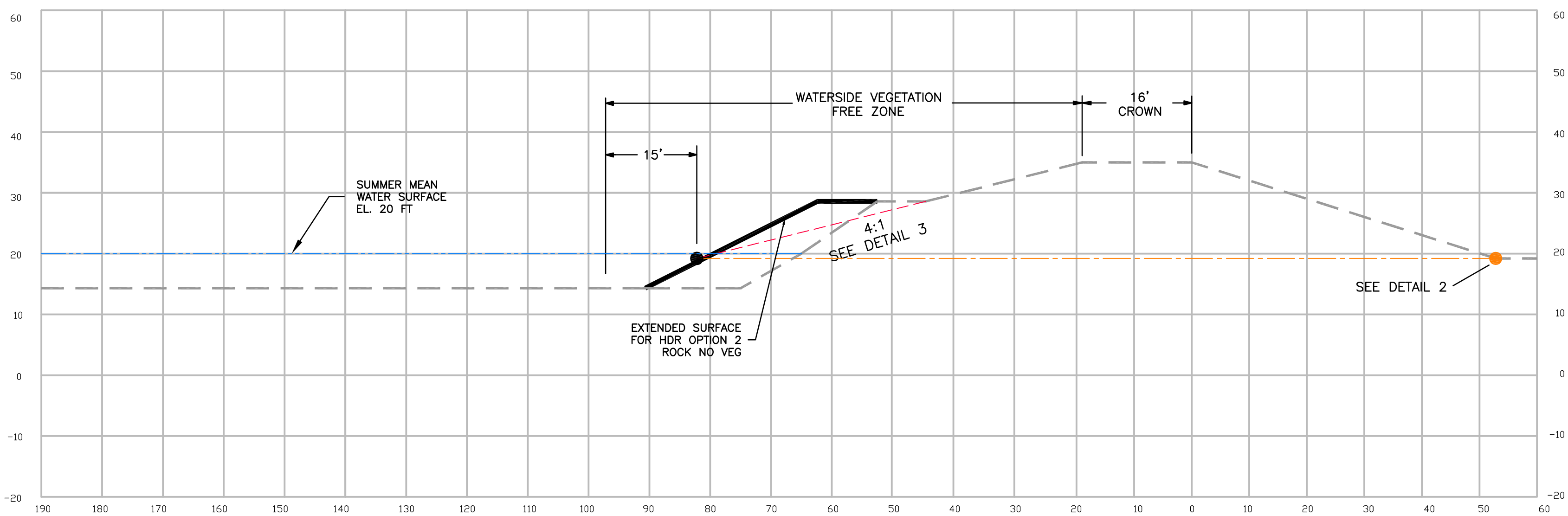
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

WILLOW SLOUGH BYPASS 6.9R

Date	3/31/11
Figure	WS 6.9R

DETAIL 1

YOLO BYPASS 2.0R



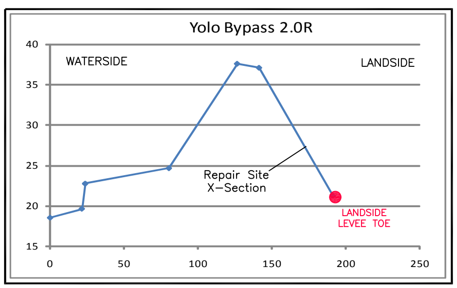
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE
- PROPOSED HDR REPAIR SURFACE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	18.6
21.4	19.7
23.7	22.8
80.3	24.7
126.4	37.6
141.2	37.1
191.2	21.3

KEY DATA FOR DETERMINING LANDSIDE TOE

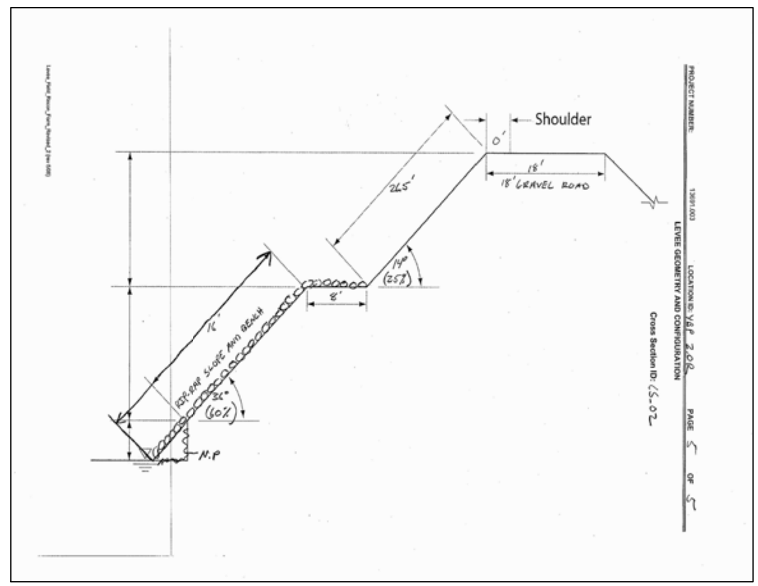
Comp Study Levee Crown Elevation	37.1
Comp Study Landside Toe Elevation	21.3
Comp Study Difference	15.8

Alternatives Report Analysis Levee Crown Elevation	35.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	19.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

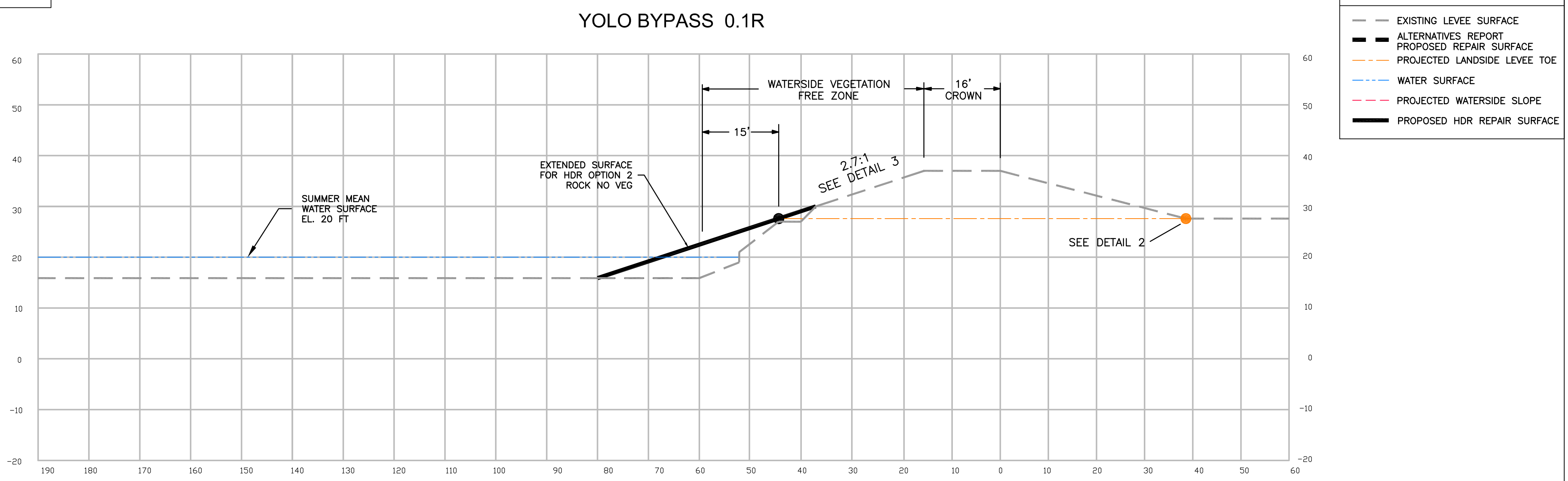
HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

YOLO BYPASS ~ 2.0R

Date	3/31/11
Figure	YBP2 2.0R

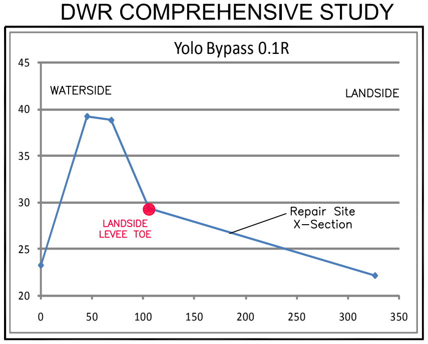
DETAIL 1



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	23.3
45.2	39.2
68.8	38.8
105.3	29.4
326.4	22.2

KEY DATA FOR DETERMINING LANDSIDE TOE

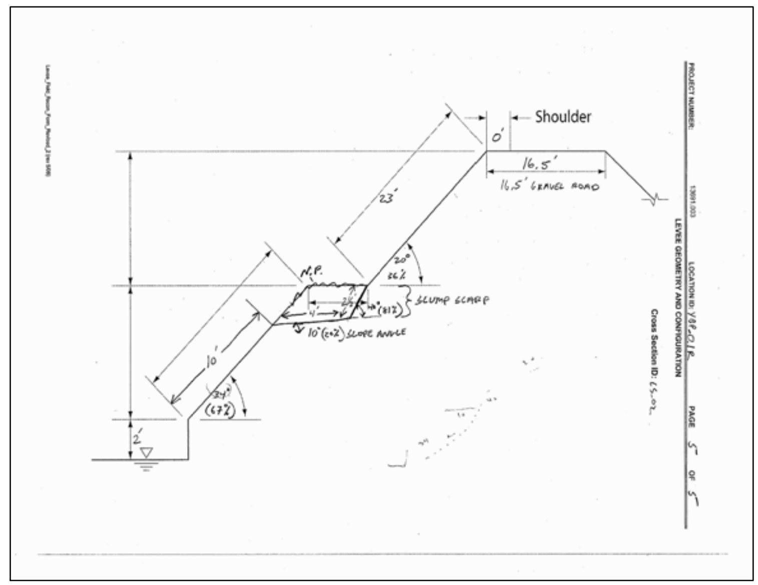
Comp Study Levee Crown Elevation	38.8
Comp Study Landside Toe Elevation	29.4
Comp Study Difference	9.4

Alternatives Report Analysis Levee Crown Elevation	37.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	27.6

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

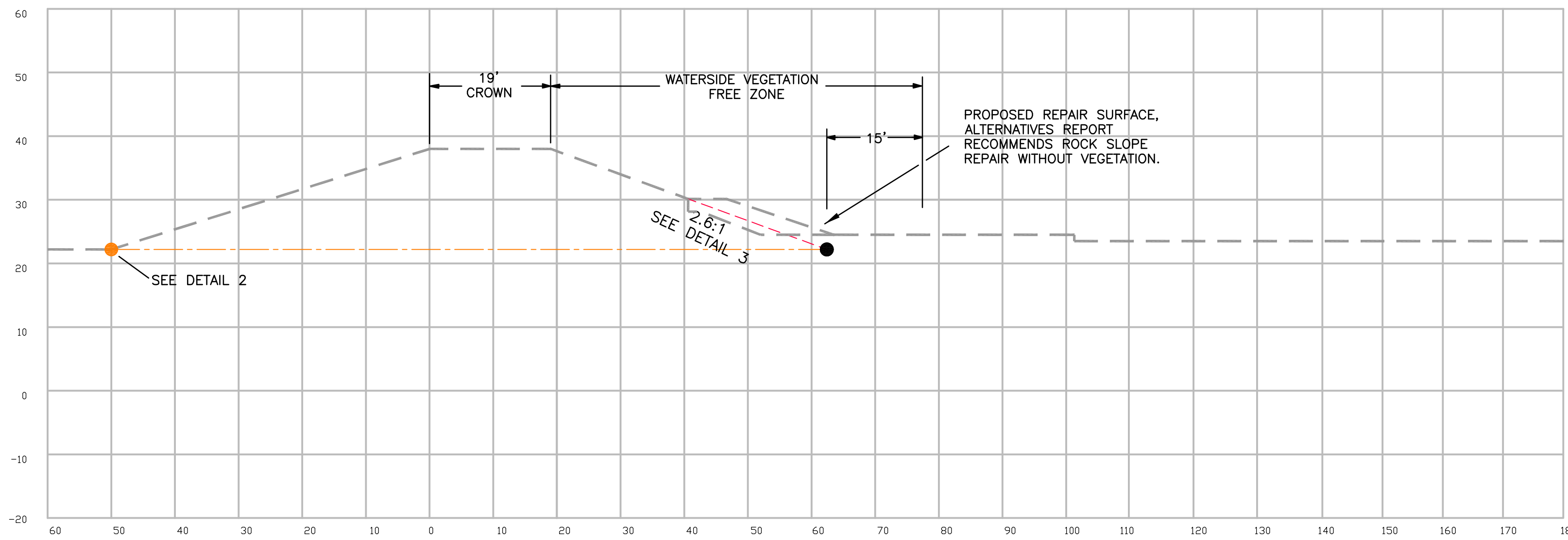
YOLO BYPASS ~ 0.1R

Date 3/31/11

Figure YBP2 0.1R

DETAIL 1

YOLO BYPASS 2.5L



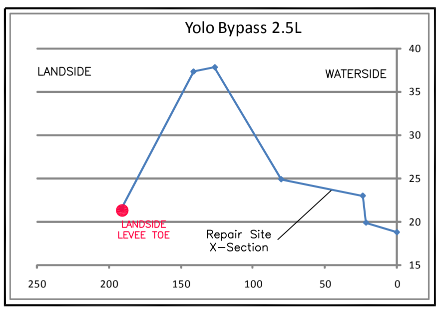
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	18.8
21.4	19.9
23.7	23.0
80.3	24.9
126.4	37.8
141.2	37.3
191.2	21.5

KEY DATA FOR DETERMINING LANDSIDE TOE

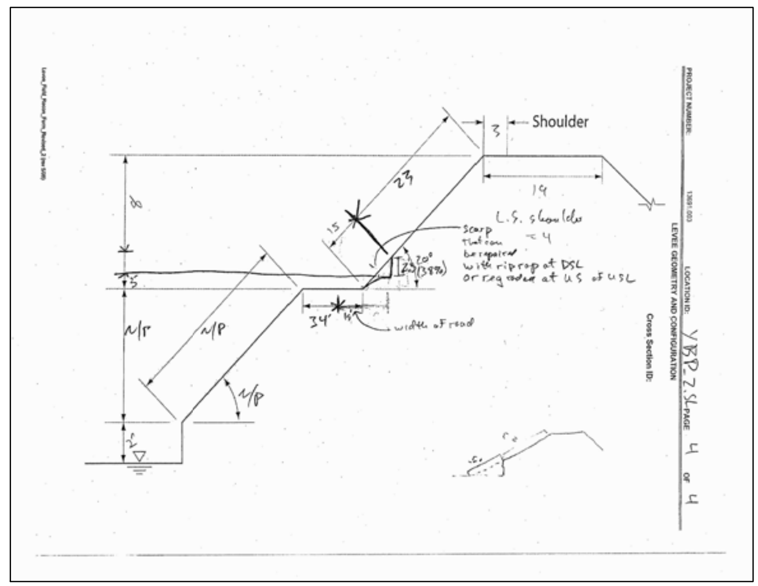
Comp Study Levee Crown Elevation	37.3
Comp Study Landside Toe Elevation	21.5
Comp Study Difference	15.8

Alternatives Report Analysis Levee Crown Elevation	38

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	22.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

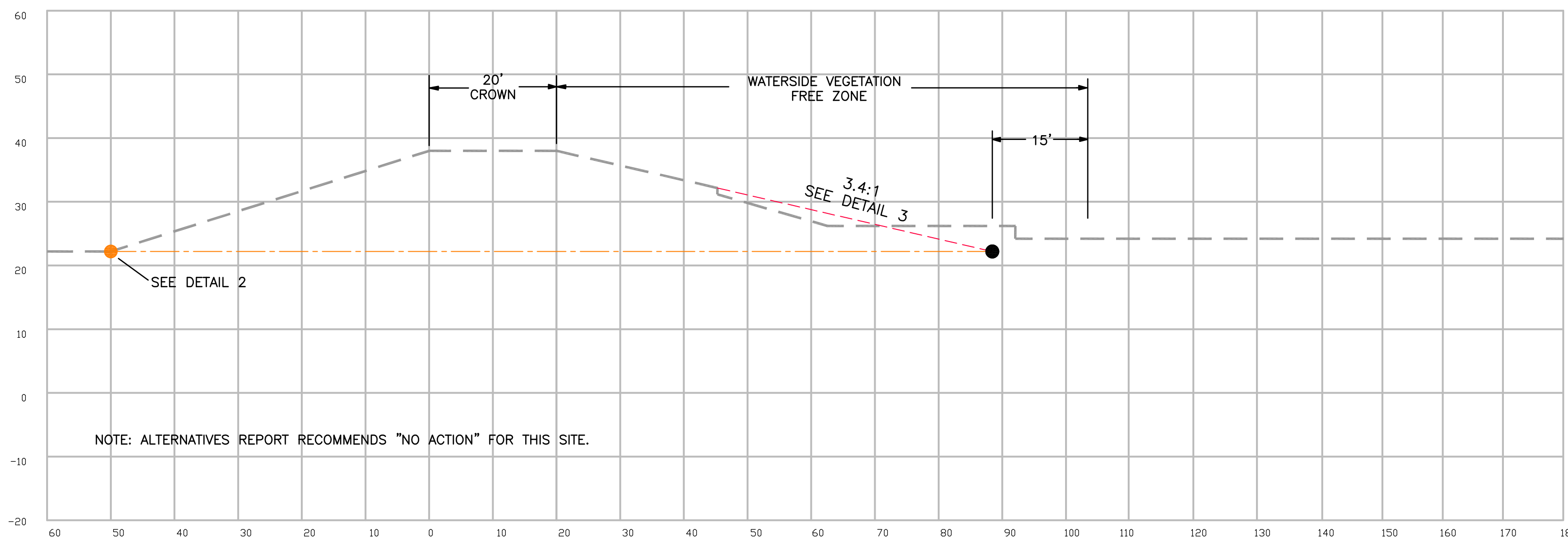
HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

YOLO BYPASS ~ 2.5L

Date	3/31/11
Figure	YBP1 2.5L

DETAIL 1



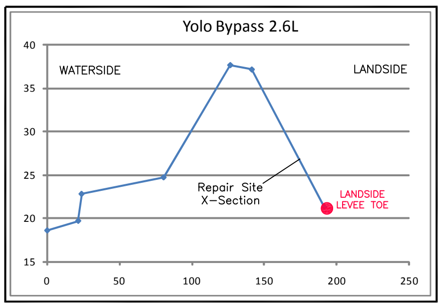
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	18.7
21.4	19.8
23.7	22.9
80.3	24.8
126.4	37.7
141.2	37.2
191.2	21.4

KEY DATA FOR DETERMINING LANDSIDE TOE

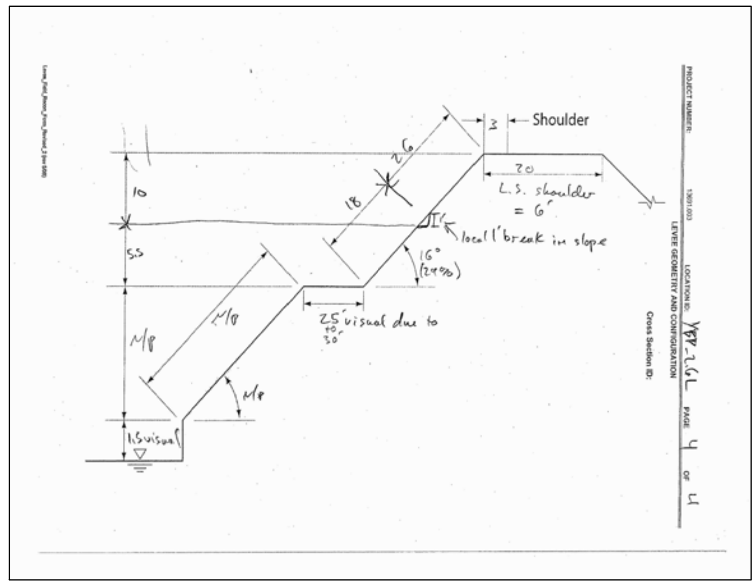
Comp Study Levee Crown Elevation	37.2
Comp Study Landside Toe Elevation	21.4
Comp Study Difference	15.8

Alternatives Report Analysis Levee Crown Elevation	38

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	22.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

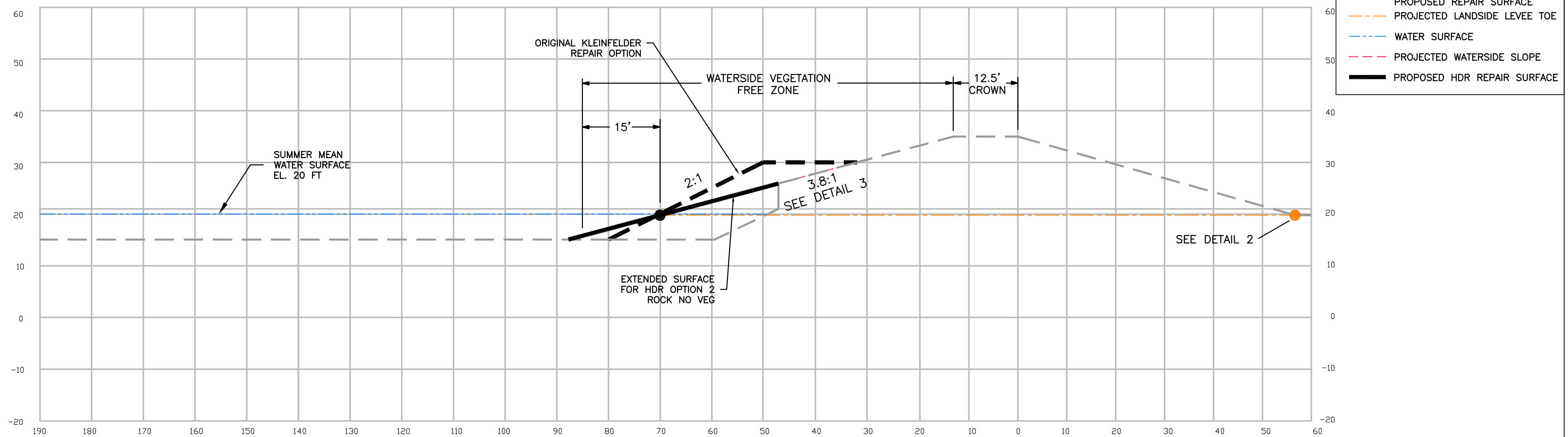
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

YOLO BYPASS ~ 2.6L

Date	3/31/11
Figure	YBP1 2.6L

DETAIL 1

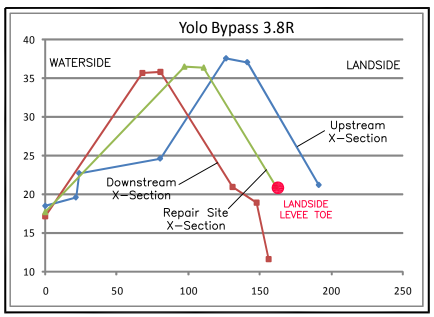
YOLO BYPASS LEVEE 3.8R



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	17.7
97.3	36.5
110.8	36.3
161.1	21.1

KEY DATA FOR DETERMINING LANDSIDE TOE

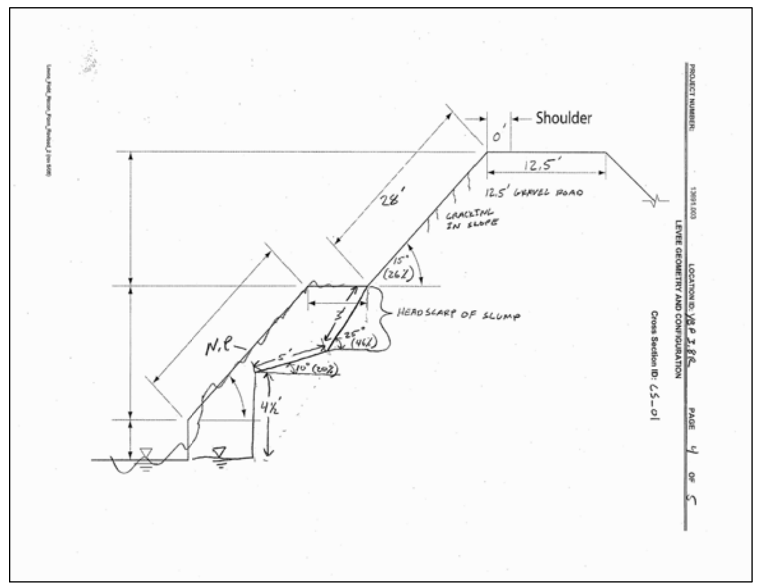
Comp Study Levee Crown Elevation	36.3
Comp Study Landside Toe Elevation	21.1
Comp Study Difference	15.2

Alternatives Report Analysis Levee Crown Elevation	35.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	19.8

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

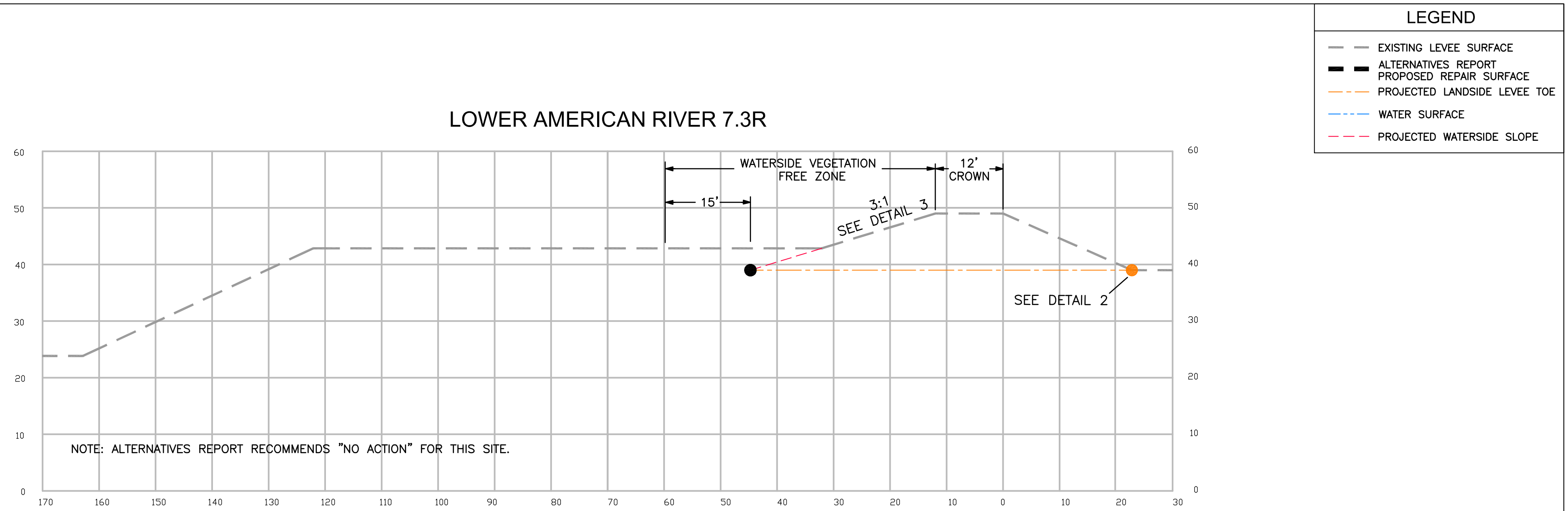
HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

YOLO BYPASS ~ 3.8R

Date 3/31/11
Figure YBP2 3.8R

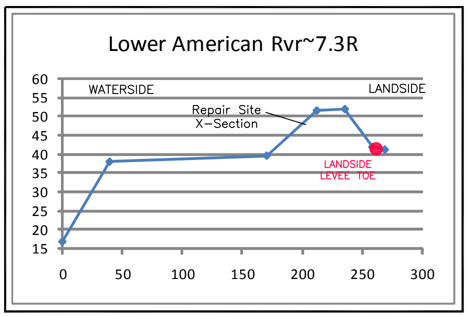
DETAIL 1



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	16.9
39.4	38.0
170.4	39.6
212.2	51.5
235.6	51.9
258.4	41.9
268.9	41.2

KEY DATA FOR DETERMINING LANDSIDE TOE

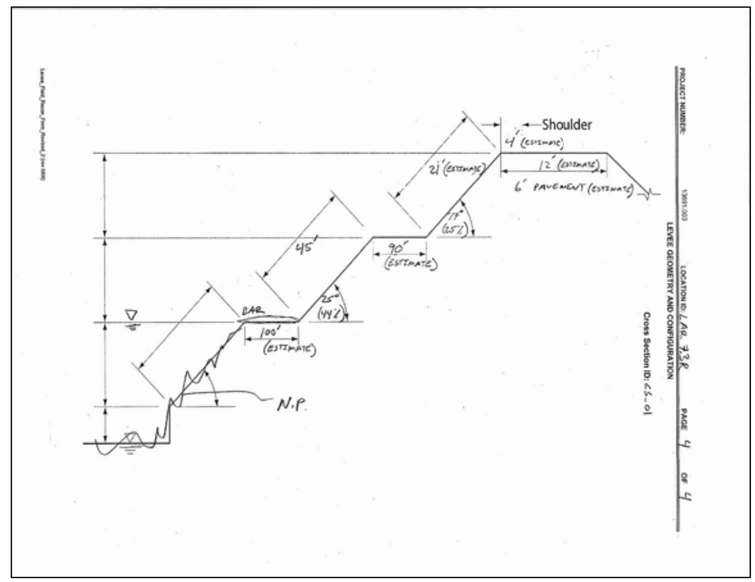
Comp Study Levee Crown Elevation	51.9
Comp Study Landside Toe Elevation	41.9
Comp Study Difference	10.0

Alternatives Report Analysis Levee Crown Elevation	49.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	39.0

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

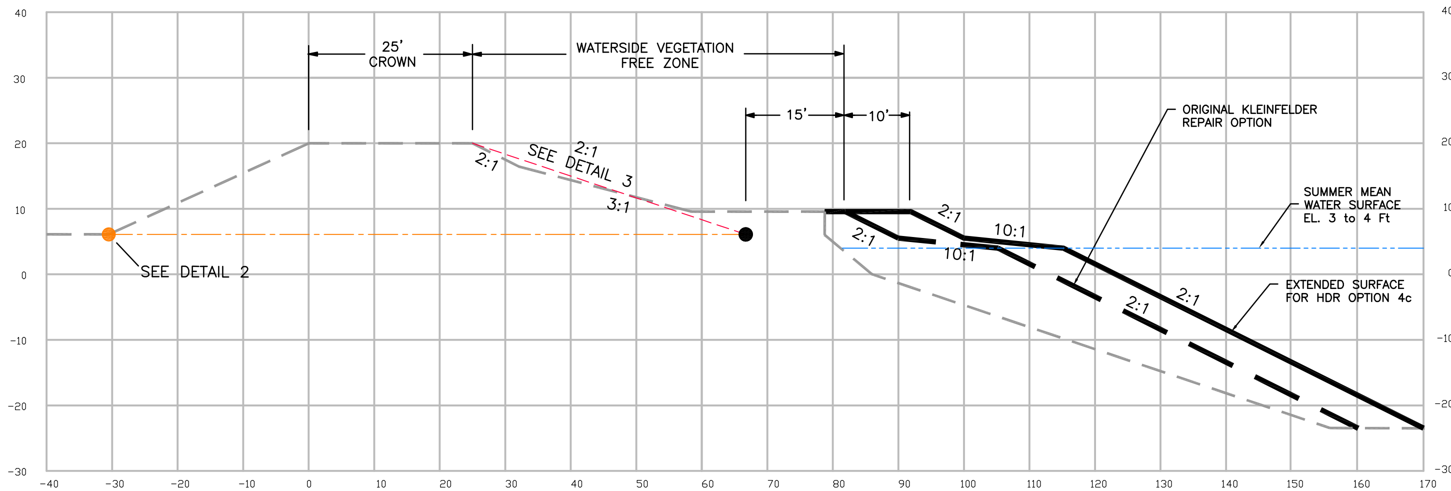
HDR Engineering, Inc.

Drawn By: E. Smith	LOWER AMERICAN RIVER ~ 7.3R	Date	3/31/11
Checked By: R. Little		Figure	LAR 7.3R
QA/QC Review: D. Teak			

DETAIL 1

LEGEND
EXISTING LEVEE SURFACE
ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
PROJECTED LANDSIDE LEVEE TOE
WATER SURFACE
PROJECTED WATERSIDE SLOPE
HDR PROPOSED REPAIR SURFACE

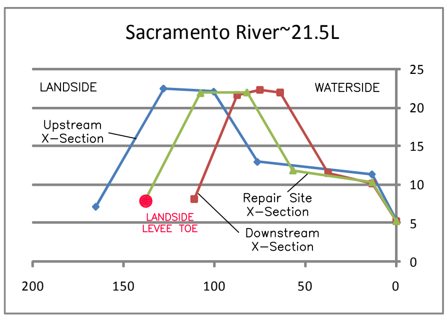
SACRAMENTO RIVER 21.5L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.3
13.2	10.4
56.8	11.9
82.1	21.9
107.6	21.9
138.1	8.0

KEY DATA FOR DETERMINING LANDSIDE TOE

Comp Study Levee Crown Elevation	21.9
Comp Study Landside Toe Elevation	8.0
Comp Study Difference	13.9

Alternatives Report Analysis Levee Crown Elevation	20.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	6.1

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES

NO EXISTING FIELD NOTES

Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

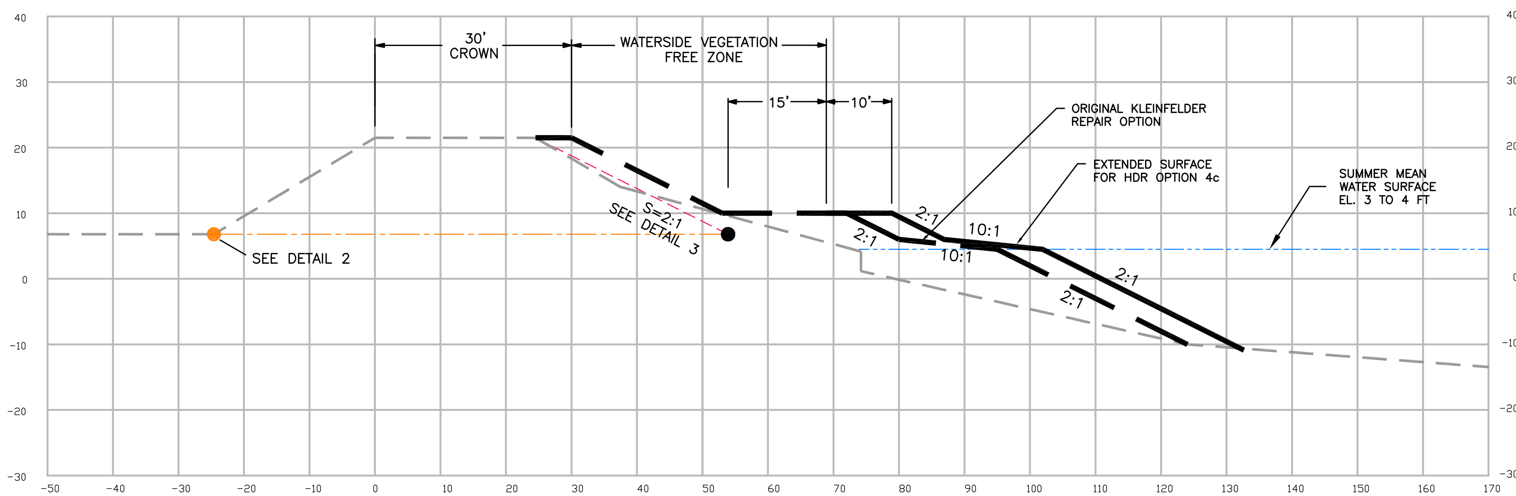
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 21.5L

Date	3/31/11
Figure	SAC 21.5L

DETAIL 1

SACRAMENTO RIVER 22.5L

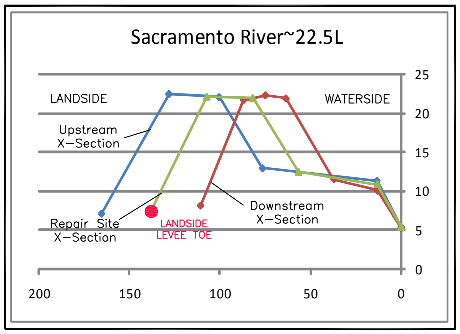


LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE
- HDR PROPOSED REPAIR SURFACE

DETAIL 2

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



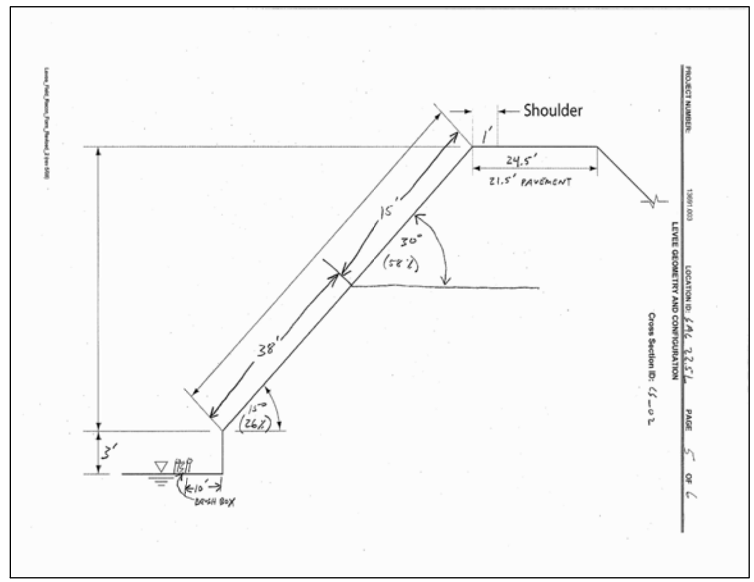
Comp Study Data Points	
Horizontal Distance From Waters Edge	Elevation
0	5.4
13.2	10.9
56.8	12.5
82.1	22.0
107.6	22.2
107.9	7.6

Waterside Toe Based on Alternatives Analysis & Comp. Study	
Comp Study Levee Crown Elevation	22.2
Comp Study Landside Toe Elevation	7.6
Comp Study Difference	14.7
Comp Study Analysis Levee Crown Elevation	21.5

Waterside Toe Defined as: (Comp. Study Crown Elev.) - (Difference)	
Determined Waterside Toe	6.8

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



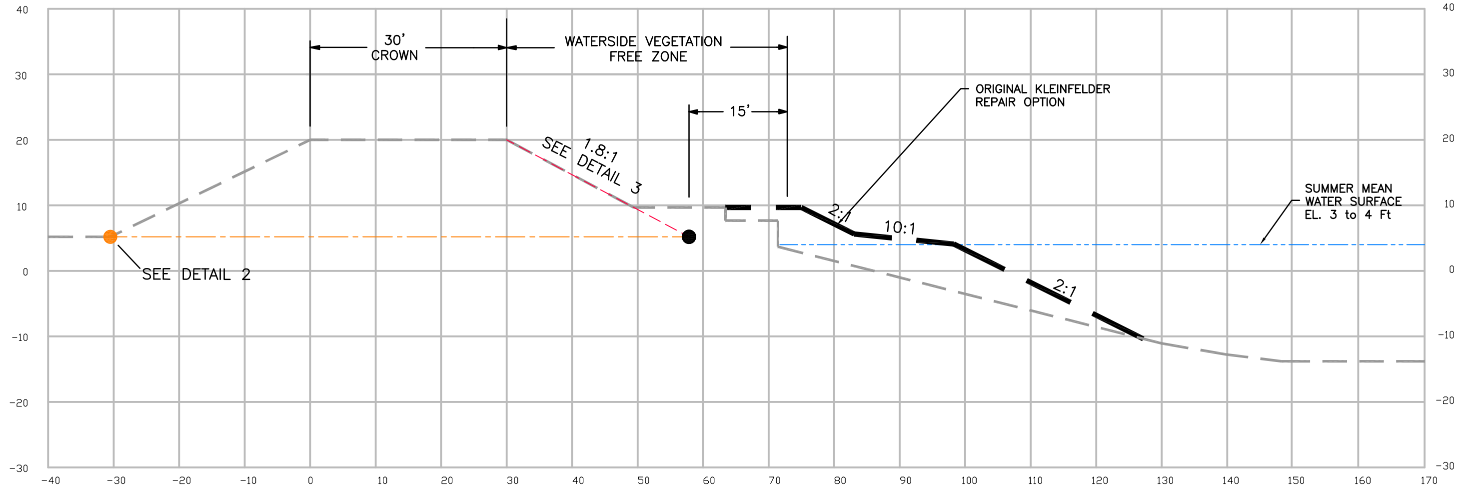
Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith	SACRAMENTO RIVER ~ 22.5L	Date	3/31/11
Checked By: R. Little		Figure	SAC 22.5L
QA/QC Review: D. Teak			

DETAIL 1

SACRAMENTO RIVER 22.7L



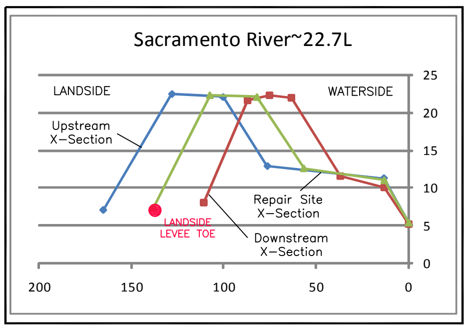
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.4
13.2	11.1
56.8	12.6
82.1	22.0
107.6	22.3
138.1	7.5

KEY DATA FOR DETERMINING LANDSIDE TOE

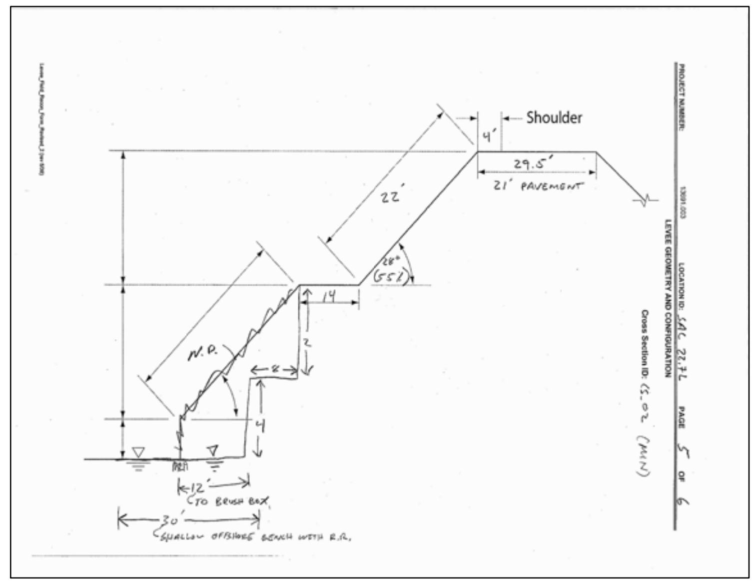
Comp Study Levee Crown Elevation	22.3
Comp Study Landside Toe Elevation	7.5
Comp Study Difference	14.8

Alternatives Report Analysis Levee Crown Elevation	20.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	5.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 22.7L

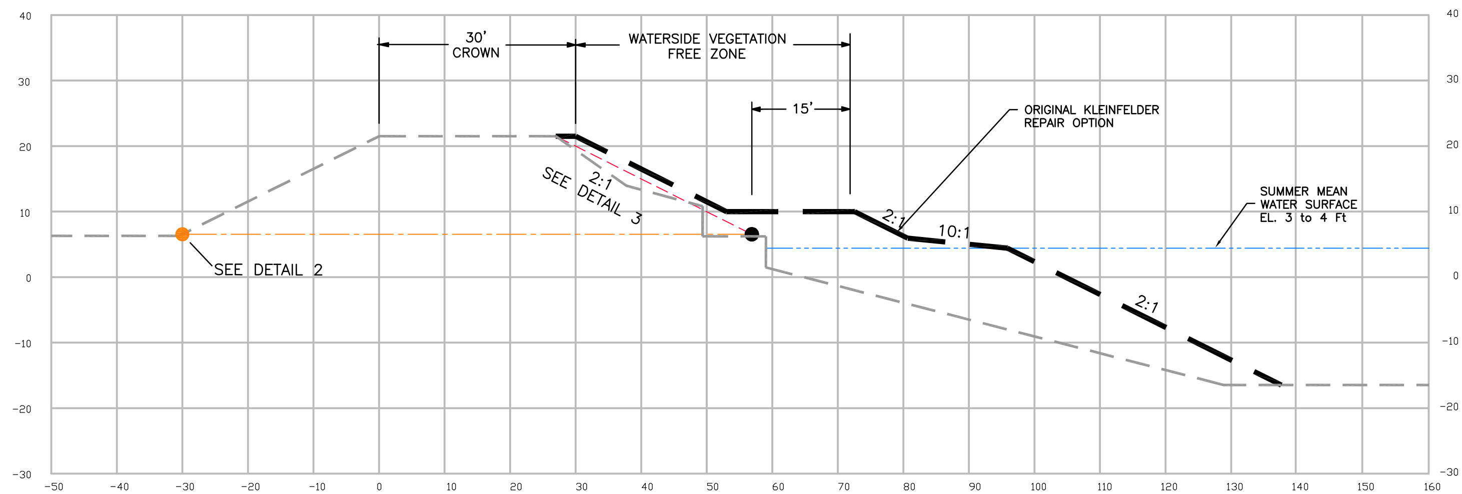
Date	3/31/11
Figure	SAC 22.7L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

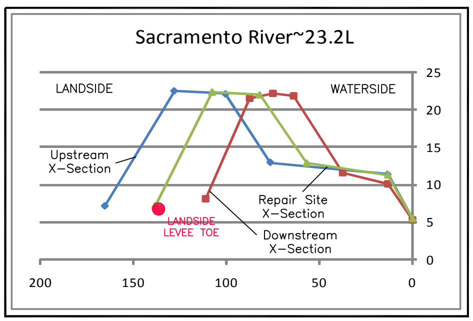
SACRAMENTO RIVER 23.2L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.5
13.2	11.3
56.8	12.9
82.1	22.1
107.6	22.5
138.1	7.2

KEY DATA FOR DETERMINING LANDSIDE TOE

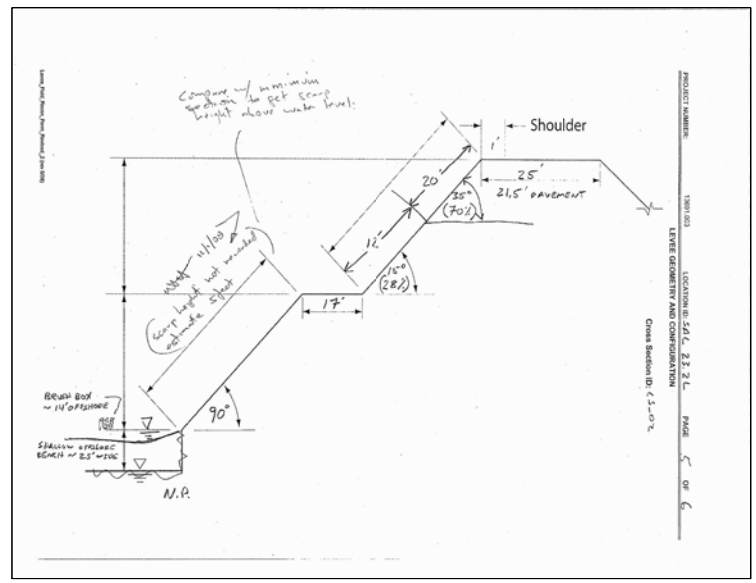
Comp Study Levee Crown Elevation	22.5
Comp Study Landside Toe Elevation	7.2
Comp Study Difference	15.2

Alternatives Report Analysis Levee Crown Elevation	21.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	6.3

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 23.2L

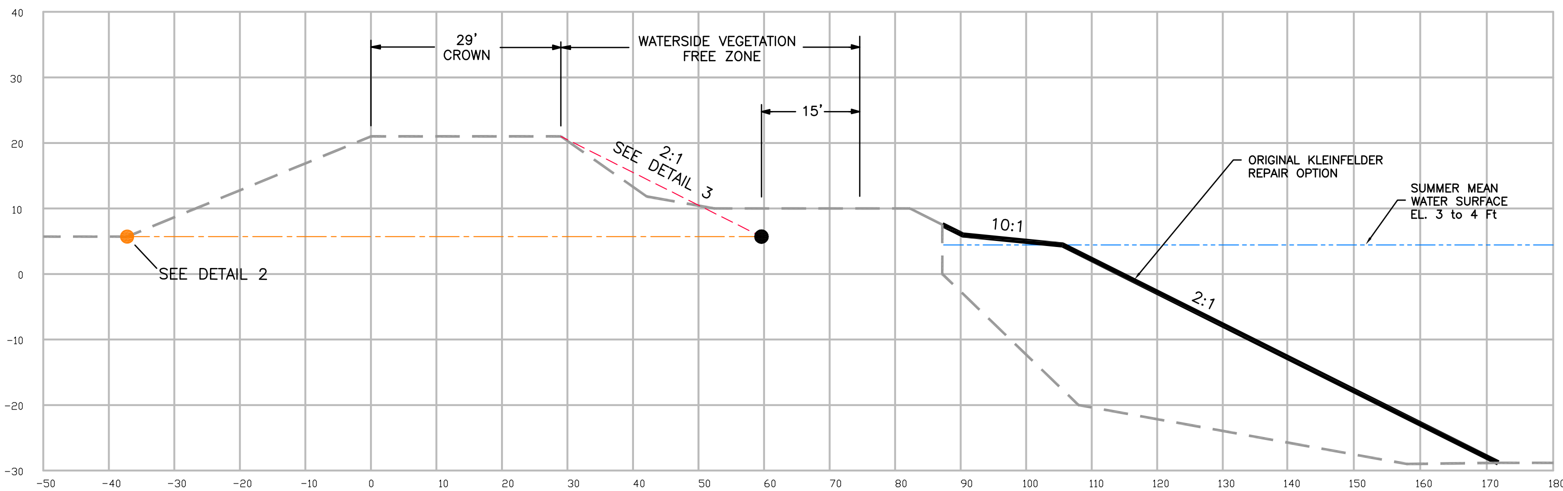
Date	3/31/11
Figure	SAC 23.2L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

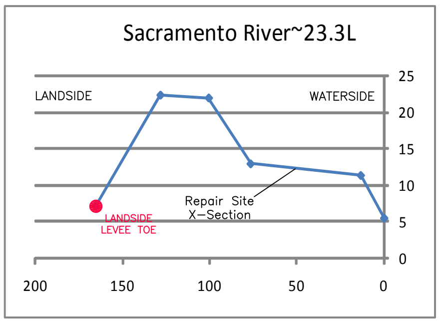
SACRAMENTO RIVER 23.3L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.5
13.4	11.4
76.4	13.0
100.4	22.1
128.0	22.5
165.2	7.2

KEY DATA FOR DETERMINING LANDSIDE TOE

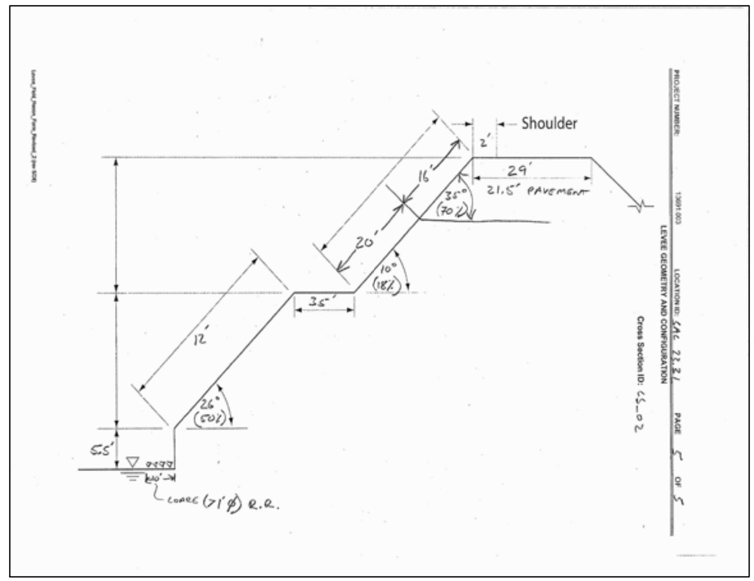
Comp Study Levee Crown Elevation	22.5
Comp Study Landside Toe Elevation	7.2
Comp Study Difference	15.3

Alternatives Report Analysis Levee Crown Elevation	21.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	5.7

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

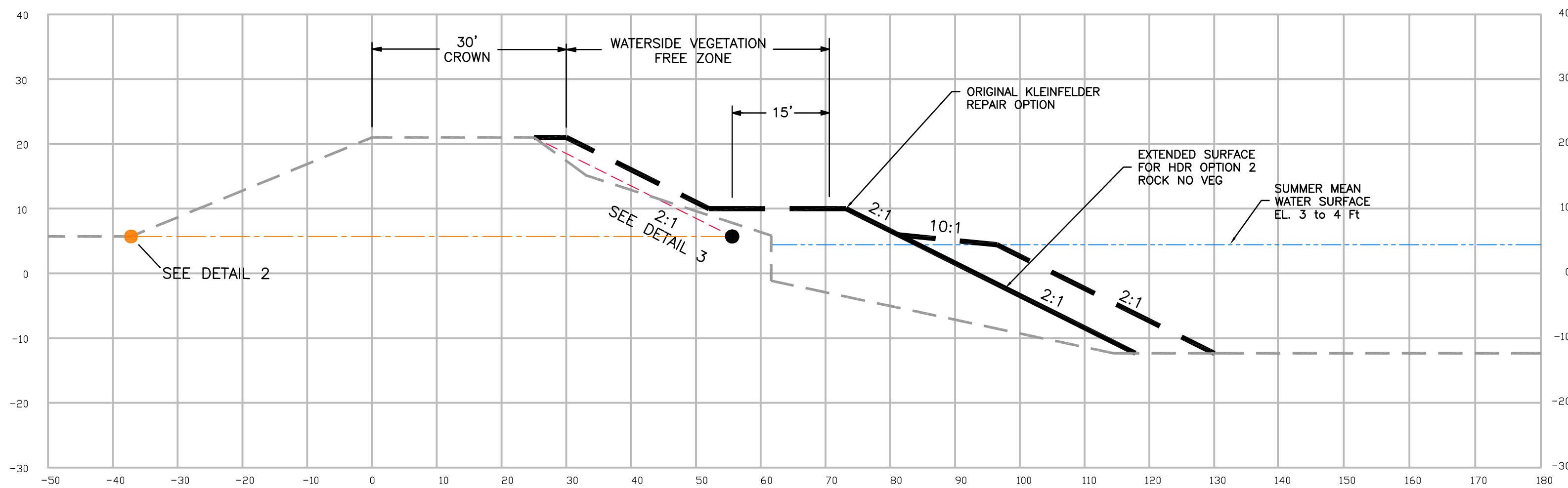
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 23.3L

Date	3/31/11
Figure	SAC 23.3L

DETAIL 1

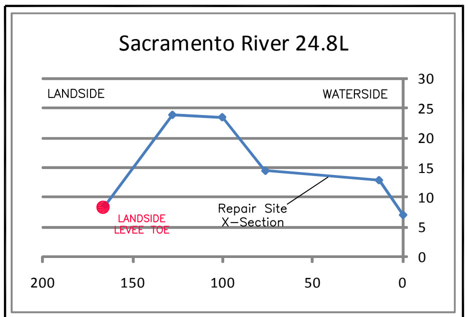
LEGEND
- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE
- HDR PROPOSED REPAIR SURFACE



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	7.1
13.4	13.0
76.4	14.6
100.4	23.7
128.0	24.1
165.2	8.8

KEY DATA FOR DETERMINING
LANDSIDE TOE

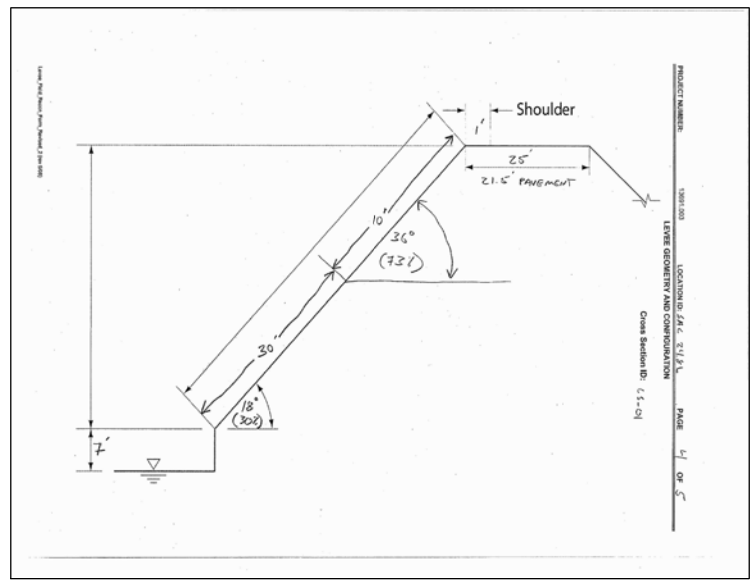
Comp Study Levee Crown Elevation	24.1
Comp Study Landside Toe Elevation	8.8
Comp Study Difference	15.3

Alternatives Report Analysis Levee Crown Elevation	21.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	5.7

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

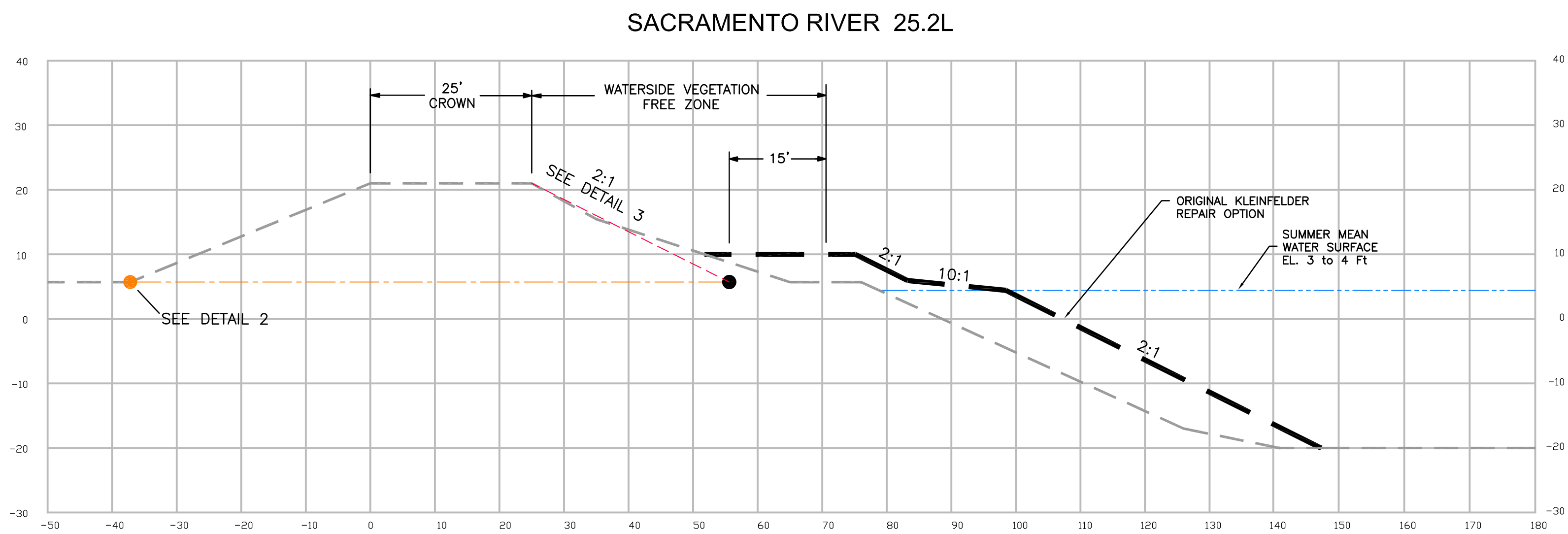
HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 24.8L

Date	3/31/11
Figure	SAC 24.8L

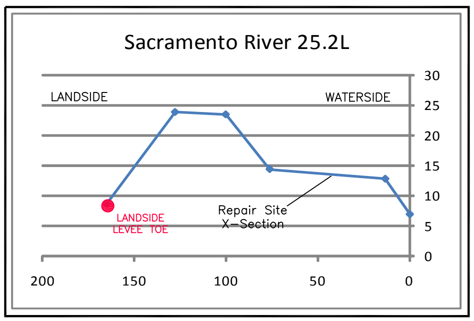
DETAIL 1



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	7.0
13.4	12.9
76.4	14.5
100.4	23.6
128.0	24.0
165.2	8.7

KEY DATA FOR DETERMINING LANDSIDE TOE

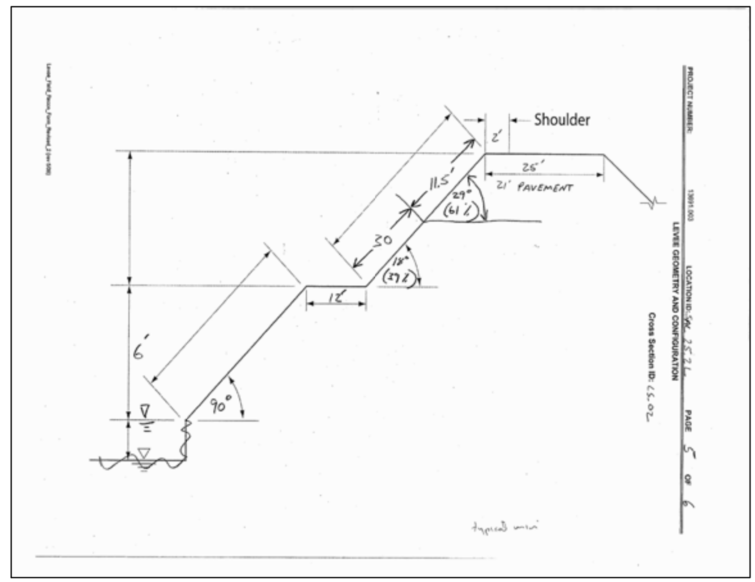
Comp Study Levee Crown Elevation	24.0
Comp Study Landside Toe Elevation	8.7
Comp Study Difference	15.3

Alternatives Report Analysis Levee Crown Elevation	22.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	6.7

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES

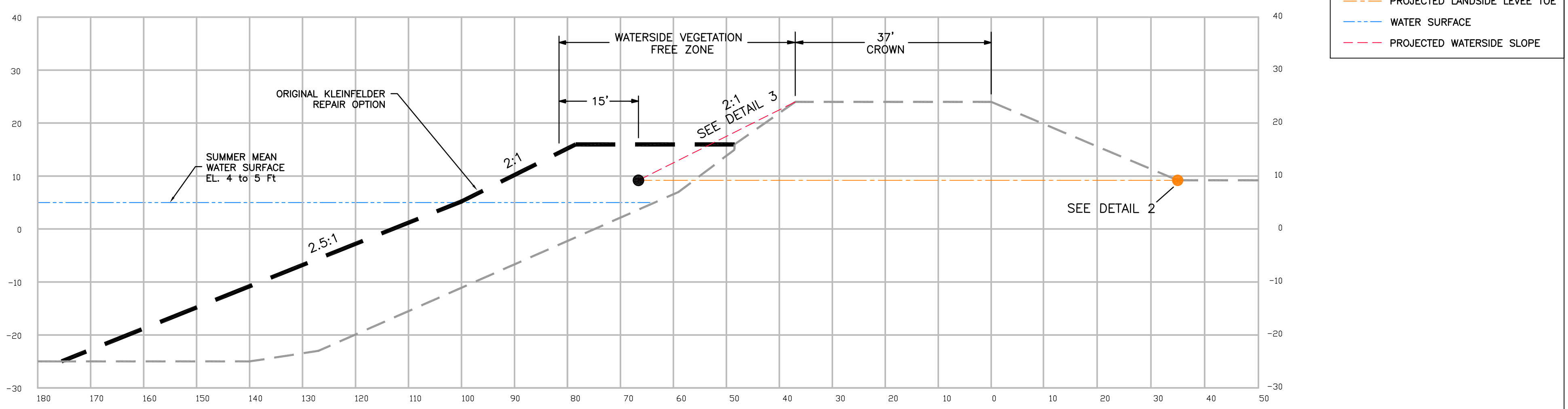


Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR HDR Engineering, Inc.	Drawn By: E. Smith Checked By: R. Little QA/QC Review: D. Teak	SACRAMENTO RIVER ~ 25.2L

Date	3/31/11
Figure	SAC 25.2L

DETAIL 1

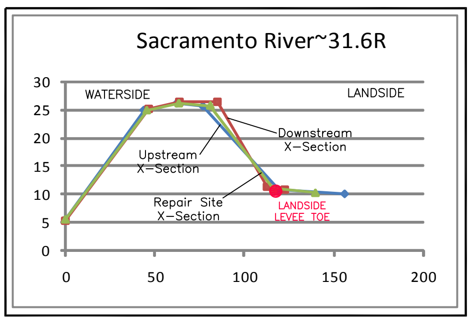
SACRAMENTO RIVER 31.6R



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	5.7
45.4	25.0
63.2	26.3
80.8	25.9
116.0	11.1
139.4	14.8

KEY DATA FOR DETERMINING LANDSIDE TOE

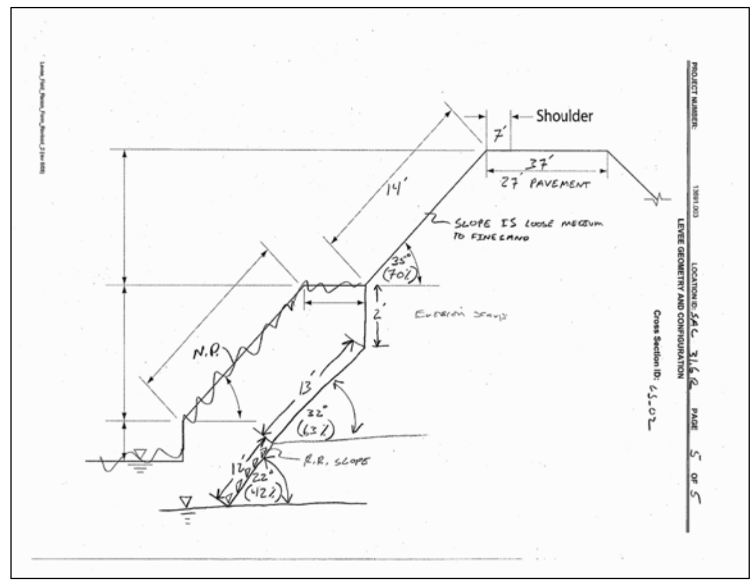
Comp Study Levee Crown Elevation	25.9
Comp Study Landside Toe Elevation	11.1
Comp Study Difference	14.8

Alternatives Report Analysis Levee Crown Elevation	24.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	9.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

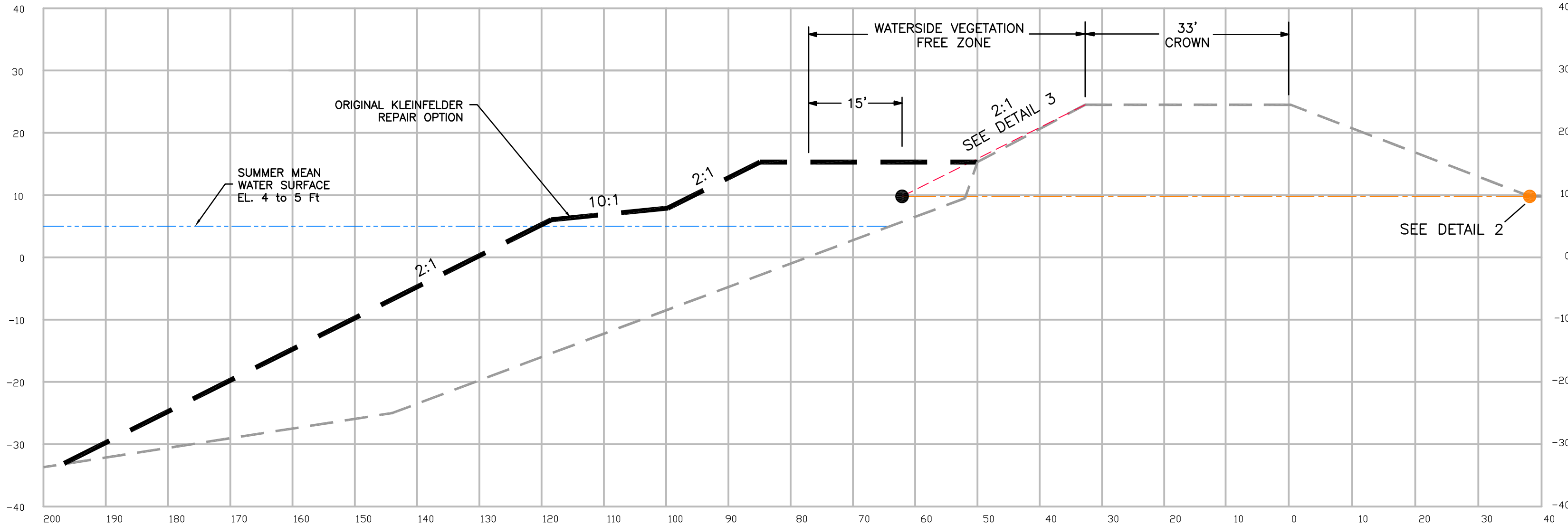
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 31.6R

Date	3/31/11
Figure	SAC 31.6R

DETAIL 1

SACRAMENTO RIVER 35.3R



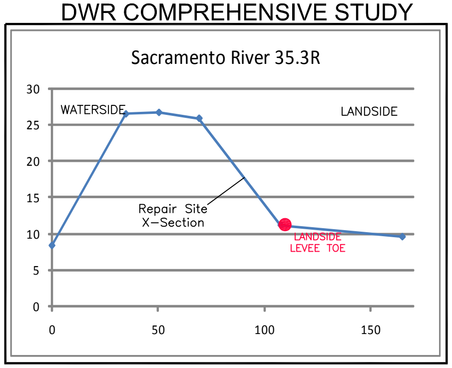
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	8.5
34.8	26.6
50.3	26.8
69.2	26.0
107.6	11.3
164.7	9.7

KEY DATA FOR DETERMINING LANDSIDE TOE

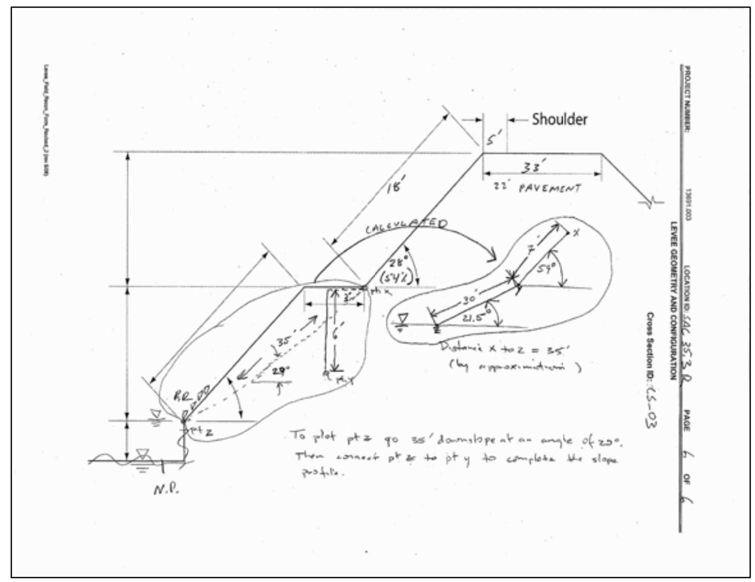
Comp Study Levee Crown Elevation	26.0
Comp Study Landside Toe Elevation	11.3
Comp Study Difference	14.7

Alternatives Report Analysis Levee Crown Elevation	24.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	9.8

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

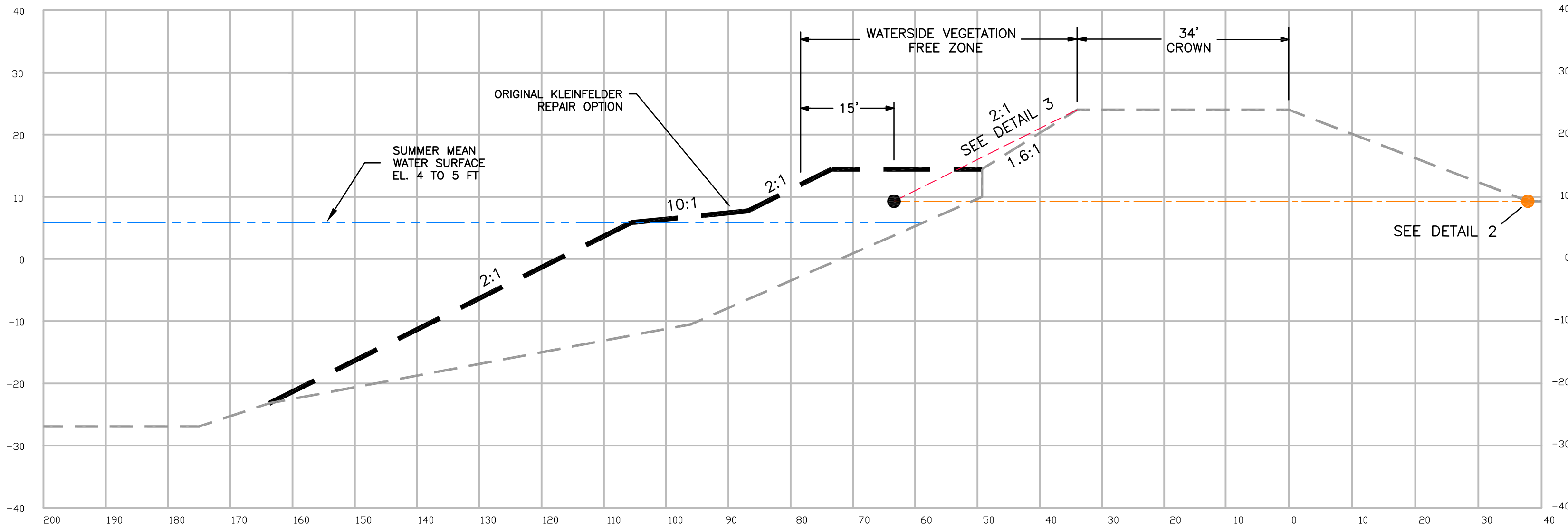
SACRAMENTO RIVER ~ 35.3R

Date 3/31/11

Figure SAC 35.3R

DETAIL 1

SACRAMENTO RIVER 35.4R



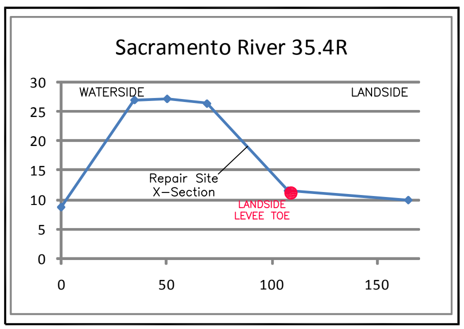
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	8.9
34.8	27.0
50.3	27.2
69.2	26.4
107.6	11.7
164.7	10.1

KEY DATA FOR DETERMINING LANDSIDE TOE

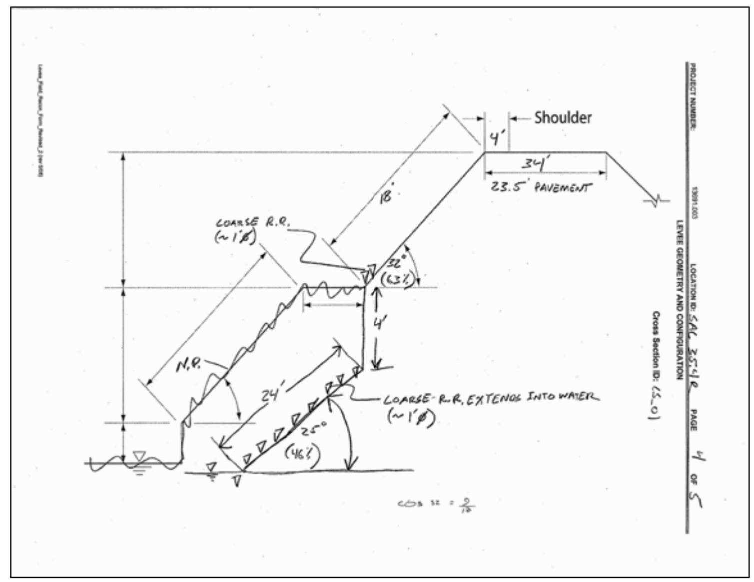
Comp Study Levee Crown Elevation	26.4
Comp Study Landside Toe Elevation	11.7
Comp Study Difference	14.7

Alternatives Report Analysis Levee Crown Elevation	24.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	9.3

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

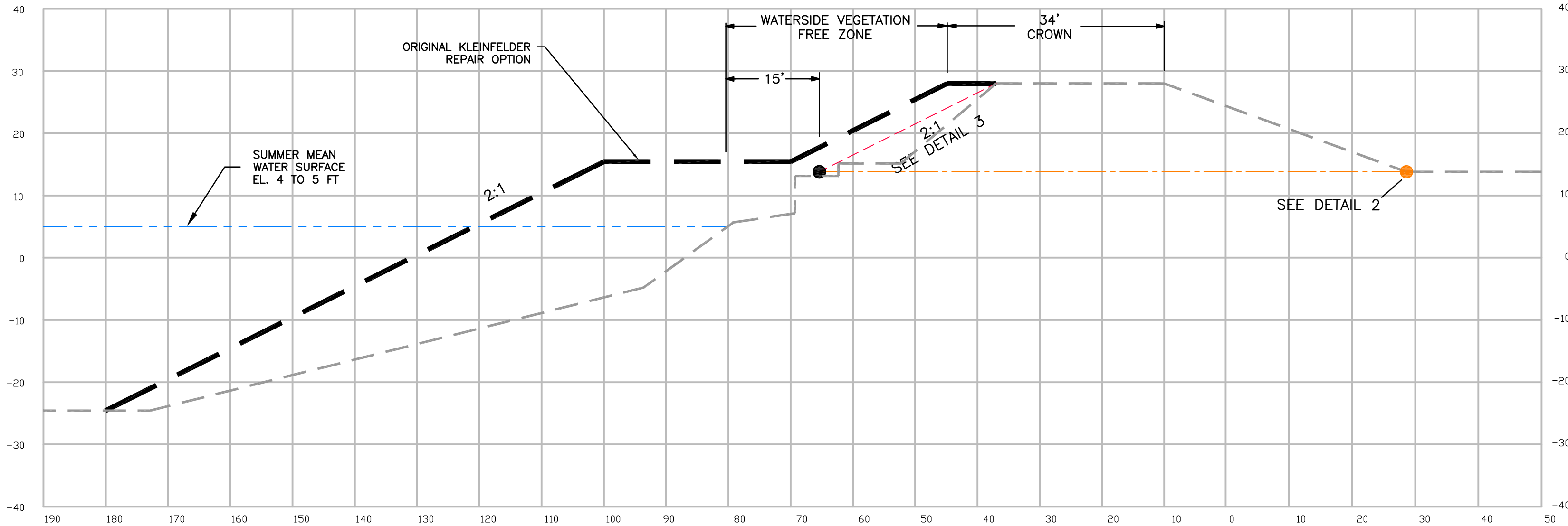
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 35.4R

Date	3/31/11
Figure	SAC 35.4R

DETAIL 1

SACRAMENTO RIVER 38.5R



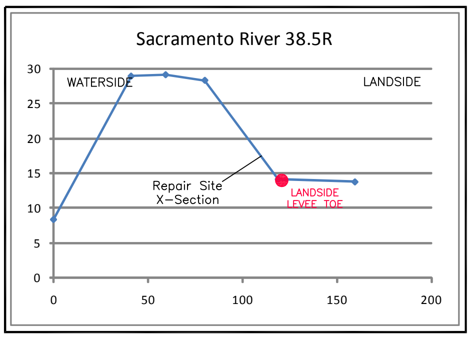
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	8.4
41.1	29.0
59.4	29.2
80.2	28.4
119.1	14.2
159.7	13.8

KEY DATA FOR DETERMINING LANDSIDE TOE

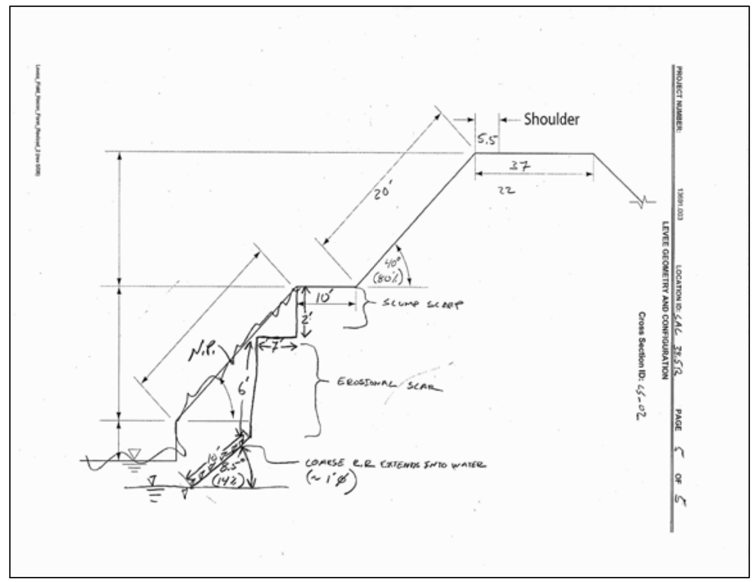
Comp Study Levee Crown Elevation	28.4
Comp Study Landside Toe Elevation	14.2
Comp Study Difference	14.2

Alternatives Report Analysis Levee Crown Elevation	28.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	13.8

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

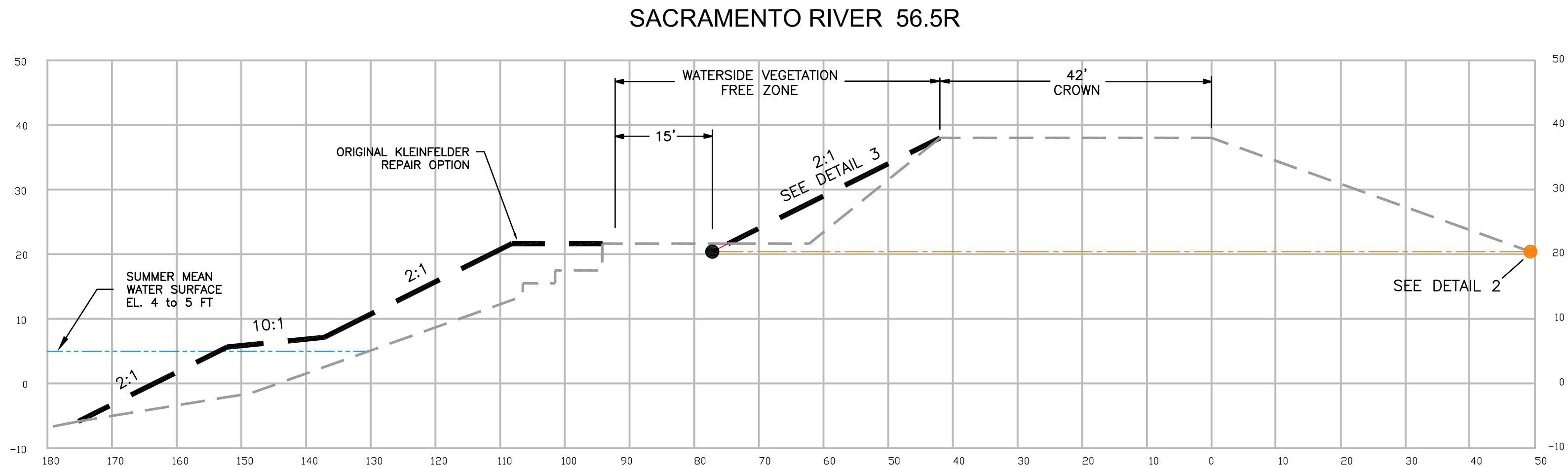
SACRAMENTO RIVER ~ 38.5R

Date	3/31/11
Figure	SAC 38.5R

DETAIL 1

LEGEND

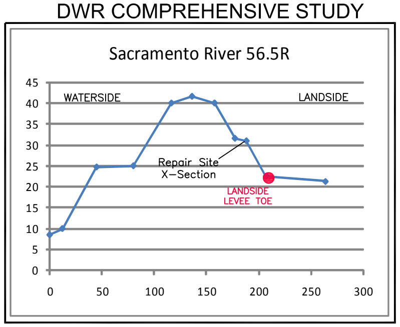
- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	8.5
12.1	10.0
44.9	24.8
80.1	25.1
116.5	40.2
136.2	41.8
157.7	40.2
177.2	31.7
188.2	31.1
207.0	22.6
263.8	21.3

KEY DATA FOR DETERMINING LANDSIDE TOE

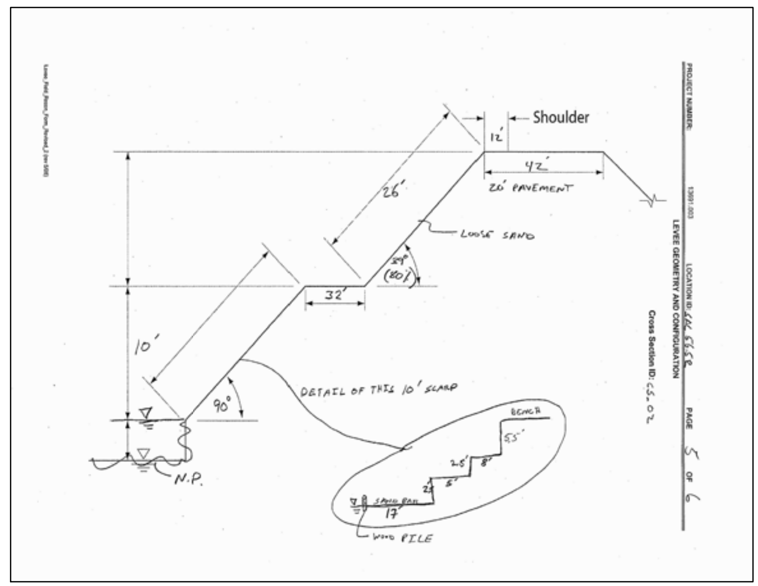
Comp Study Levee Crown Elevation	40.2
Comp Study Landside Toe Elevation	22.6
Comp Study Difference	17.6

Alternatives Report Analysis Levee Crown Elevation	38.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	20.4

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

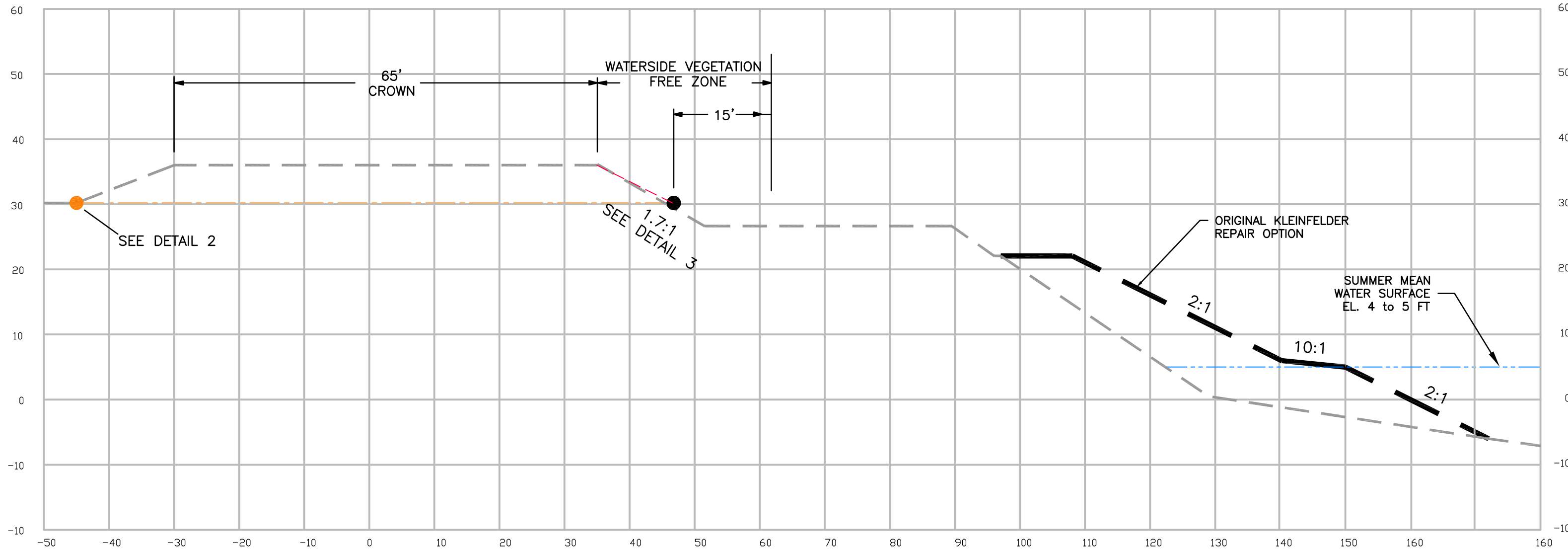
SACRAMENTO RIVER ~ 56.5R

Date 3/31/11

Figure SAC 56.5R

DETAIL 1

SACRAMENTO RIVER 56.6R



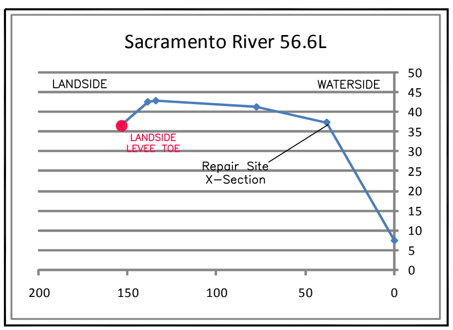
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	7.5
38.1	37.5
77.7	41.4
134.4	43.0
139.0	42.7
154.0	36.9

KEY DATA FOR DETERMINING LANDSIDE TOE

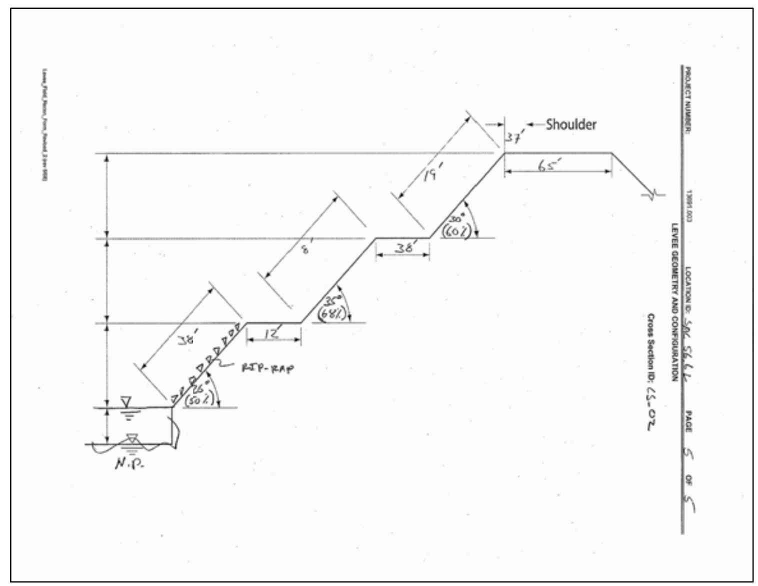
Comp Study Levee Crown Elevation	42.7
Comp Study Landside Toe Elevation	36.9
Comp Study Difference	5.8

Alternatives Report Analysis Levee Crown Elevation	36.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	30.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

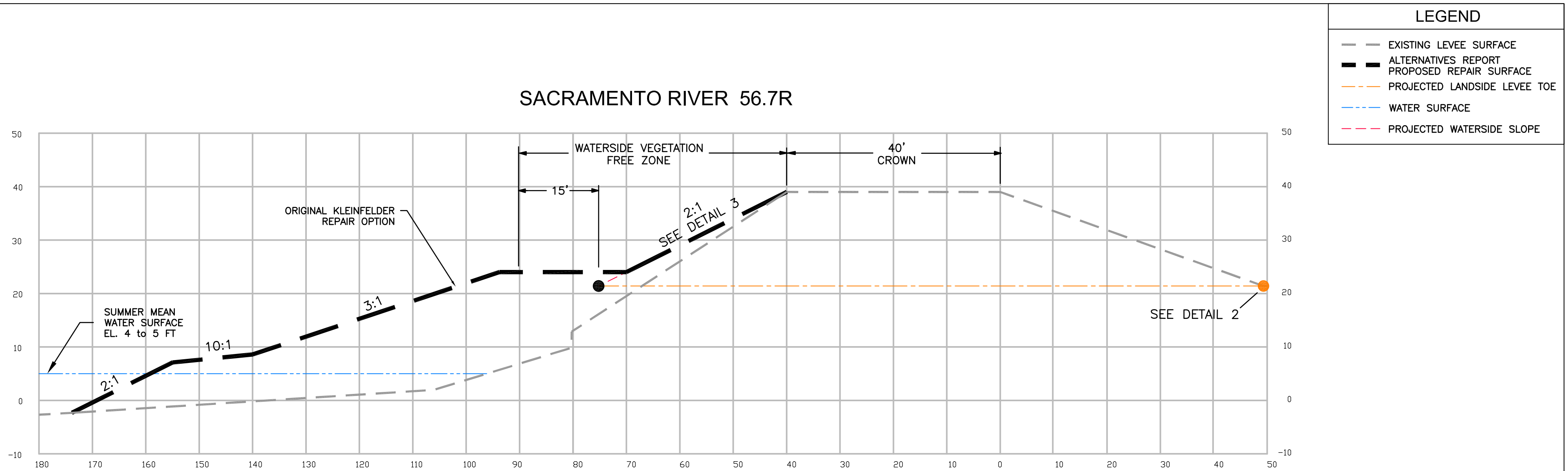
Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

SACRAMENTO RIVER ~ 56.6R

Date 3/31/11

Figure SAC 56.6R

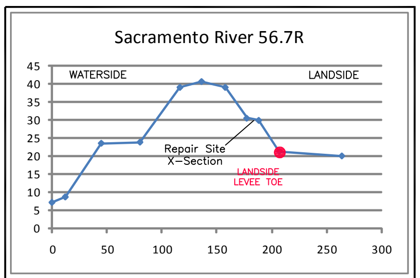
DETAIL 1



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	7.3
12.1	8.8
44.9	23.6
80.1	23.9
116.5	39.0
136.2	40.6
157.7	39.0
177.2	30.5
188.2	29.9
207.0	21.4
263.8	20.1

KEY DATA FOR DETERMINING LANDSIDE TOE

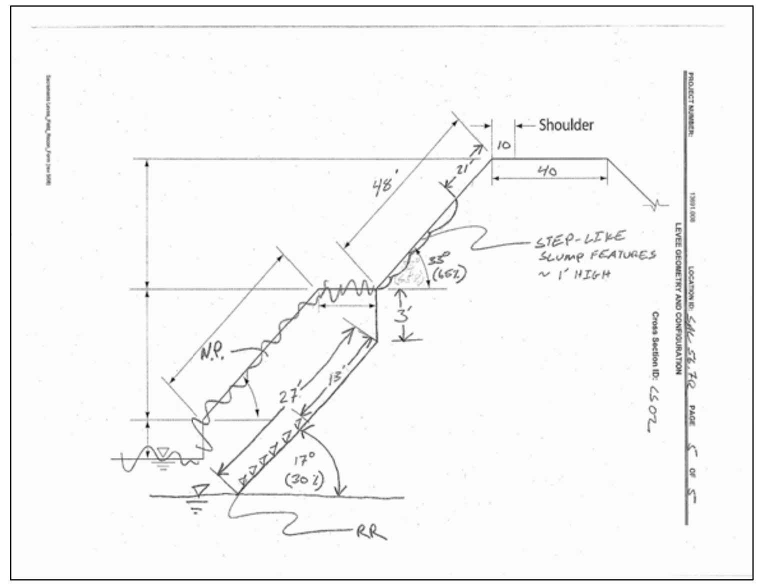
Comp Study Levee Crown Elevation	39.0
Comp Study Landside Toe Elevation	21.4
Comp Study Difference	17.6

Alternatives Report Analysis Levee Crown Elevation	39.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	21.4

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

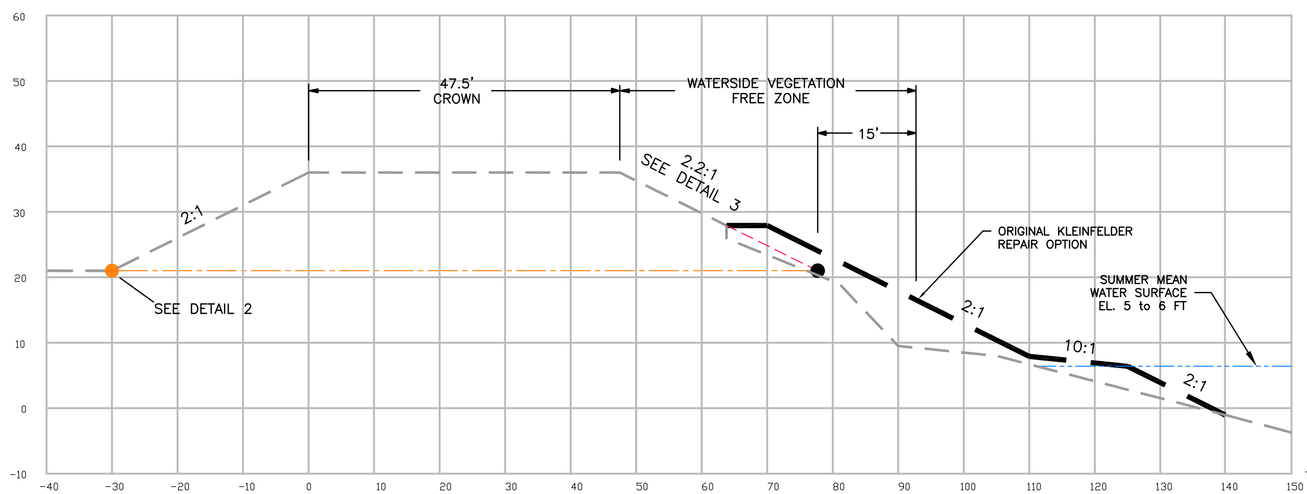
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 56.7R

Date	3/31/11
Figure	SAC 56.7R

DETAIL 1

SACRAMENTO RIVER 58.4L



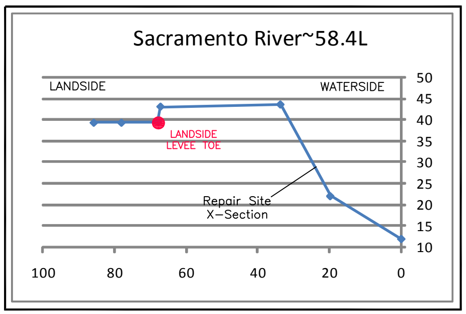
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	11.9
19.8	22.0
33.7	43.6
67.1	43.1
67.8	39.5
78.0	39.3
85.7	39.3

KEY DATA FOR DETERMINING LANDSIDE TOE

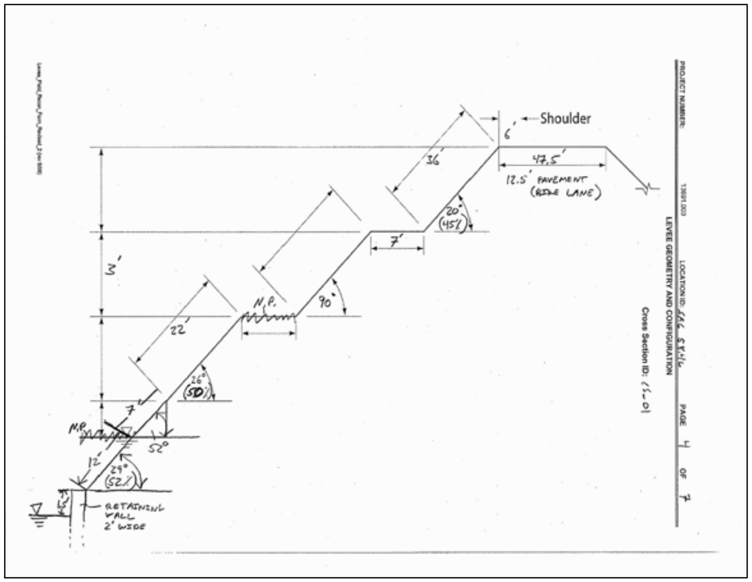
Comp Study Levee Crown Elevation	43.6
Comp Study Landside Toe Elevation	39.5
Comp Study Difference	4.1

Alternatives Report Analysis Levee Crown Elevation	36.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	31.9

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

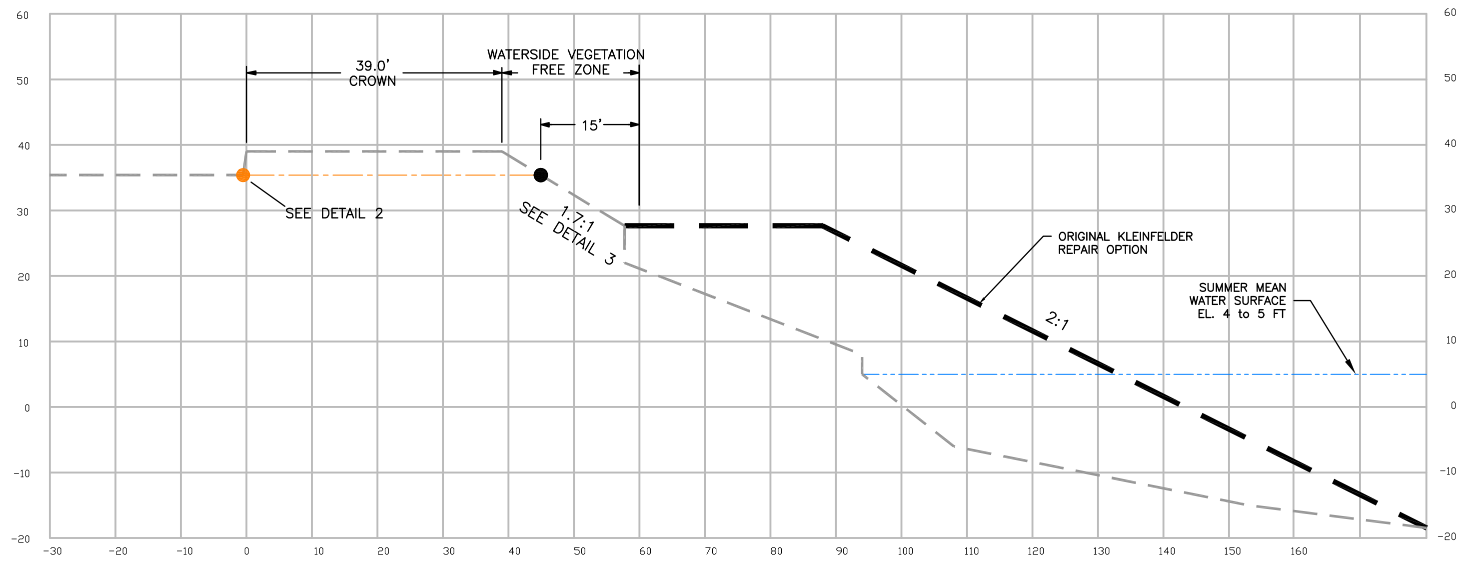
SACRAMENTO RIVER ~ 58.4L

Date 3/31/11

Figure SAC 58.4L

DETAIL 1

SACRAMENTO RIVER 60.1L



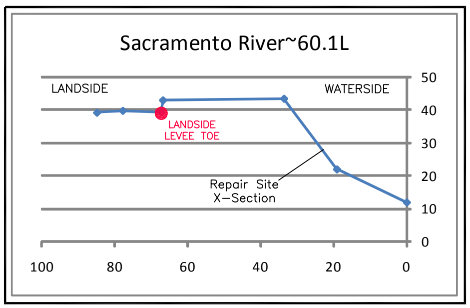
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	11.9
19.2	22.0
33.7	43.6
66.9	43.1
77.9	39.5
85.0	39.3

KEY DATA FOR DETERMINING LANDSIDE TOE

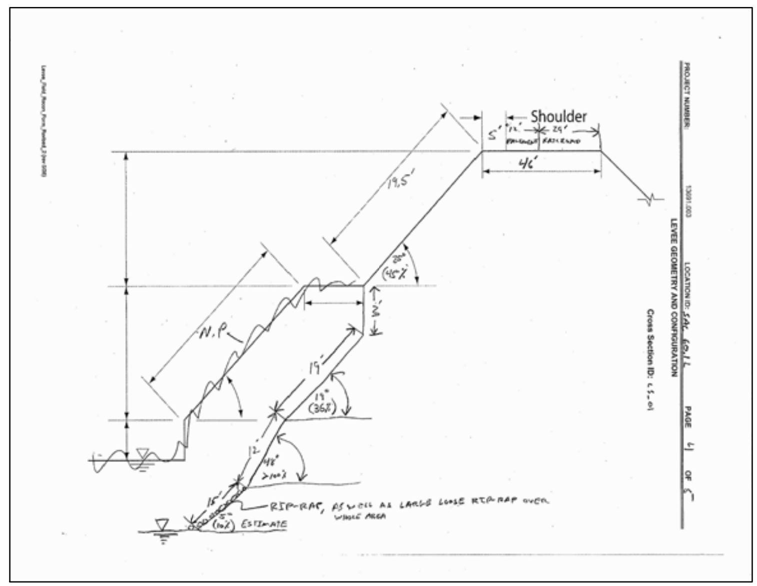
Comp Study Levee Crown Elevation	43.1
Comp Study Landside Toe Elevation	39.5
Comp Study Difference	3.6

Alternatives Report Analysis Levee Crown Elevation	39.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	35.4

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 60.1L

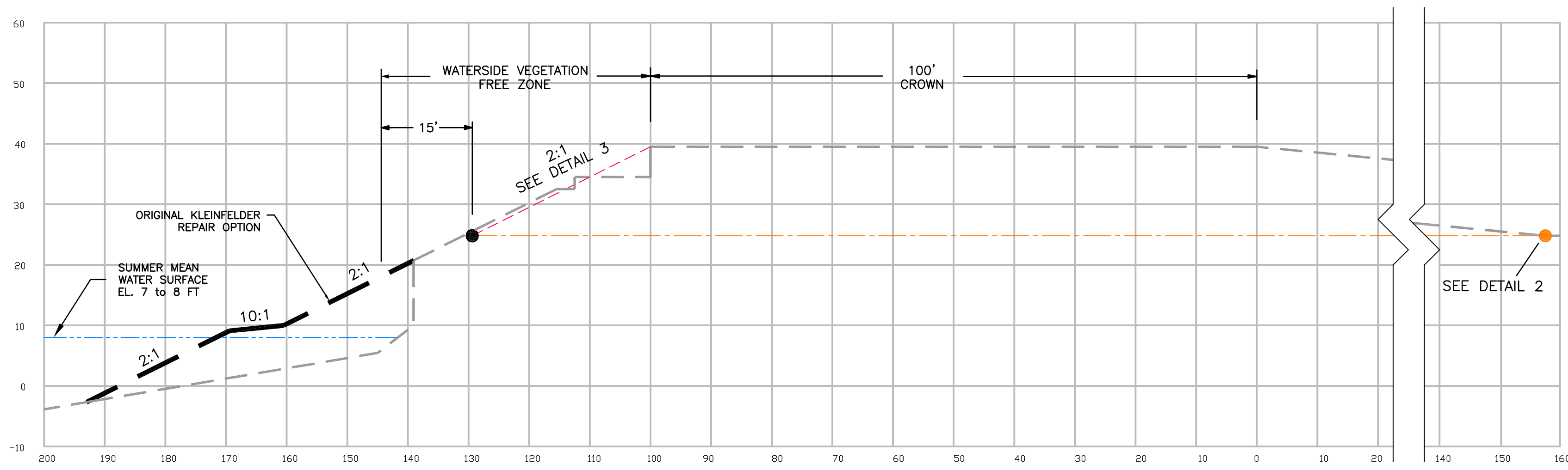
Date	3/31/11
Figure	SAC 60.1L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

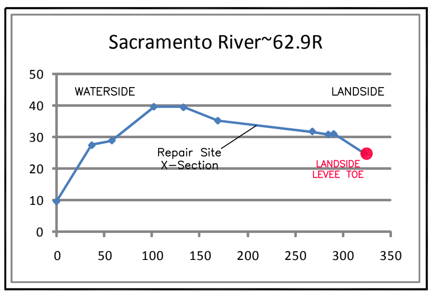
SACRAMENTO RIVER 62.9R



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	9.8
36.9	27.7
57.9	29.1
101.9	39.9
132.9	39.7
168.9	35.5
267.8	32.0
284.5	31.1
290.2	31.3
322.3	25.0

KEY DATA FOR DETERMINING LANDSIDE TOE

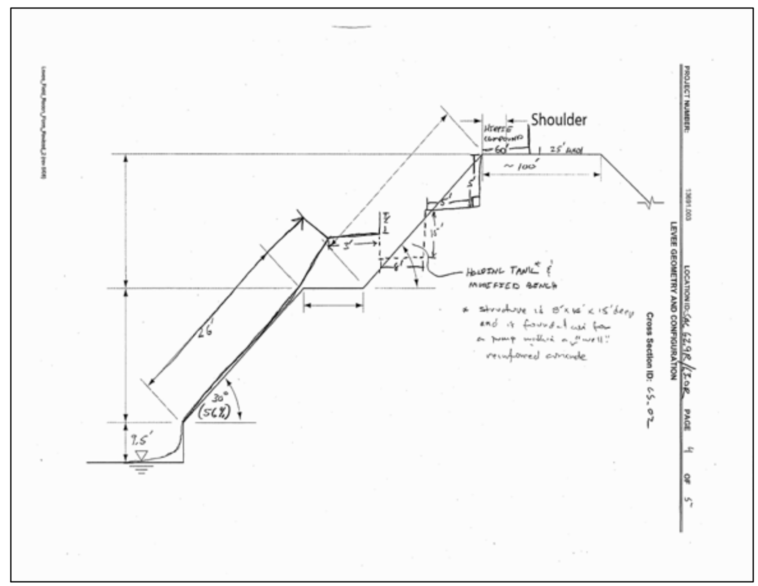
Comp Study Levee Crown Elevation	39.7
Comp Study Landside Toe Elevation	25.0
Comp Study Difference	14.7

Alternatives Report Analysis Levee Crown Elevation	39.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	24.8

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

SACRAMENTO RIVER ~ 62.9R

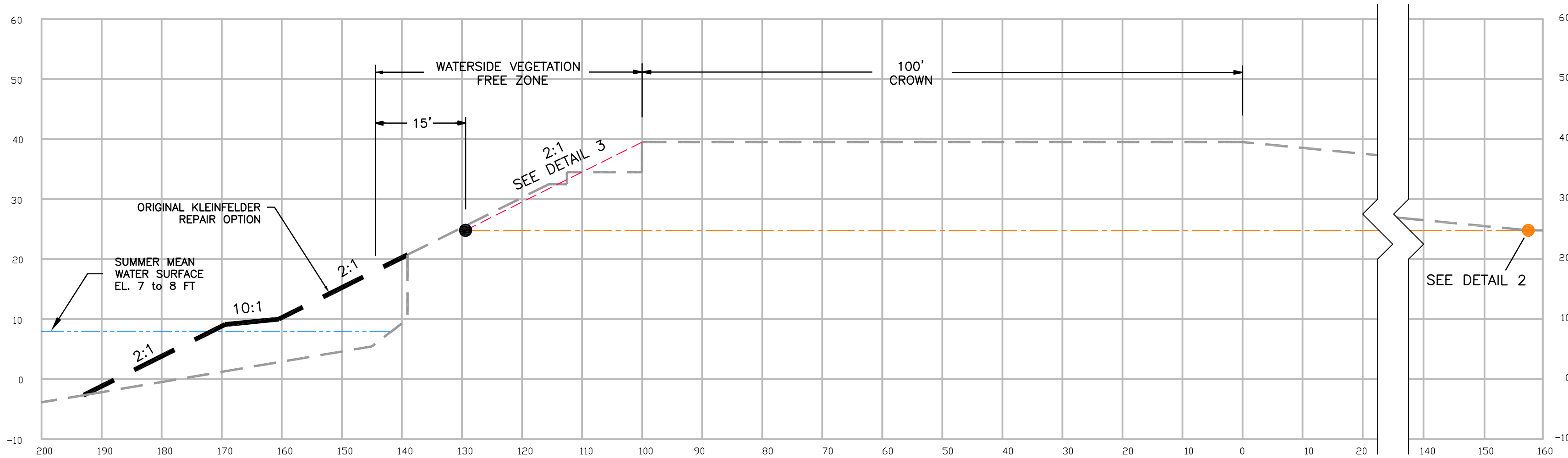
Date 3/31/11
Figure SAC 62.9R

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

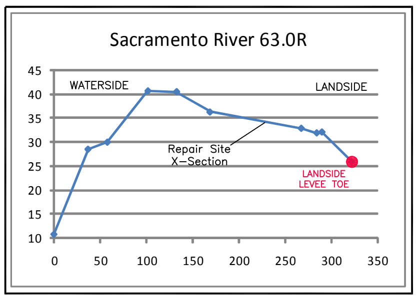
SACRAMENTO RIVER 63.0R



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	10.8
36.9	28.7
57.9	30.1
101.9	40.9
132.9	40.7
168.9	36.5
267.8	33.0
284.5	32.1
290.2	32.3
322.3	26.0

KEY DATA FOR DETERMINING LANDSIDE TOE

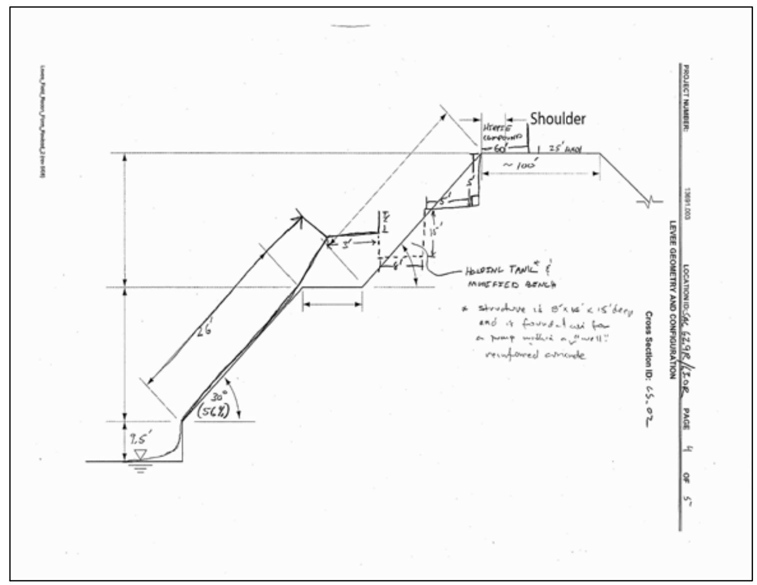
Comp Study Levee Crown Elevation	40.7
Comp Study Landside Toe Elevation	26.0
Comp Study Difference	14.7

Alternatives Report Analysis Levee Crown Elevation	39.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	24.8

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

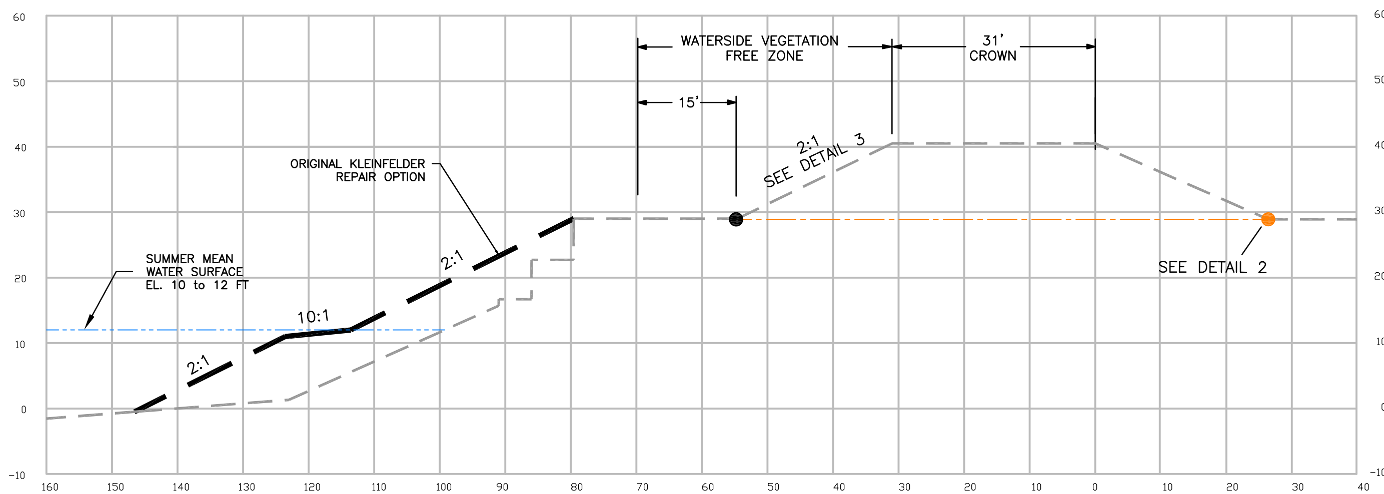
Drawn By: E. Smith	SACRAMENTO RIVER ~ 63.0R	Date	3/31/11
Checked By: R. Little		Figure	SAC 63.0R
QA/QC Review: D. Teak			

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

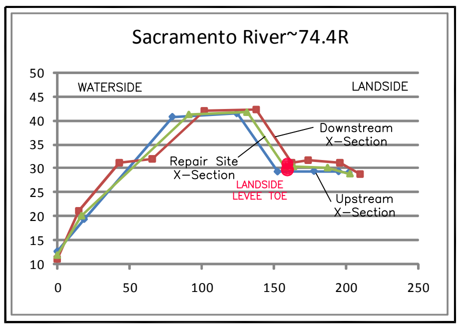
SACRAMENTO RIVER 74.4R



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	12.0
16.8	20.3
131.2	41.4
157.6	41.9
163.4	30.3
187.0	29.0
202.5	29.0

KEY DATA FOR DETERMINING LANDSIDE TOE

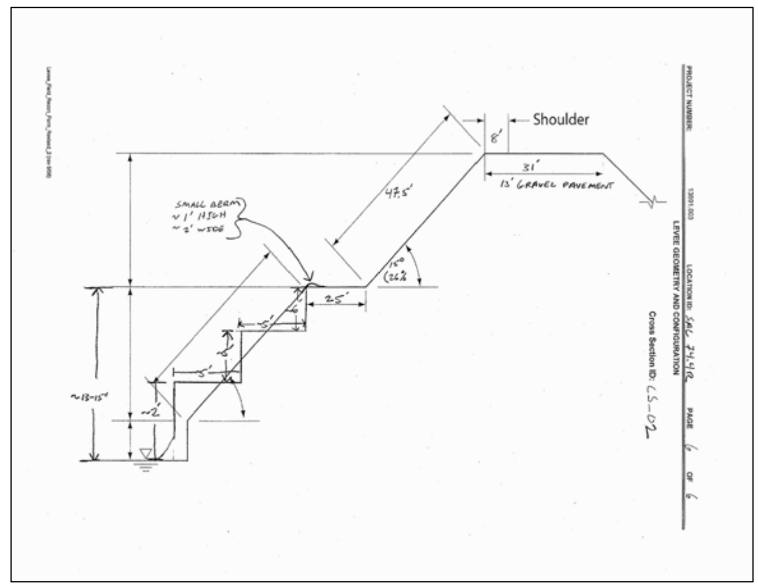
Comp Study Levee Crown Elevation	41.9
Comp Study Landside Toe Elevation	30.3
Comp Study Difference	11.6

Alternatives Report Analysis Levee Crown Elevation	40.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	28.9

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

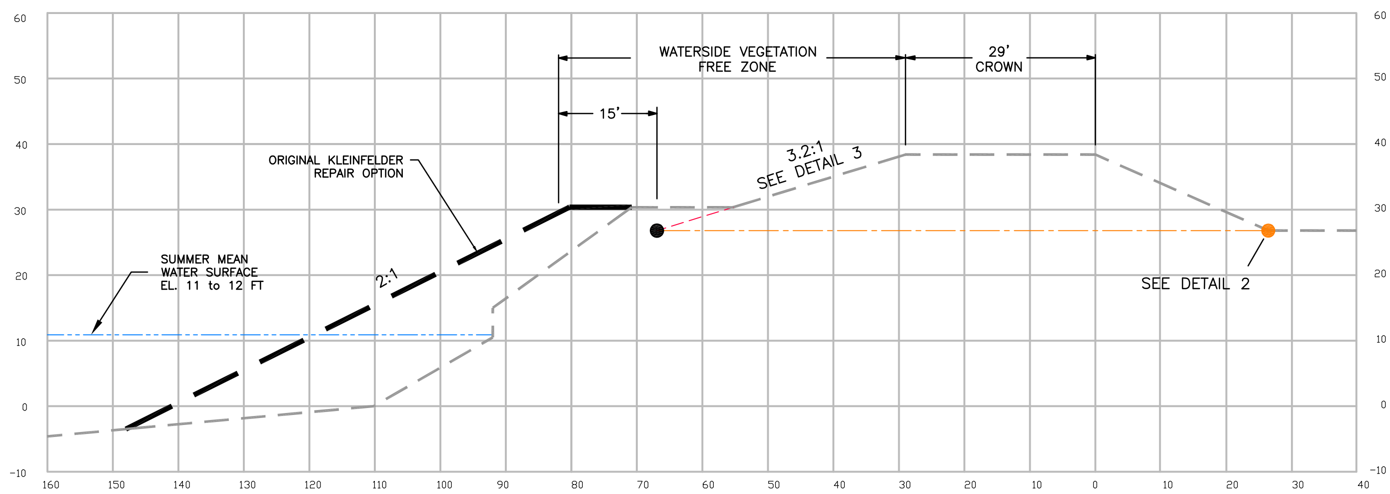
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 74.4R

Date	3/31/11
Figure	SAC 74.4R

DETAIL 1

SACRAMENTO RIVER 75.3R



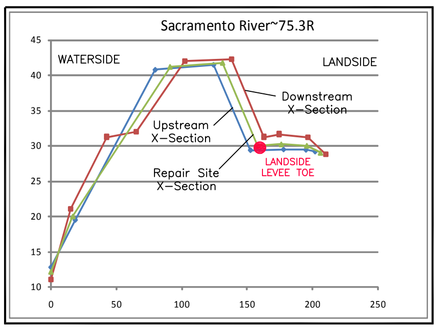
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	12.2
16.8	20.1
91.0	41.2
131.2	41.8
157.6	30.1
176.1	30.3
195.6	30.1
206.0	29.1

KEY DATA FOR DETERMINING LANDSIDE TOE

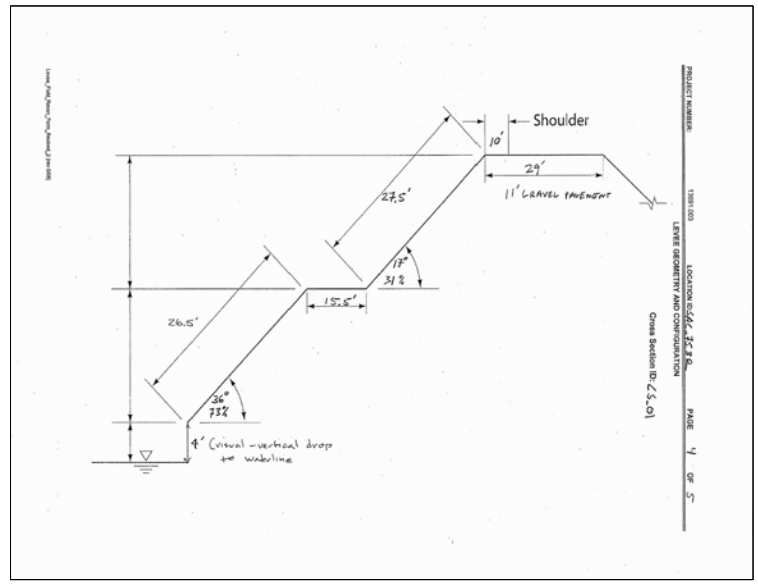
Comp Study Levee Crown Elevation	41.8
Comp Study Landside Toe Elevation	30.1
Comp Study Difference	11.7

Alternatives Report Analysis Levee Crown Elevation	38.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	26.8

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR

HDR Engineering, Inc.

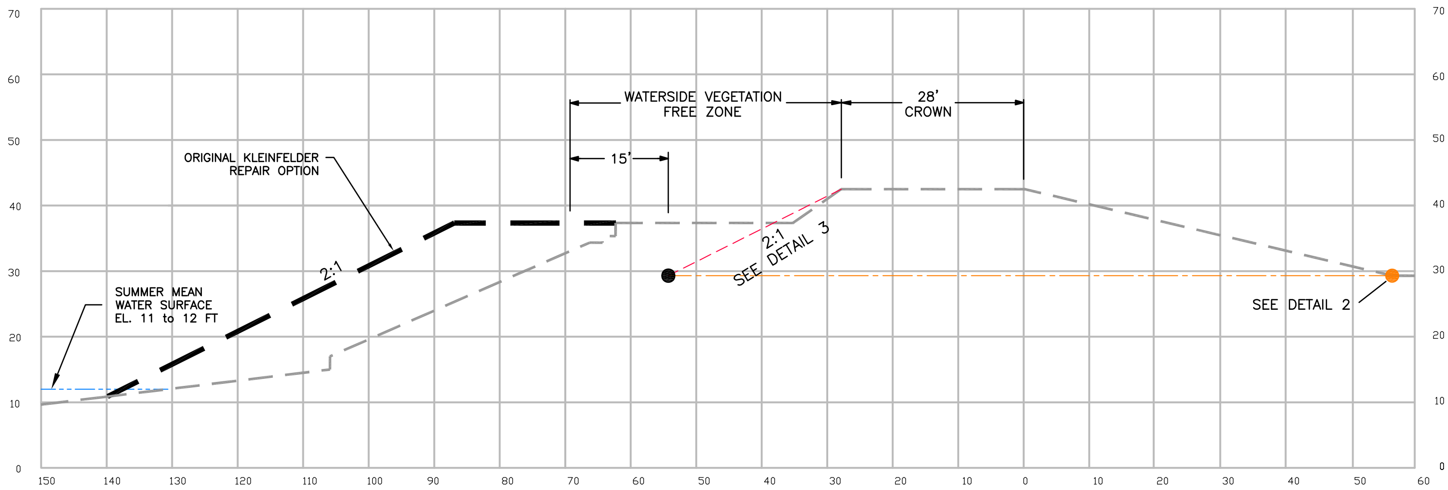
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 75.3R

Date	3/31/11
Figure	SAC 75.3R

DETAIL 1

SACRAMENTO RIVER 77.7R



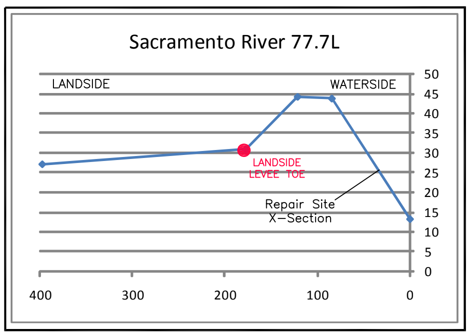
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	13.4
84.4	43.8
121.3	44.2
177.4	31.0
397.1	27.2

KEY DATA FOR DETERMINING LANDSIDE TOE

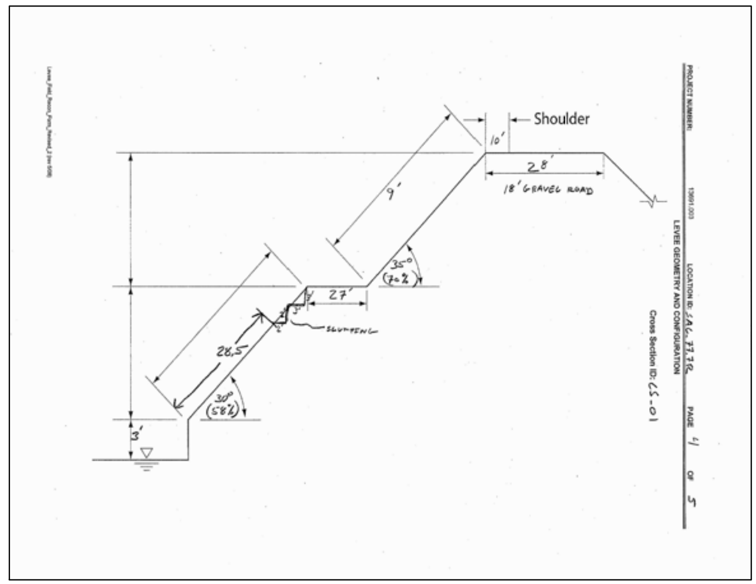
Comp Study Levee Crown Elevation	44.2
Comp Study Landside Toe Elevation	31.0
Comp Study Difference	13.2

Alternatives Report Analysis Levee Crown Elevation	42.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	29.3

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES

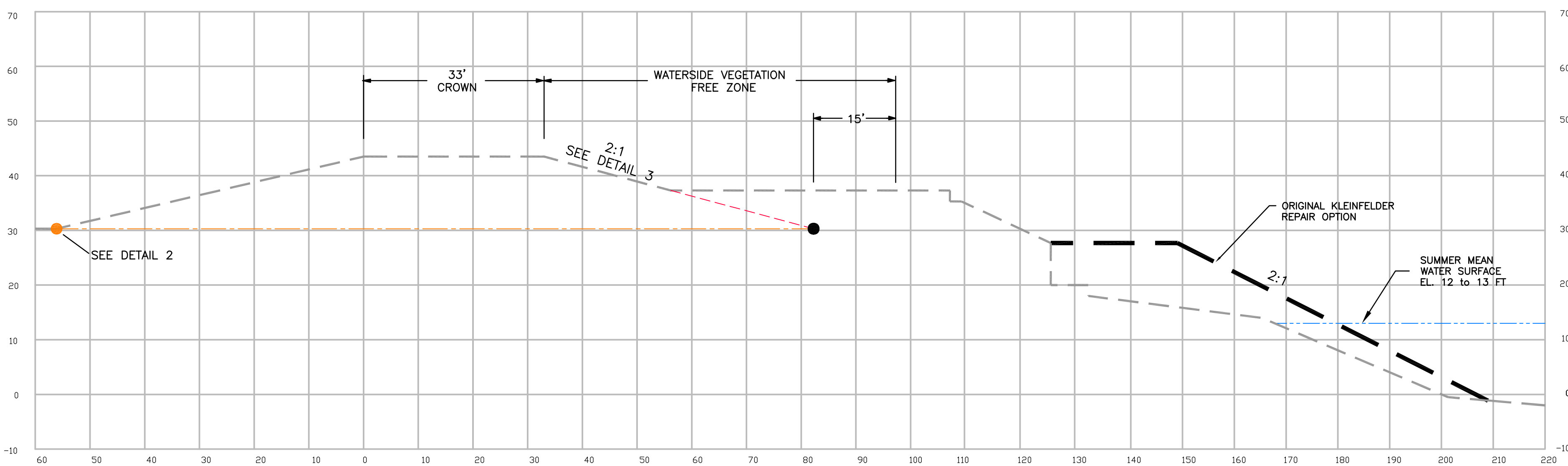


DETAIL 1

LEGEND

EXISTING LEVEE SURFACE	WATER SURFACE
ALTERNATIVES REPORT PROPOSED REPAIR SURFACE	PROJECTED WATERSIDE SLOPE
PROJECTED LANDSIDE LEVEE TOE	

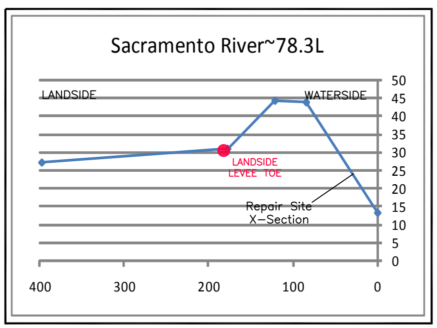
SACRAMENTO RIVER 78.3L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	13.5
84.4	43.9
121.3	44.4
177.4	31.1
397.1	27.4

KEY DATA FOR DETERMINING LANDSIDE TOE

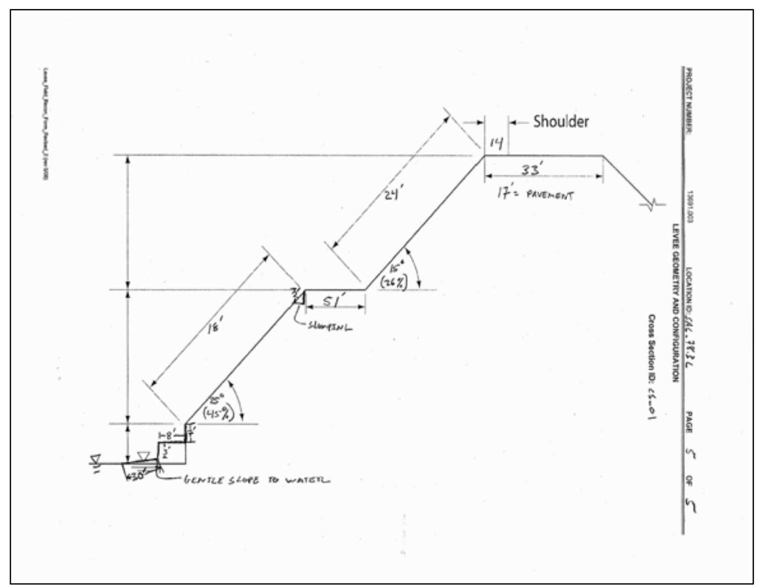
Comp Study Levee Crown Elevation	44.4
Comp Study Landside Toe Elevation	31.1
Comp Study Difference	13.2

Alternatives Report Analysis Levee Crown Elevation	43.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	30.3

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

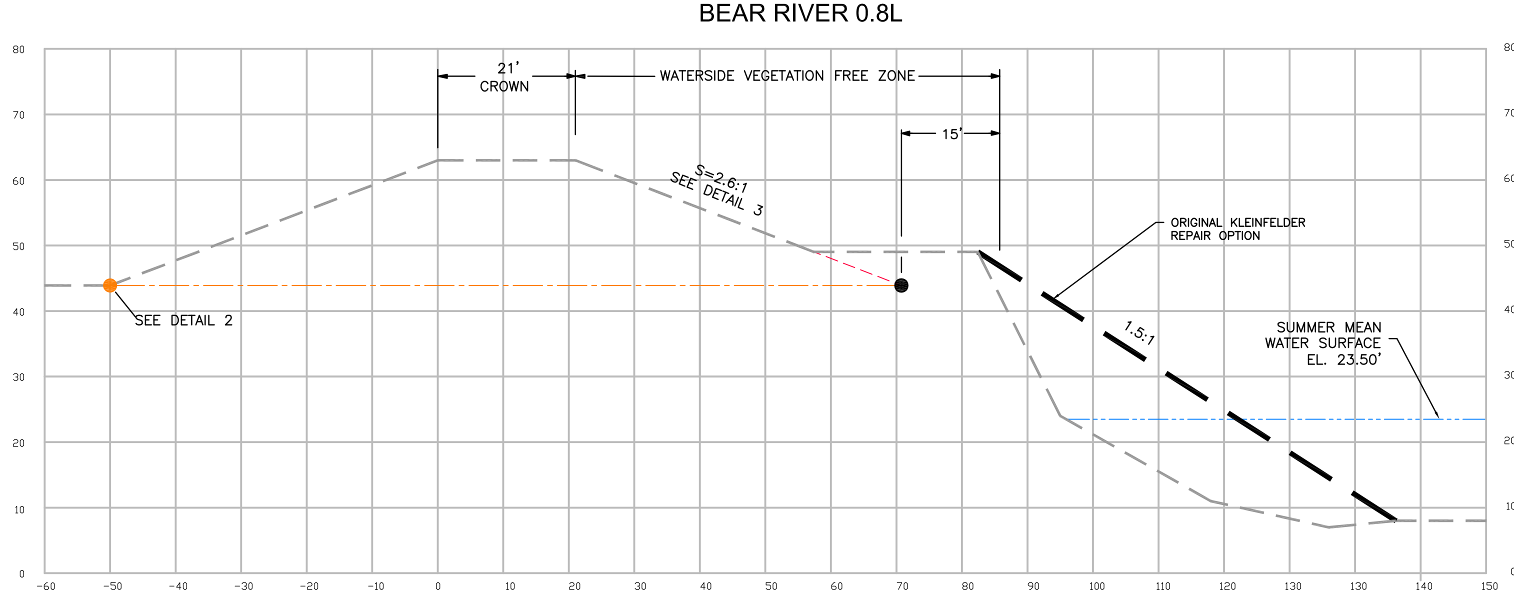
HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 78.3L

Date	3/31/11
Figure	SAC 78.3L

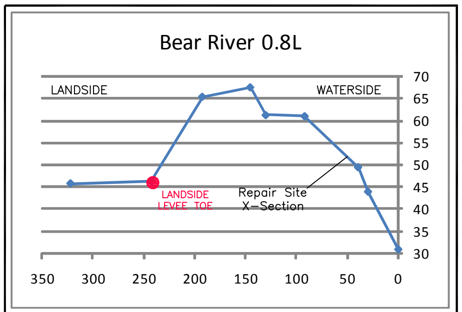
DETAIL 1



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	31.1
29.7	44.1
39.4	49.7
91.8	61.2
130.0	61.4
145.2	67.7
192.3	65.6
242.3	46.5
321.6	46.0

KEY DATA FOR DETERMINING LANDSIDE TOE

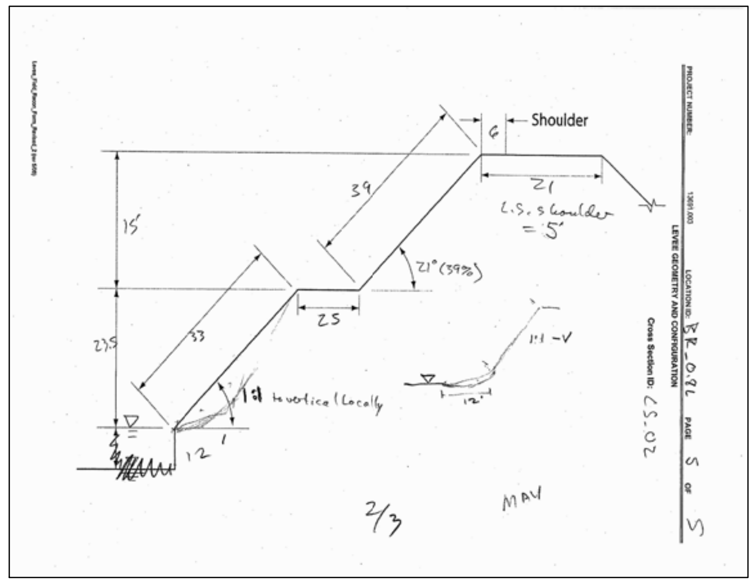
Comp Study Levee Crown Elevation	65.6
Comp Study Landside Toe Elevation	46.5
Comp Study Difference	19.1

Alternatives Report Analysis Levee Crown Elevation	63.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	43.9

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR Engineering, Inc.

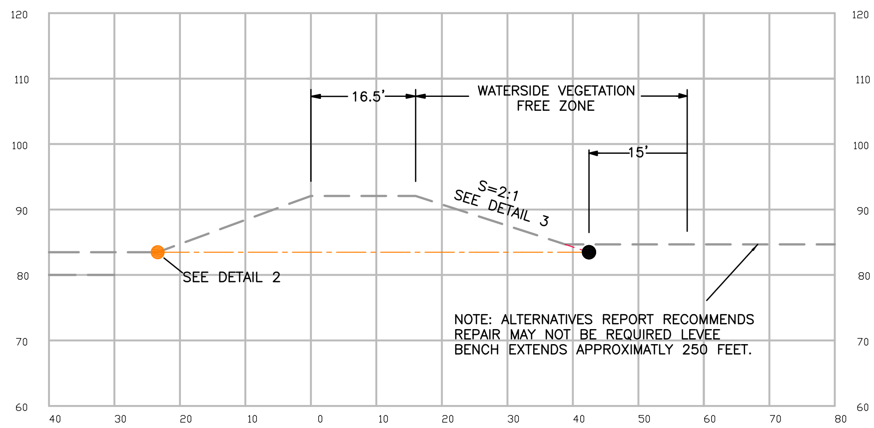
Drawn By: E. Smith	BEAR RIVER ~ 0.8L	Date	3/31/11
Checked By: R. Little		Figure	BR 0.8L
QA/QC Review: D. Teak			

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

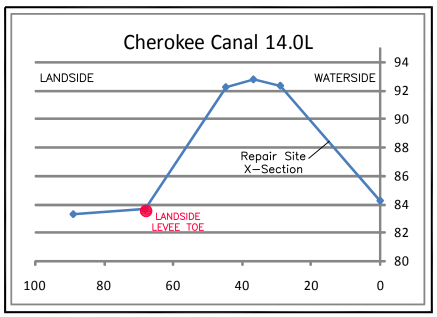
CHEROKEE CANAL 14.0L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	84.3
29.0	92.3
36.8	92.8
44.8	92.2
68.2	83.6
89.0	83.3

KEY DATA FOR DETERMINING LANDSIDE TOE

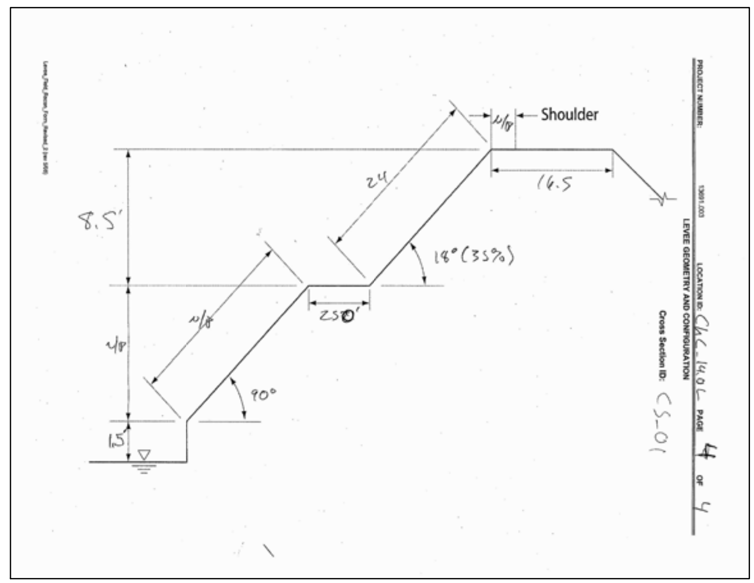
Comp Study Levee Crown Elevation	92.2
Comp Study Landside Toe Elevation	83.6
Comp Study Difference	8.6

Alternatives Report Analysis Levee Crown Elevation	95.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	86.4

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

CHEROKEE CANAL ~ 14.0L

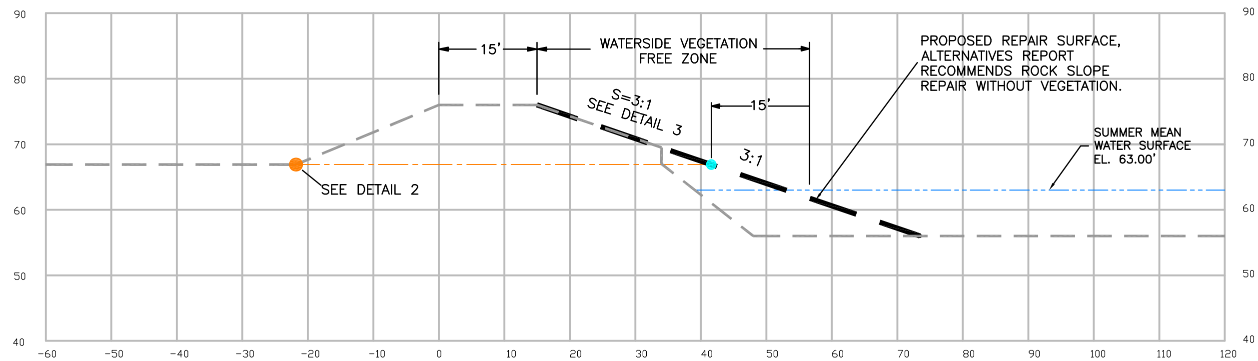
Date	3/31/11
Figure	ChC 14.0L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

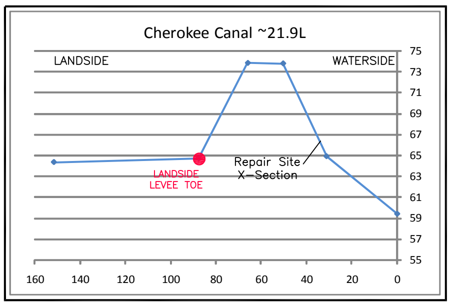
CHEROKEE CANAL 21.9L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	59.5
31.2	65.0
50.3	73.8
65.9	73.9
87.7	64.8
151.5	64.4

KEY DATA FOR DETERMINING LANDSIDE TOE

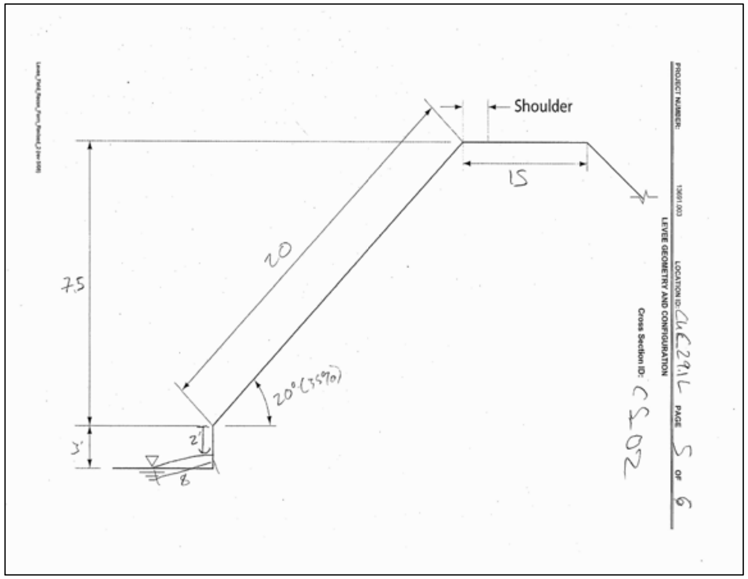
Comp Study Levee Crown Elevation	73.8
Comp Study Landside Toe Elevation	64.8
Comp Study Difference	9.1

Alternatives Report Analysis Levee Crown Elevation	76.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	66.9

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

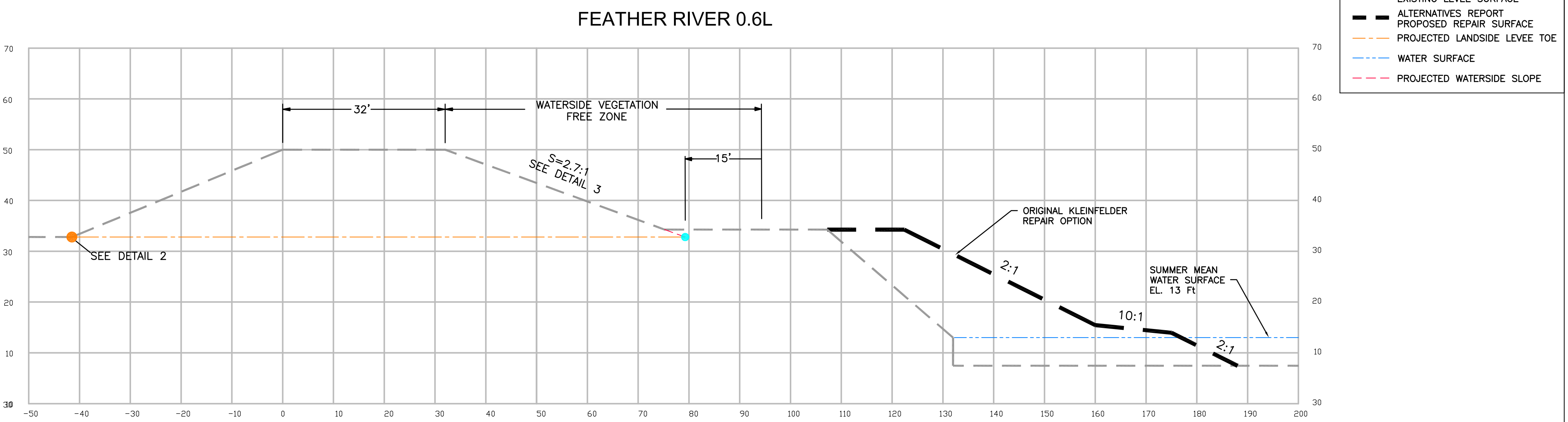
Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

CHEROKEE CANAL ~ 21.9L

Date 3/31/11

Figure ChC 21.9L

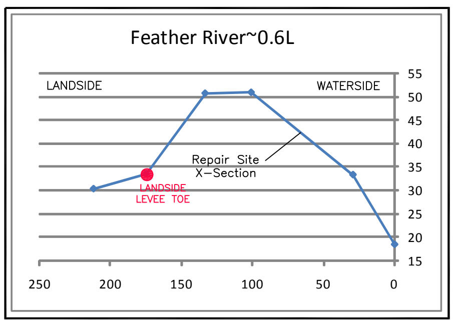
DETAIL 1



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	18.6
29.4	33.5
101.2	51.1
133.7	50.9
175.2	33.7
212.4	30.5

KEY DATA FOR DETERMINING LANDSIDE TOE

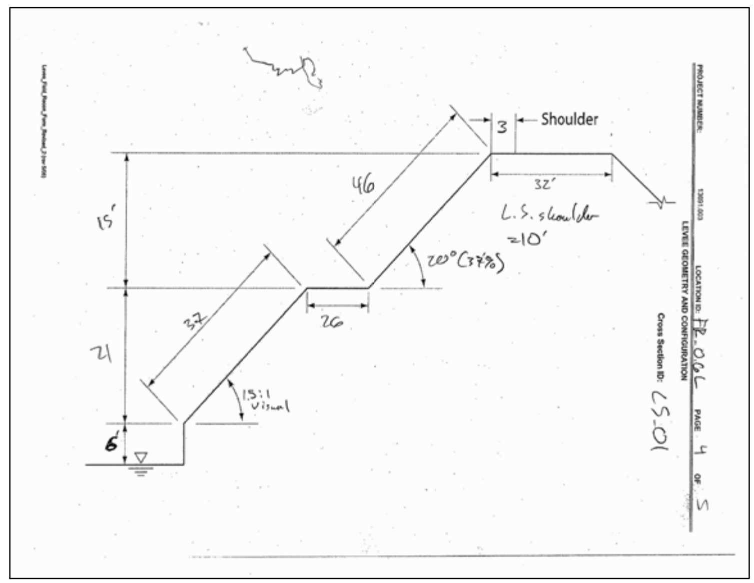
Comp Study Levee Crown Elevation	50.9
Comp Study Landside Toe Elevation	33.7
Comp Study Difference	17.2

Alternatives Report Analysis Levee Crown Elevation	50.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	32.8

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

FEATHER RIVER ~ 0.6L

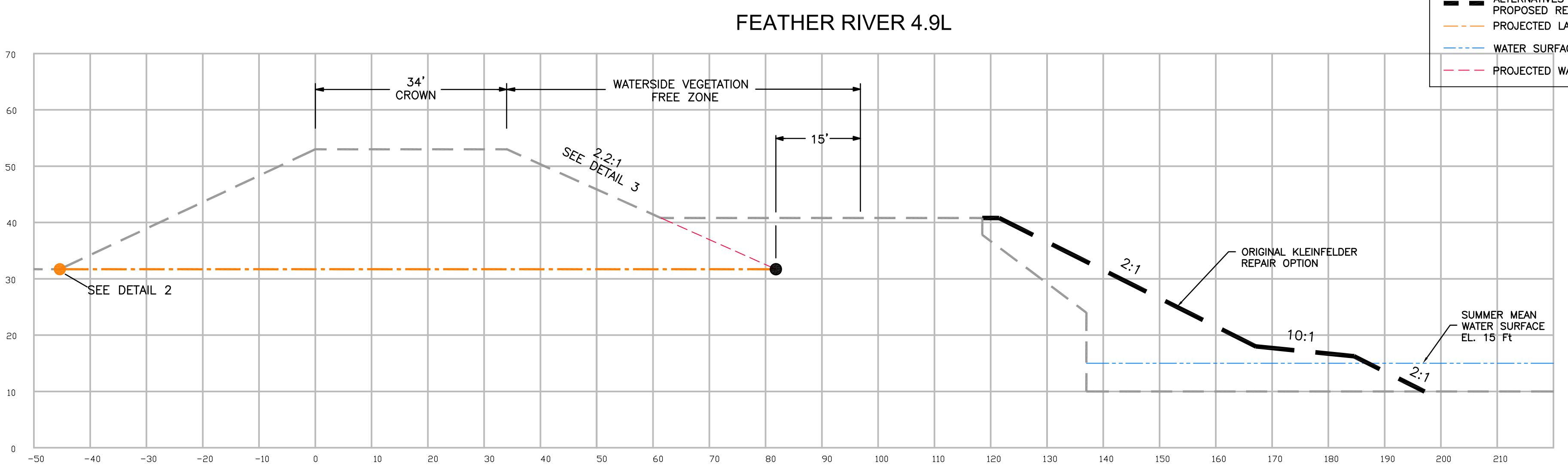
Date	3/31/11
Figure	FR 0.6L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

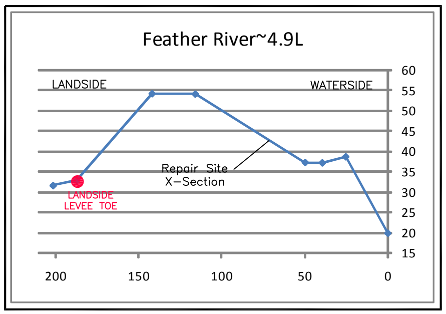
FEATHER RIVER 4.9L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



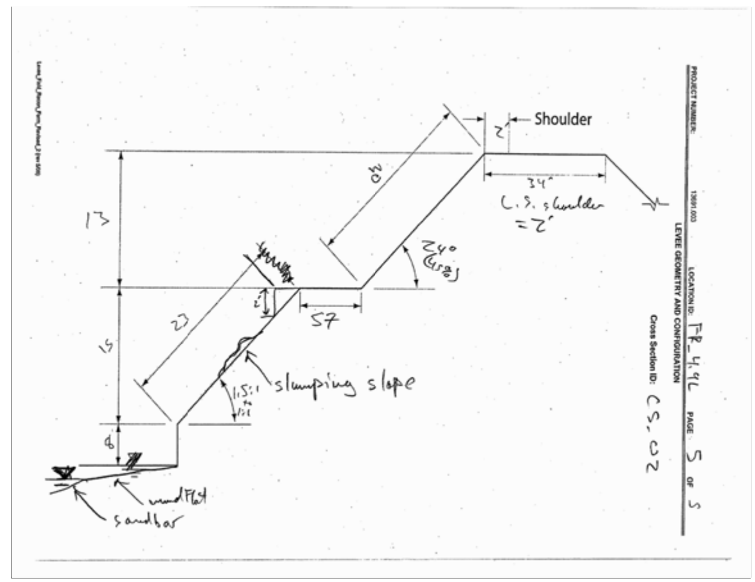
Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	20.0
25.4	38.8
39.5	37.3
49.8	37.4
116.0	54.2
142.0	54.3
187.4	33.0
201.6	31.7

KEY DATA FOR DETERMINING LANDSIDE TOE

Comp Study Levee Crown Elevation	54.3
Comp Study Landside Toe Elevation	33.0
Comp Study Difference	21.3
Alternatives Report Analysis Levee Crown Elevation	53.0
Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	31.7

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR
HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

FEATHER RIVER ~ 4.9L

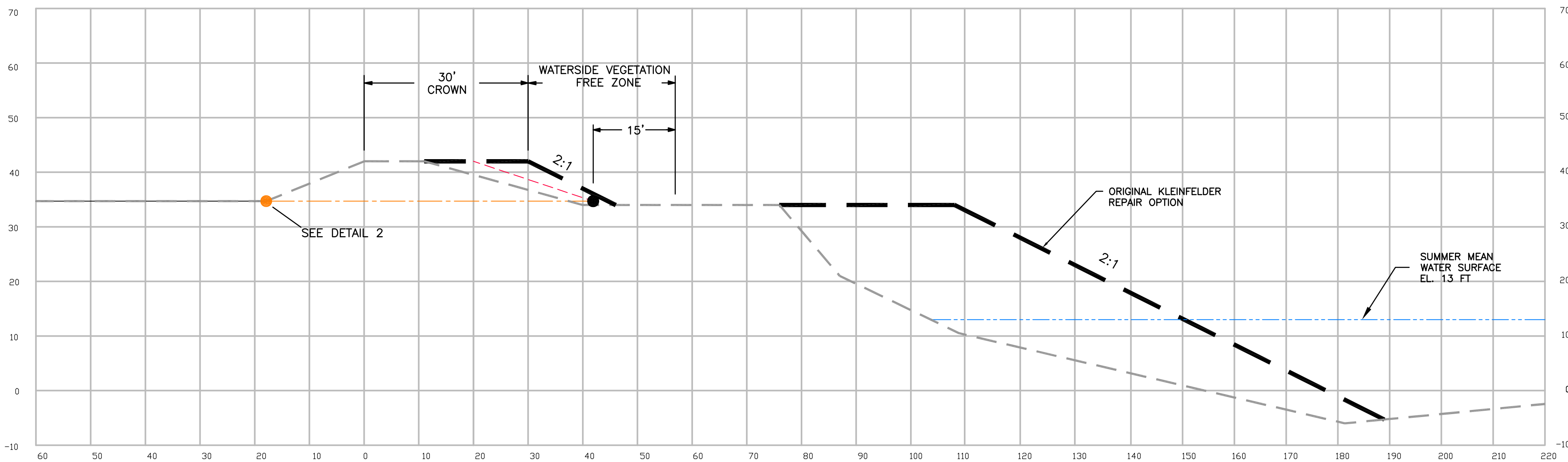
Date	3/31/11
Figure	FR 4.9L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

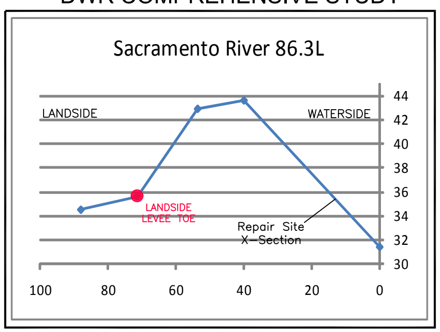
SACRAMENTO RIVER 86.3L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	31.5
40.0	43.7
53.6	43.0
71.5	35.7
88.0	34.6

KEY DATA FOR DETERMINING LANDSIDE TOE

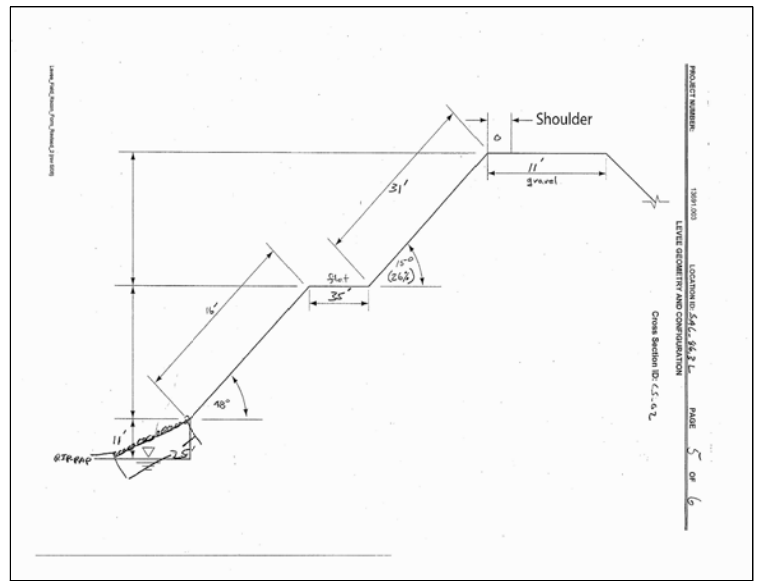
Comp Study Levee Crown Elevation	43.0
Comp Study Landside Toe Elevation	35.7
Comp Study Difference	7.3

Alternatives Report Analysis Levee Crown Elevation	42.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	34.7

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 86.3L

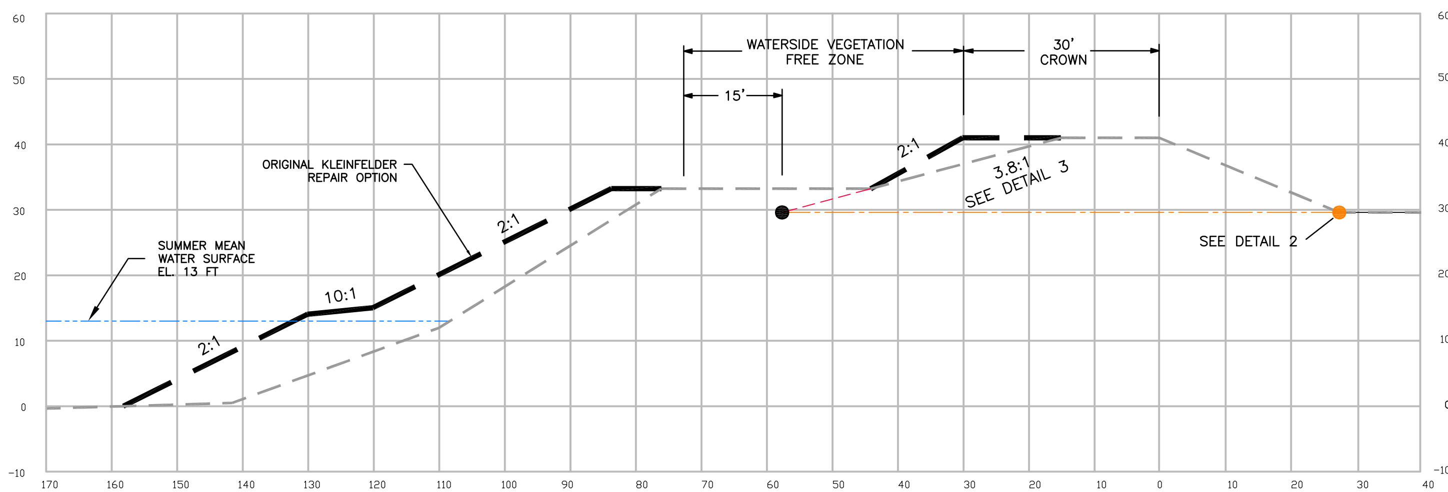
Date	3/31/11
Figure	SAC 86.3L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

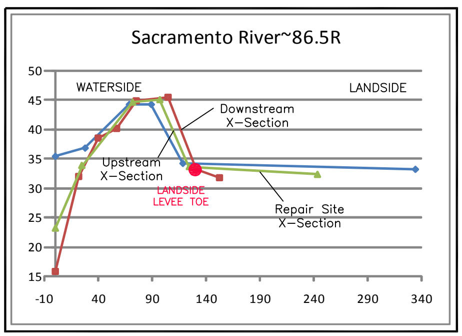
SACRAMENTO RIVER 86.5R



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	23.2
24.6	33.9
71.9	44.6
97.1	45.1
124.6	33.7
243.7	32.4

KEY DATA FOR DETERMINING LANDSIDE TOE

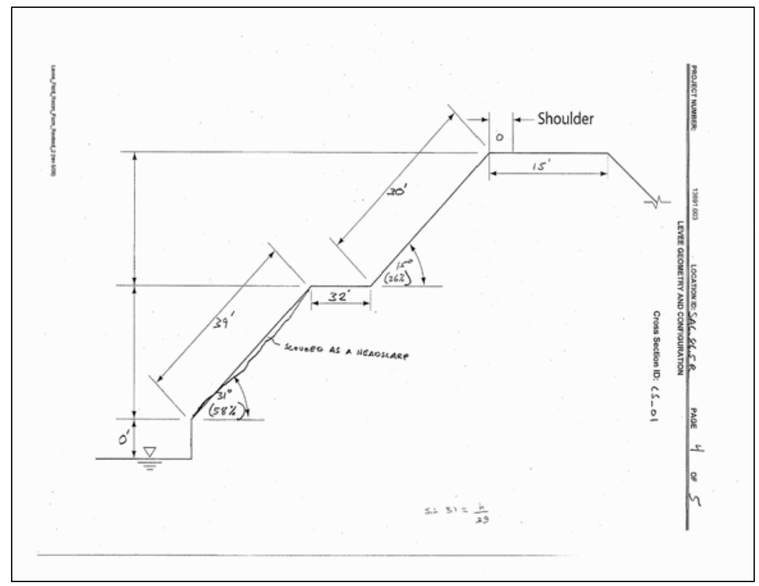
Comp Study Levee Crown Elevation	45.1
Comp Study Landside Toe Elevation	33.7
Comp Study Difference	11.4

Alternatives Report Analysis Levee Crown Elevation	41.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	29.6

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR Engineering, Inc.	Drawn By: E. Smith Checked By: R. Little QA/QC Review: D. Teak	SACRAMENTO RIVER ~ 86.5R

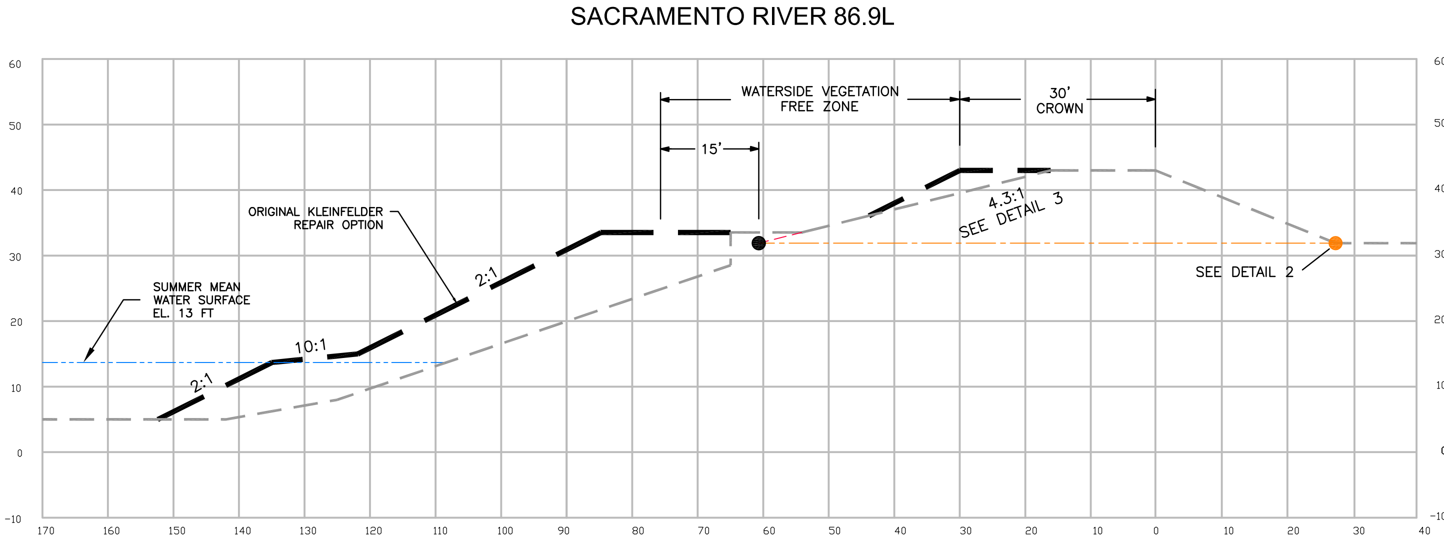
Date 3/31/11

Figure SAC 86.5R

DETAIL 1

LEGEND

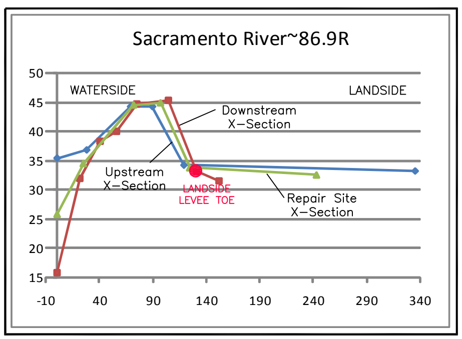
- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	25.8
24.6	34.5
71.9	44.5
97.1	44.9
124.6	33.8
243.7	32.6

KEY DATA FOR DETERMINING LANDSIDE TOE

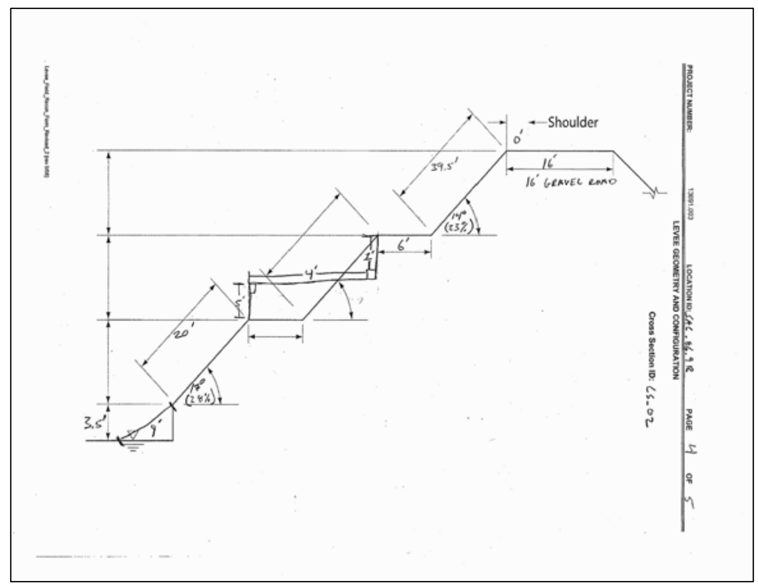
Comp Study Levee Crown Elevation	44.9
Comp Study Landside Toe Elevation	33.8
Comp Study Difference	11.1

Alternatives Report Analysis Levee Crown Elevation	43.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	31.9

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

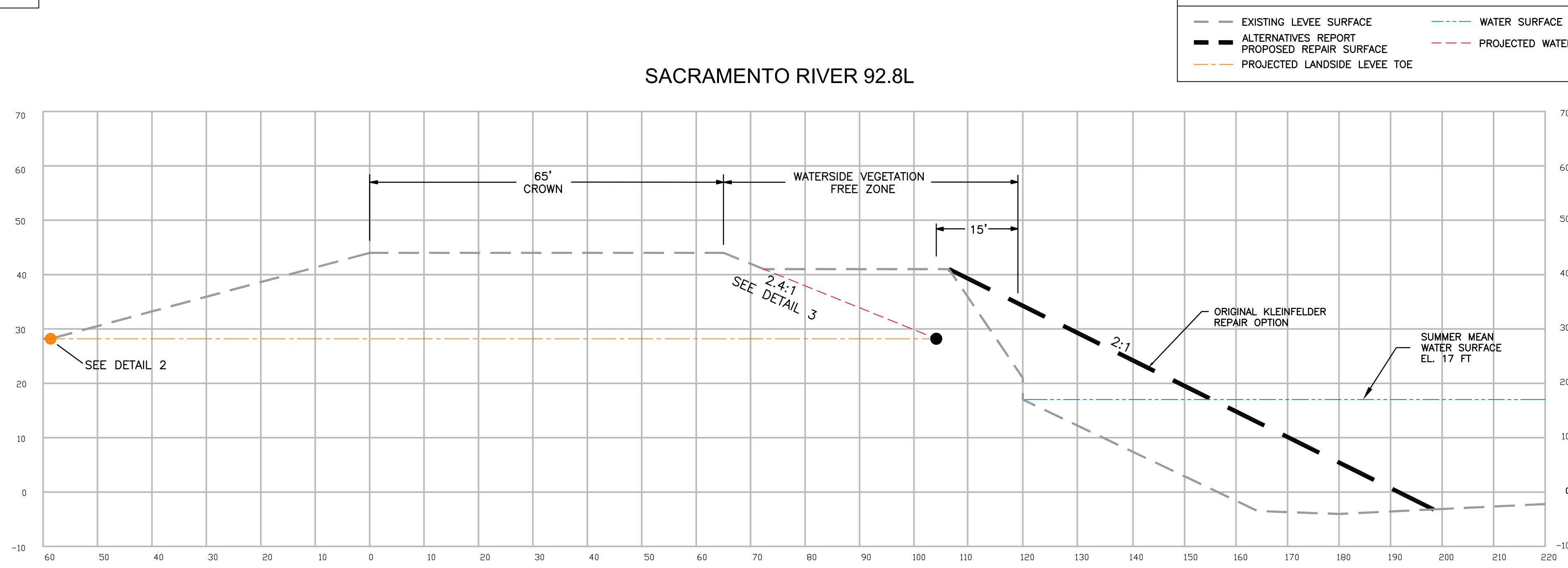
HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 86.9R

Date	3/31/11
Figure	SAC 86.9R

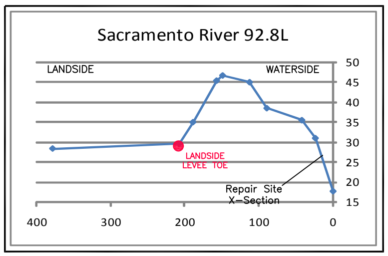
DETAIL 1



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



KEY DATA FOR DETERMINING LANDSIDE TOE

Comp Study Levee Crown Elevation	45.5
Comp Study Landside Toe Elevation	29.7
Comp Study Difference	15.8

Alternatives Report Analysis Levee Crown Elevation	44.0

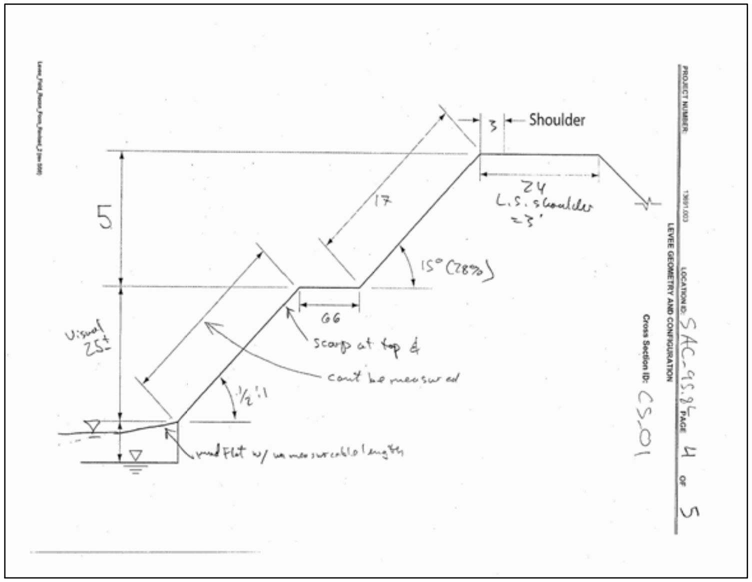
Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	28.2

Comp Study Data Points

Horizontal Distance to Data Point	Elevation at Data Point
0.0	17.8
23.9	31.1
42.2	35.6
89.5	38.6
112.8	45.1
149.2	46.8
157.4	45.5
189.3	35.1
207.8	29.7
378.8	28.5

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~92.8L

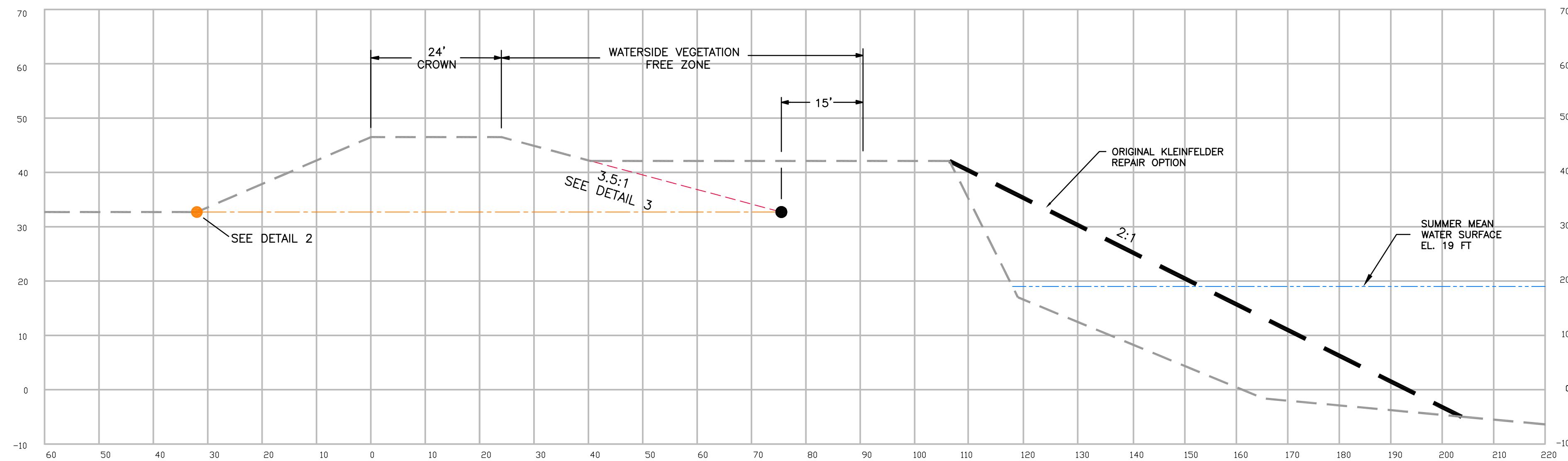
Date	3/31/11
Figure	SAC 92.8L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

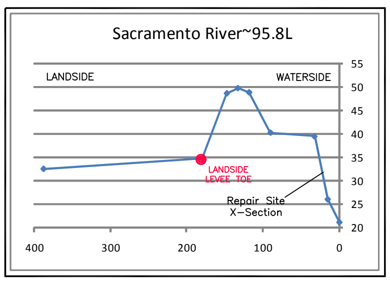
SACRAMENTO RIVER 95.8L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	20.1
15.3	25.0
32.7	38.4
90.3	38.9
118.4	47.8
133.1	48.8
147.6	47.6
179.6	33.8
388.0	31.5

KEY DATA FOR DETERMINING LANDSIDE TOE

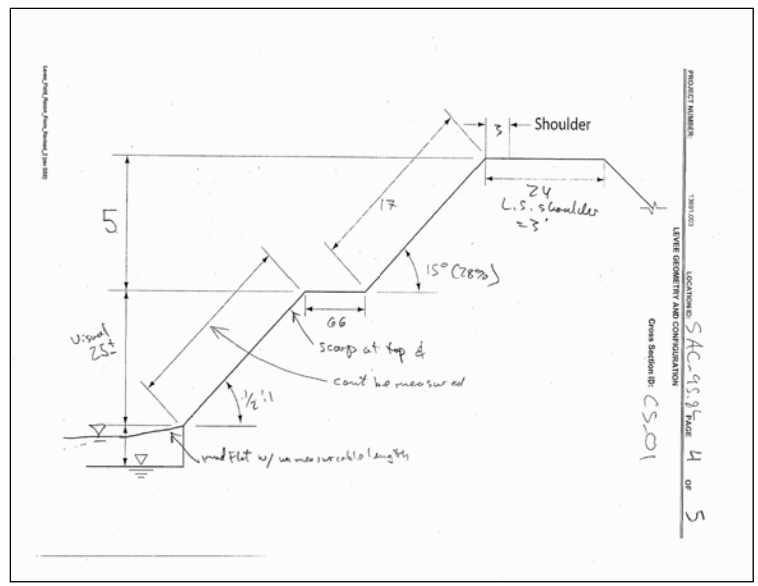
Comp Study Levee Crown Elevation	47.6
Comp Study Landside Toe Elevation	33.8
Comp Study Difference	13.8

Alternatives Report Analysis Levee Crown Elevation	46.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	32.7

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~95.8L

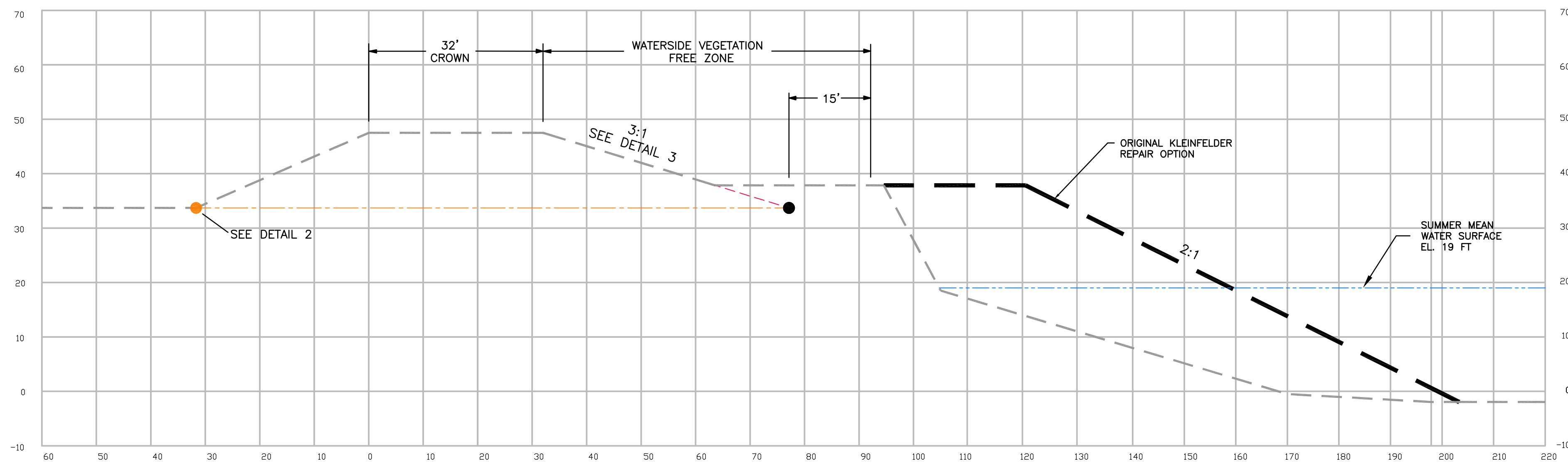
Date	3/31/11
Figure	SAC 95.8L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

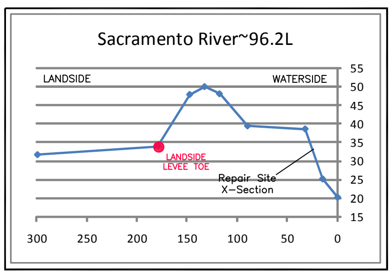
SACRAMENTO RIVER 96.2L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	20.4
15.0	25.3
32.4	38.7
90.0	39.5
118.0	48.1
132.9	50.0
147.7	47.9
179.4	34.1
299.7	31.8

KEY DATA FOR DETERMINING LANDSIDE TOE

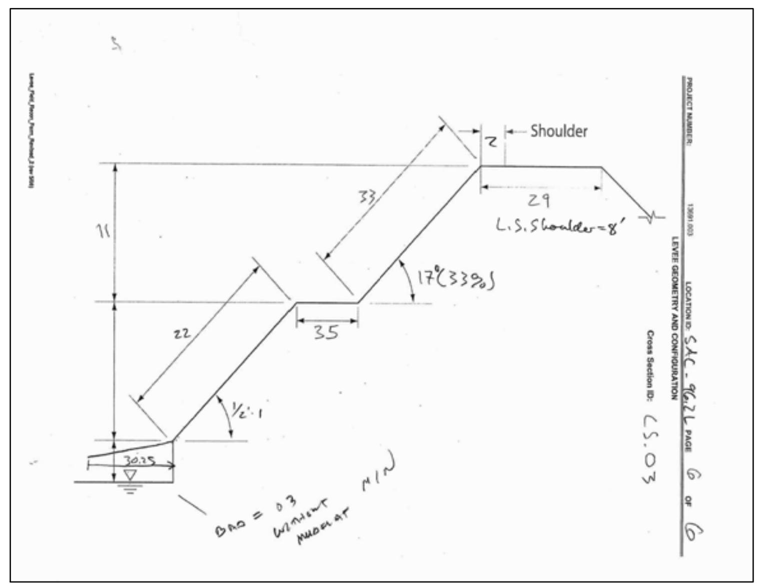
Comp Study Levee Crown Elevation	47.9
Comp Study Landside Toe Elevation	34.1
Comp Study Difference	13.8

Alternatives Report Analysis Levee Crown Elevation	47.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	33.7

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 96.2L

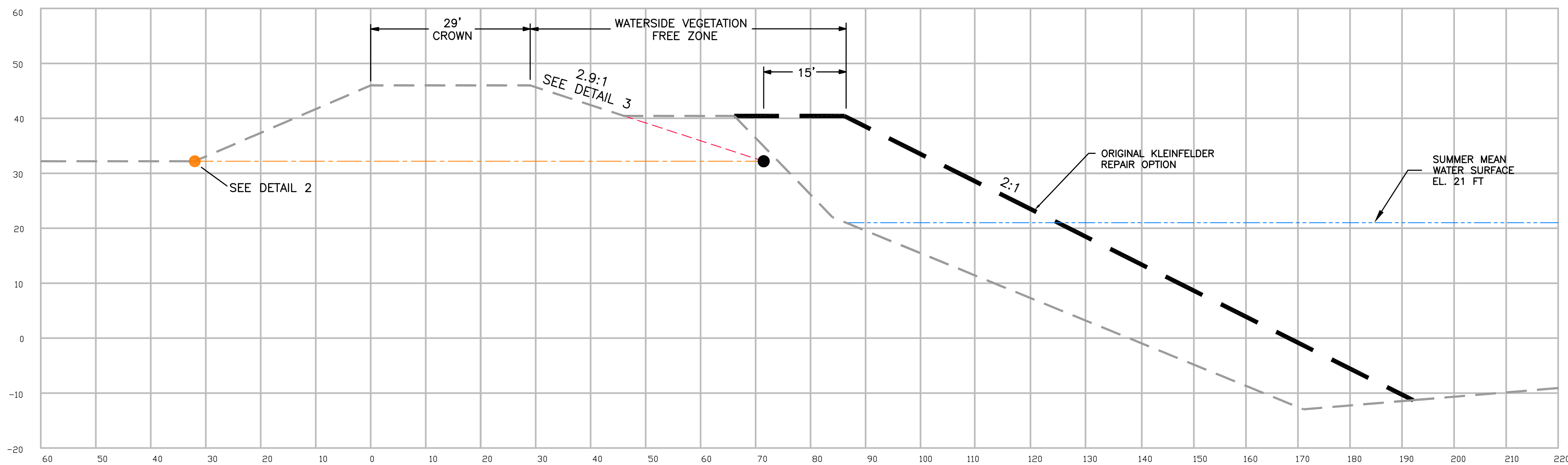
Date	3/31/11
Figure	SAC 96.2L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

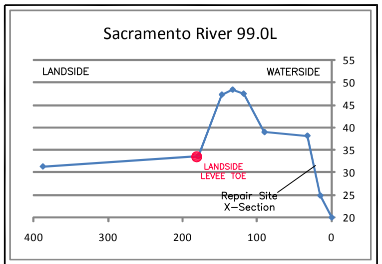
SACRAMENTO RIVER 99.0L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



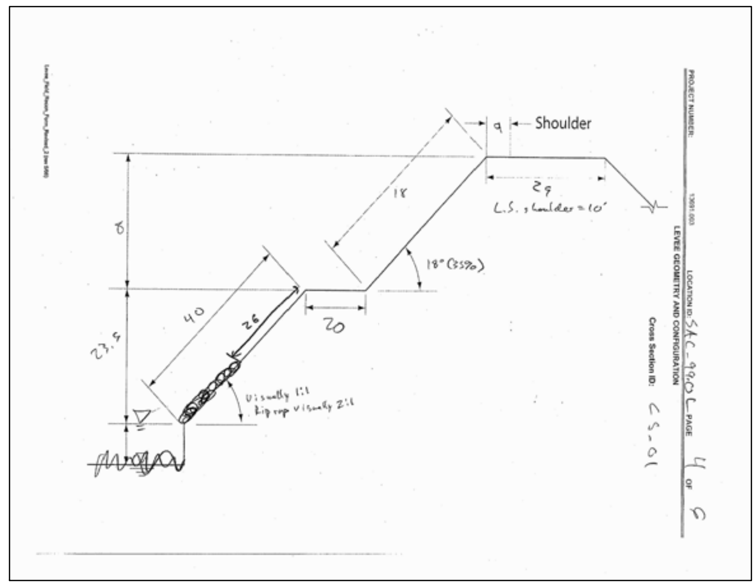
Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	20.0
15.3	24.9
32.7	38.3
90.3	39.1
118.4	47.7
133.1	48.6
147.6	47.5
179.6	33.7
388.0	31.4

KEY DATA FOR DETERMINING
LANDSIDE TOE

Comp Study Levee Crown Elevation	47.5
Comp Study Landside Toe Elevation	33.7
Comp Study Difference	13.8
Alternatives Report Analysis Levee Crown Elevation	46.0
Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	32.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

SACRAMENTO RIVER ~ 99.0L

Date 3/31/11

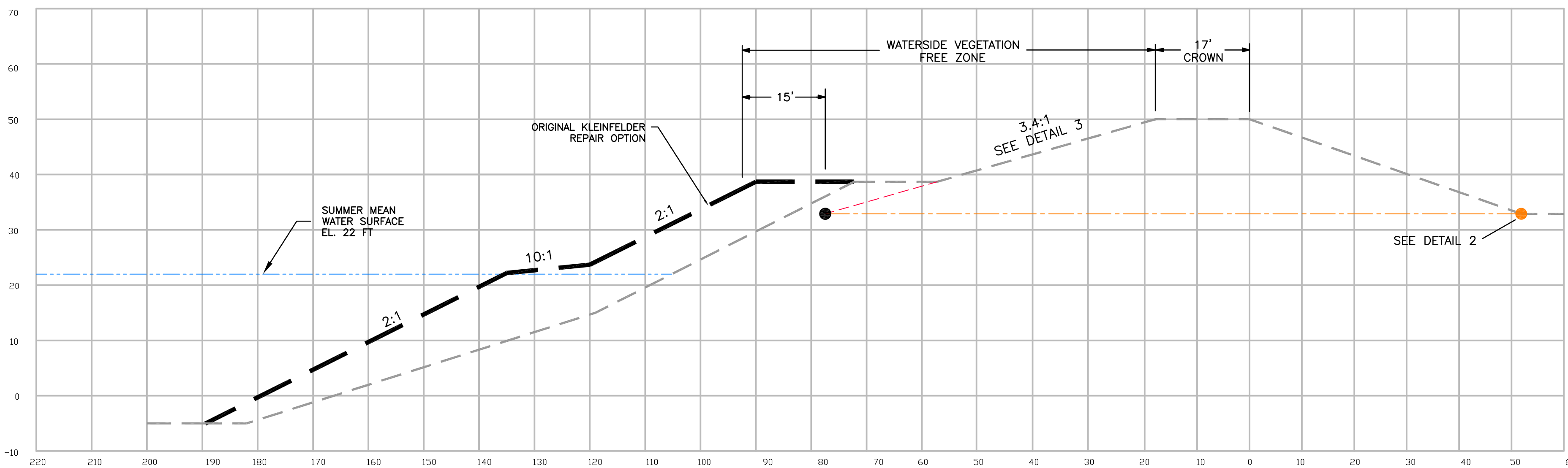
Figure SAC 99.0L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

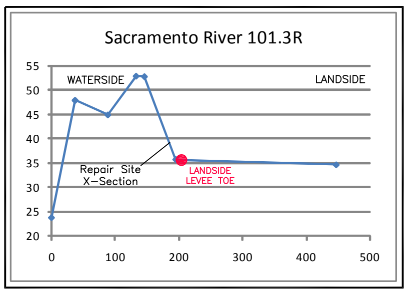
SACRAMENTO RIVER 101.3R



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points

Horizontal Distance to Data Point	Elevation at Data Point
0.0	23.8
37.1	48.0
88.6	45.0
132.9	53.0
146.1	52.9
195.2	35.8
447.2	34.7

KEY DATA FOR DETERMINING LANDSIDE TOE

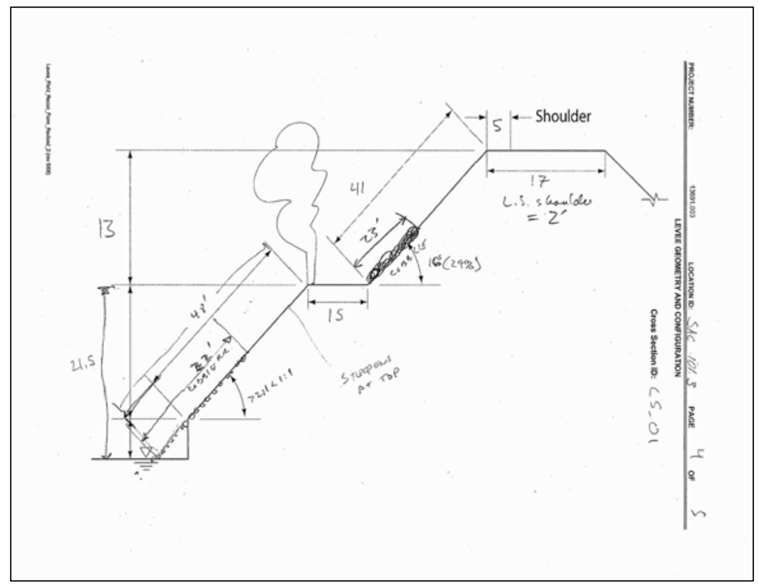
Comp Study Levee Crown Elevation	52.9
Comp Study Landside Toe Elevation	35.8
Comp Study Difference	17.1

Alternatives Report Analysis Levee Crown Elevation	50.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	32.9

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 101.3R

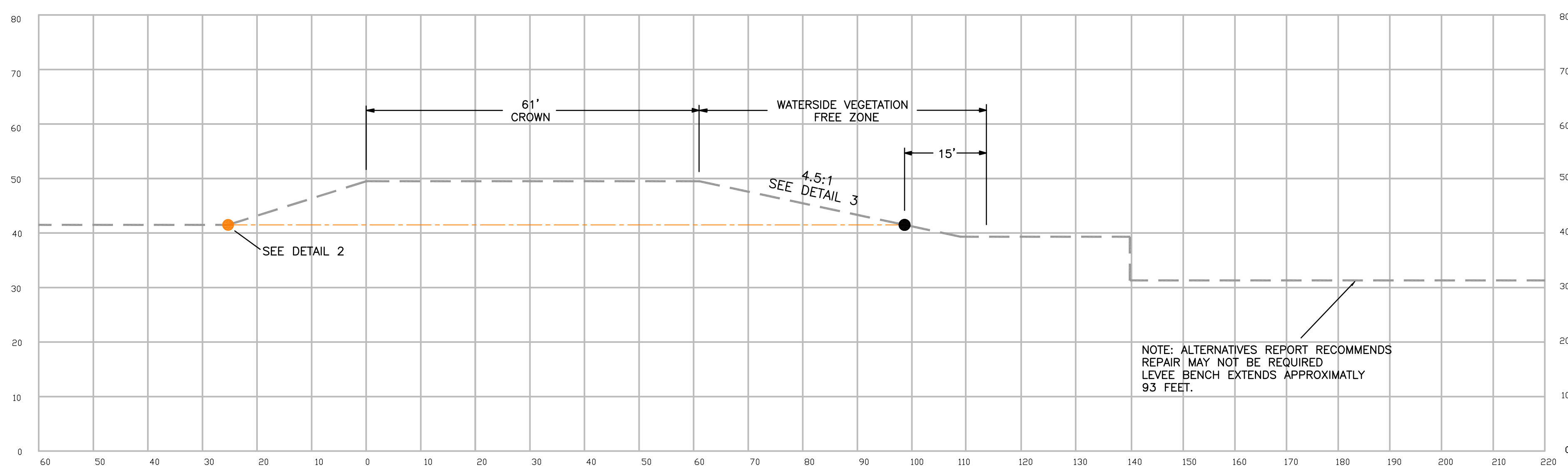
Date	3/31/11
Figure	SAC 101.3R

DETAIL 1

LEGEND

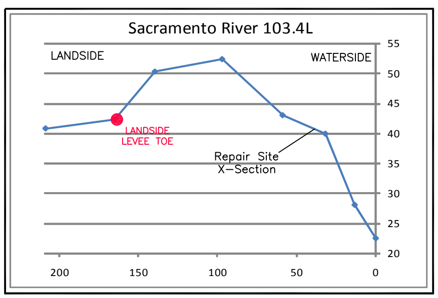
- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

SACRAMENTO RIVER 103.4L



DETAIL 2

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	22.5
13.3	28.1
31.8	40.0
58.9	43.1
97.1	52.5
139.6	50.4
164.9	42.4
208.7	40.9

KEY DATA FOR DETERMINING
LANDSIDE TOE

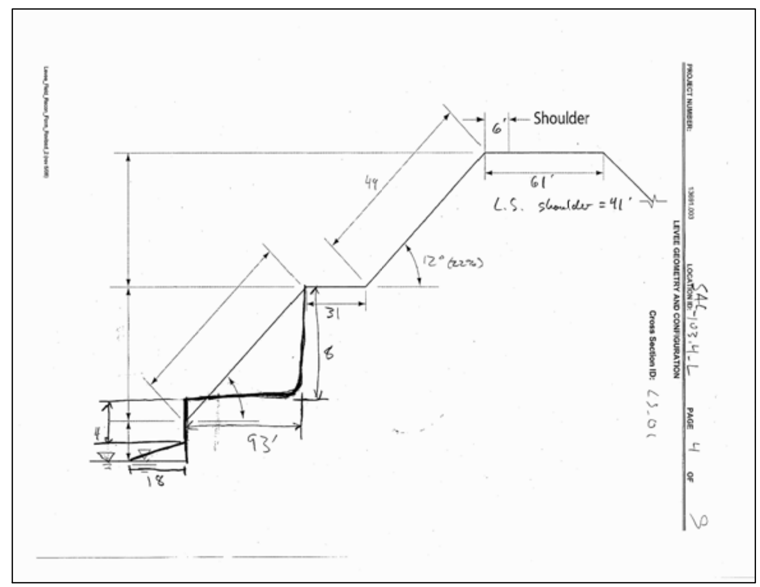
Comp Study Levee Crown Elevation	50.4
Comp Study Landside Toe Elevation	42.4
Comp Study Difference	8.0

Alternatives Report Analysis Levee Crown Elevation	49.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	41.5

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

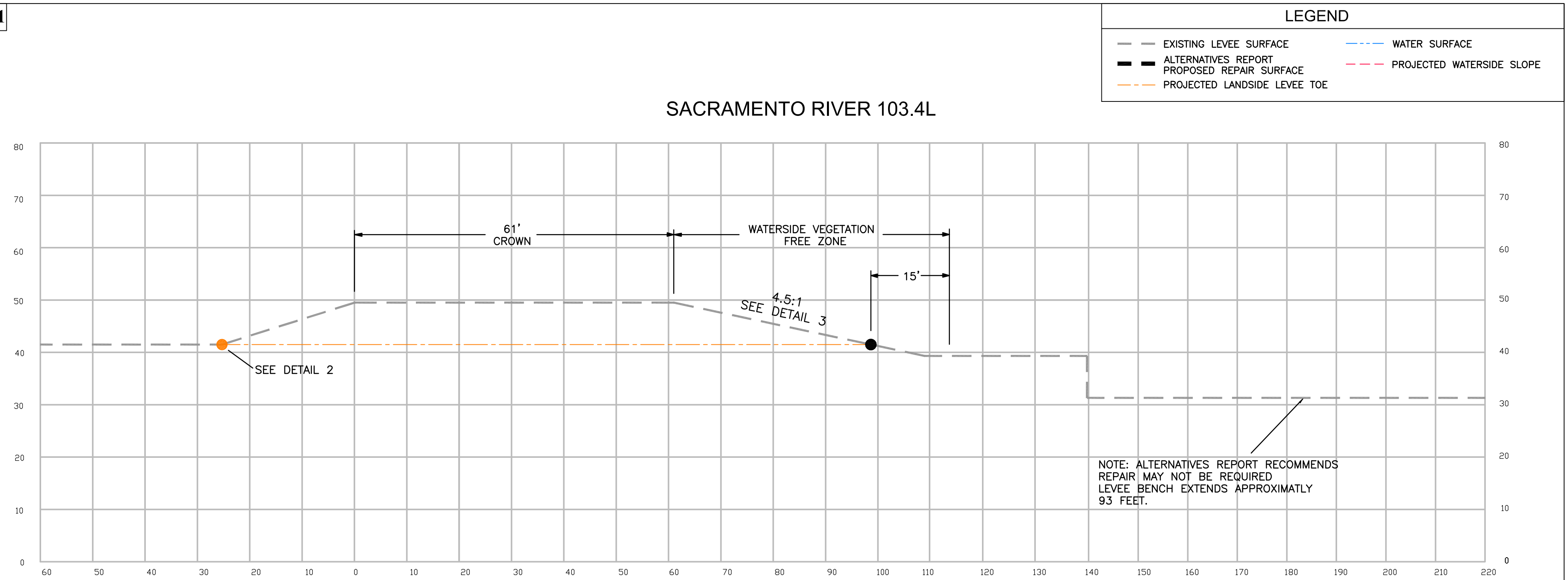
HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 103.4L

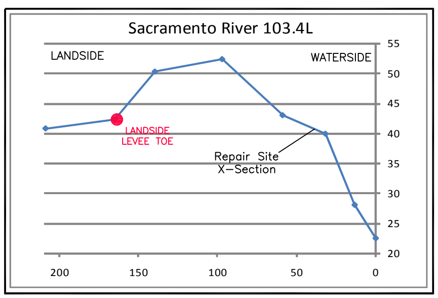
Date	3/31/11
Figure	SAC 103.4L

DETAIL 1



DETAIL 2

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	22.5
13.3	28.1
31.8	40.0
58.9	43.1
97.1	52.5
139.6	50.4
164.9	42.4
208.7	40.9

KEY DATA FOR DETERMINING LANDSIDE TOE

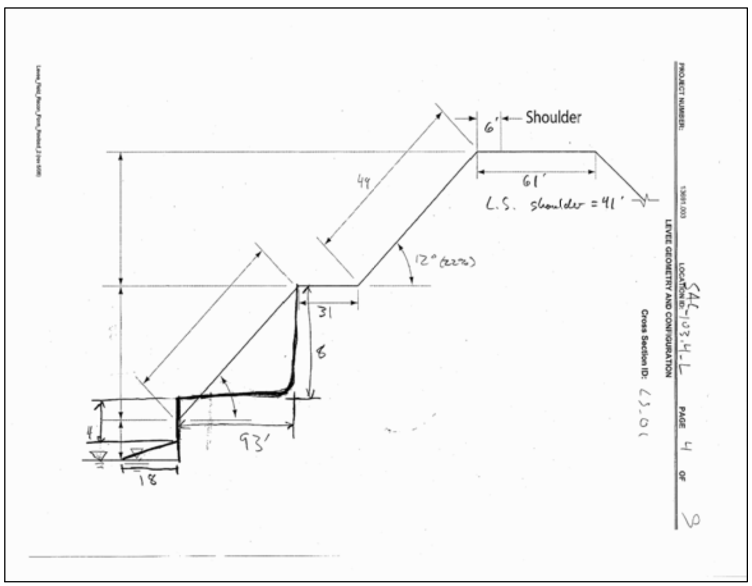
Comp Study Levee Crown Elevation	50.4
Comp Study Landside Toe Elevation	42.4
Comp Study Difference	8.0

Alternatives Report Analysis Levee Crown Elevation	49.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	41.5

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

SACRAMENTO RIVER ~ 103.4L

Date 3/31/11

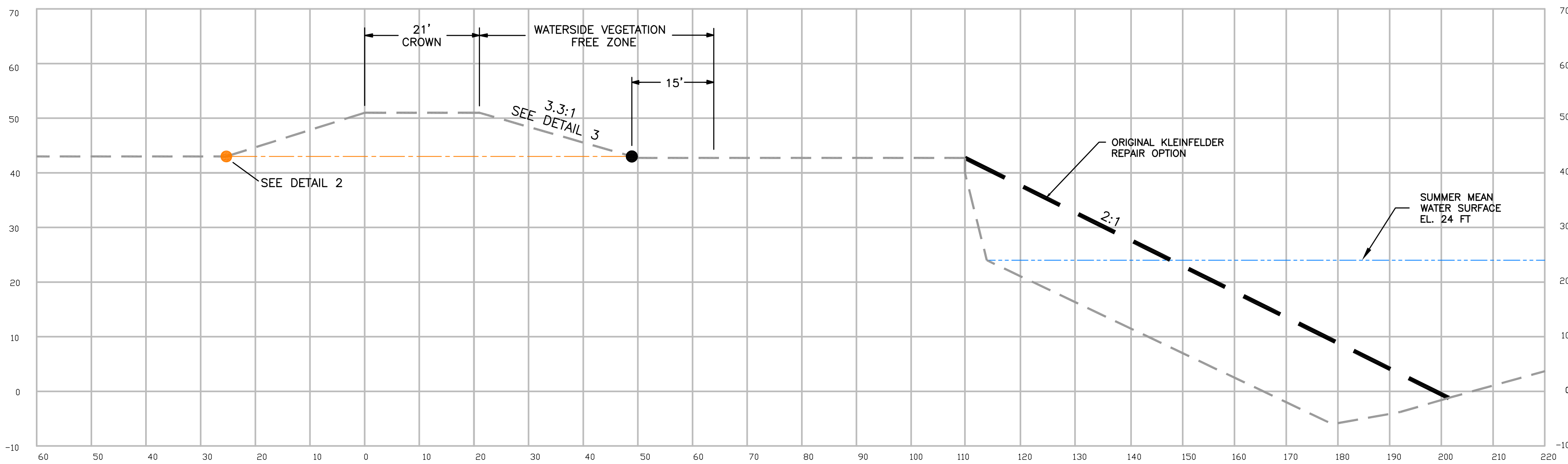
Figure SAC 103.4L

DETAIL 1

LEGEND

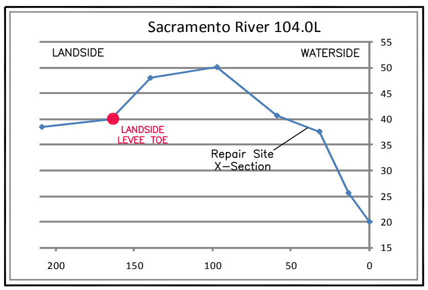
- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

SACRAMENTO RIVER 104.0L



DETAIL 2

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	20.1
13.3	25.7
31.8	37.6
58.9	40.7
97.1	50.1
139.6	48.0
164.9	40.0
208.7	38.5

KEY DATA FOR DETERMINING LANDSIDE TOE

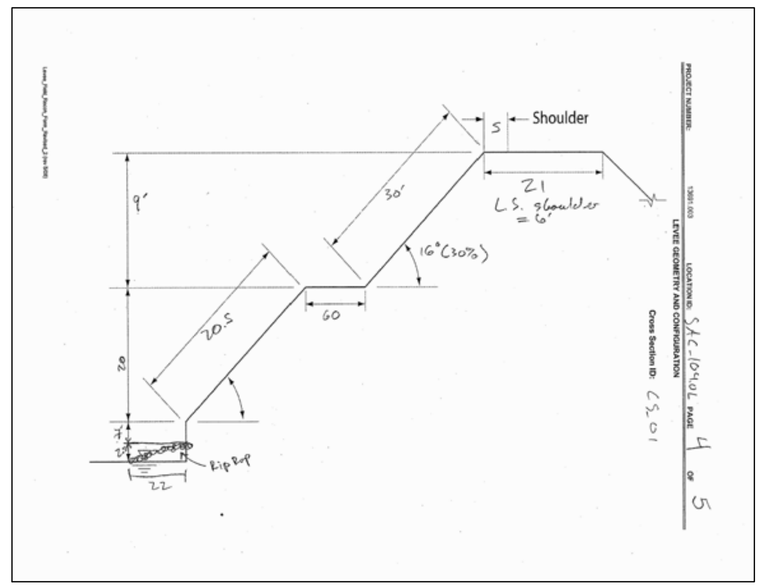
Comp Study Levee Crown Elevation	48.0
Comp Study Landside Toe Elevation	40.0
Comp Study Difference	8.0

Alternatives Report Analysis Levee Crown Elevation	51.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	43.0

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

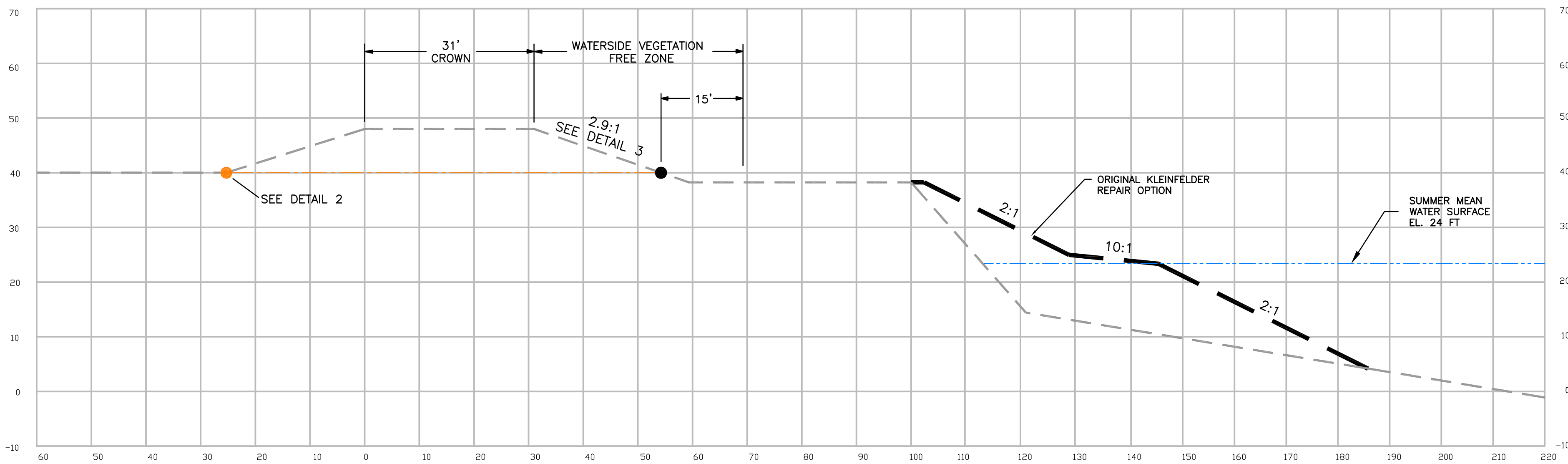
Drawn By: E. Smith	SACRAMENTO RIVER ~ 104.0L	Date	3/31/11
Checked By: R. Little		Figure	SAC 104.0L
QA/QC Review: D. Teak			

DETAIL 1

LEGEND

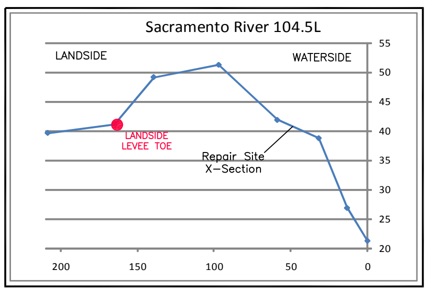
- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

SACRAMENTO RIVER 104.5L



DETAIL 2

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	21.3
13.3	26.9
31.8	38.8
58.9	41.9
97.1	51.3
139.6	49.2
164.9	41.2
208.7	39.7

KEY DATA FOR DETERMINING LANDSIDE TOE

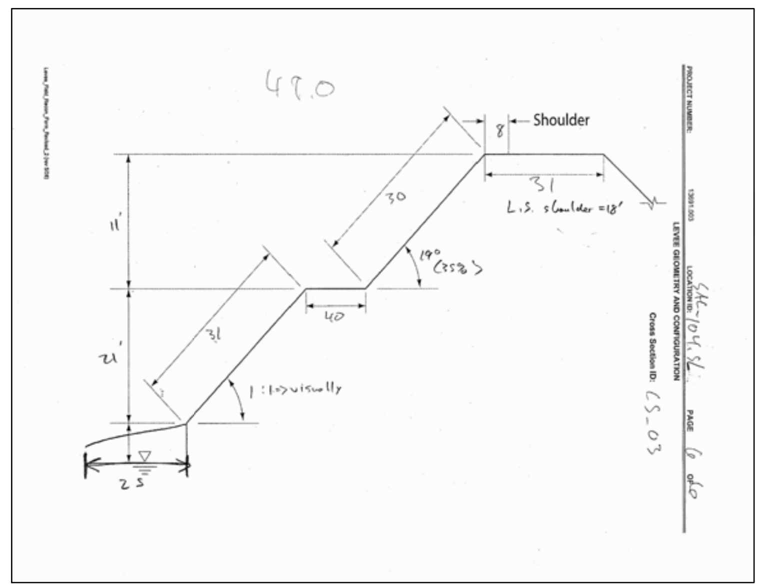
Comp Study Levee Crown Elevation	49.2
Comp Study Landside Toe Elevation	41.2
Comp Study Difference	8.0

Alternatives Report Analysis Levee Crown Elevation	48.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	40.0

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 104.5L

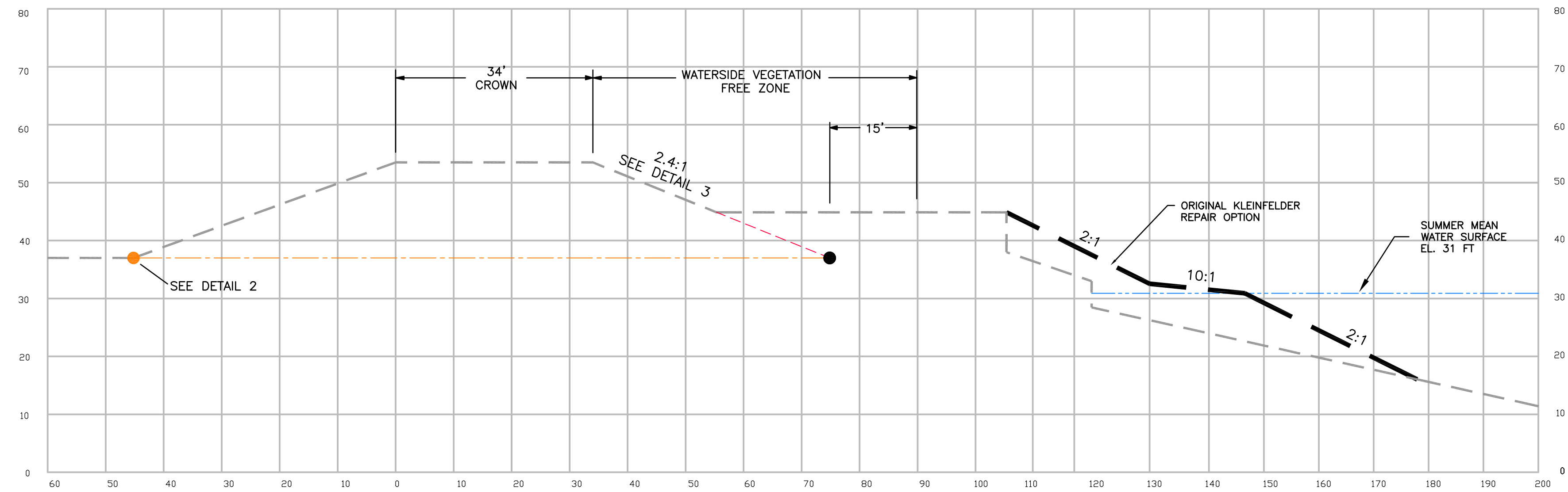
Date	3/31/11
Figure	SAC 104.5L

DETAIL 1

LEGEND

EXISTING LEVEE SURFACE	WATER SURFACE
ALTERNATIVES REPORT PROPOSED REPAIR SURFACE	PROJECTED WATERSIDE SLOPE
PROJECTED LANDSIDE LEVEE TOE	

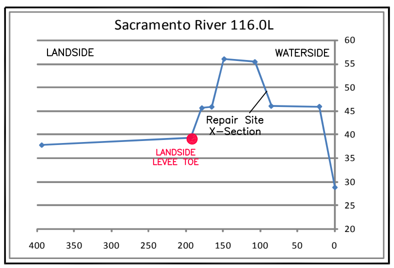
SACRAMENTO RIVER 116.0L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	28.9
20.9	45.9
85.7	46.0
107.6	55.3
148.9	55.9
165.7	45.8
178.8	45.6
194.1	39.4
394.1	37.8

KEY DATA FOR DETERMINING LANDSIDE TOE

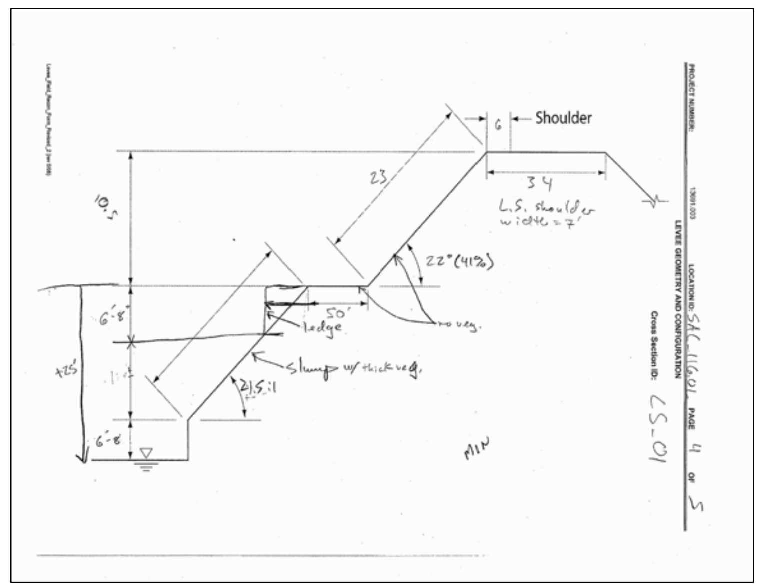
Comp Study Levee Crown Elevation	55.9
Comp Study Landside Toe Elevation	39.4
Comp Study Difference	16.5

Alternatives Report Analysis Levee Crown Elevation	53.5

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	37.0

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



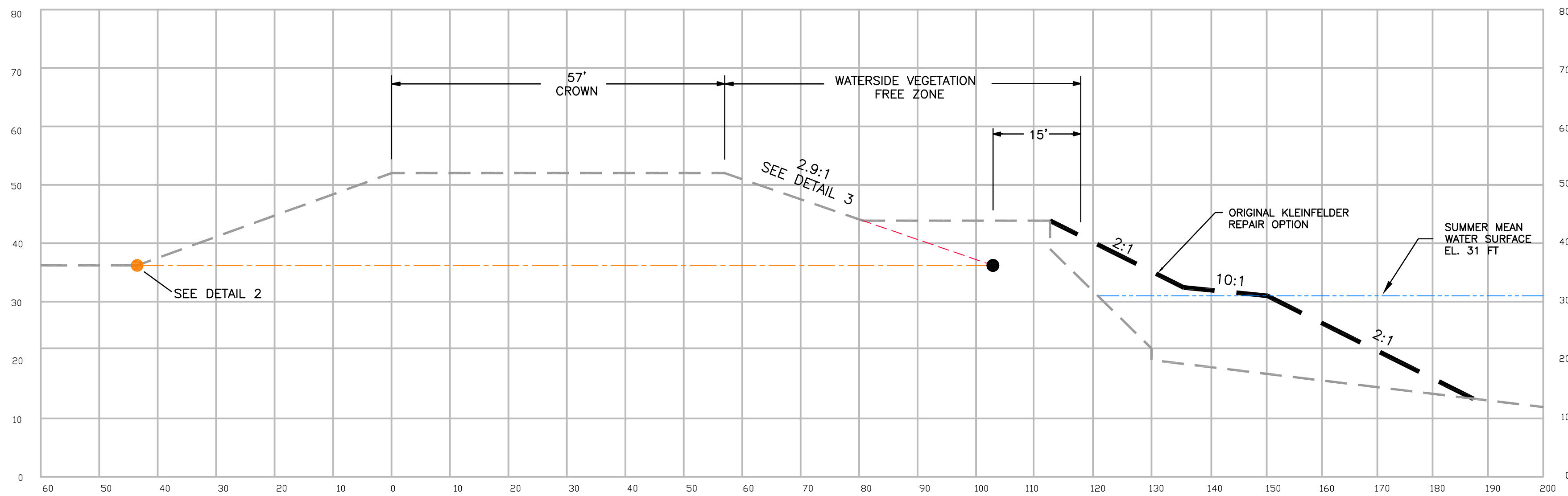
Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR HDR Engineering, Inc.	Drawn By: E. Smith Checked By: R. Little QA/QC Review: D. Teak	SACRAMENTO RIVER ~ 116.0L
		Date: 3/31/11	Figure: SAC 116.0L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

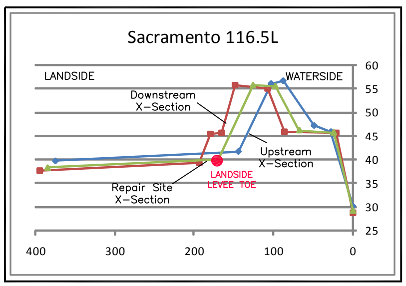
SACRAMENTO RIVER 116.5L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	29.3
24.3	45.9
67.4	46.4
97.8	55.8
125.8	56.0
169.3	40.1
384.4	38.4

KEY DATA FOR DETERMINING LANDSIDE TOE

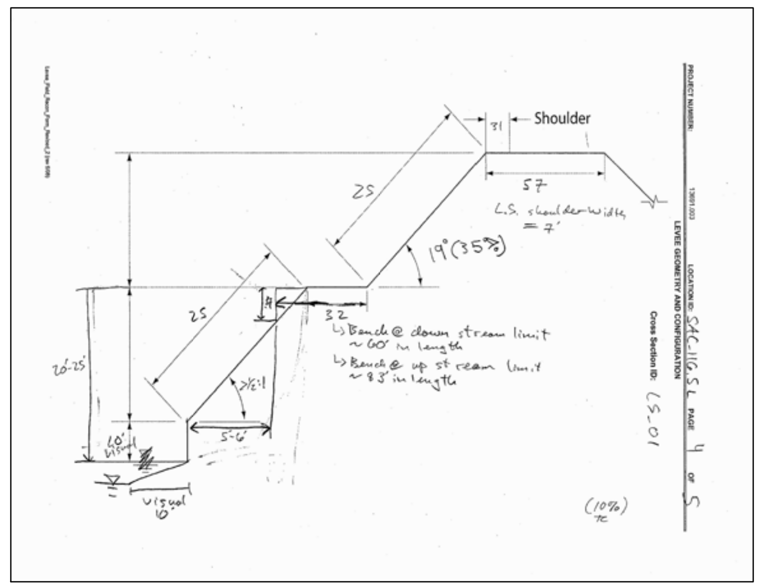
Comp Study Levee Crown Elevation	56.0
Comp Study Landside Toe Elevation	40.1
Comp Study Difference	15.8

Alternatives Report Analysis Levee Crown Elevation	52.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	36.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

SACRAMENTO RIVER ~ 116.5L

Date 3/31/11

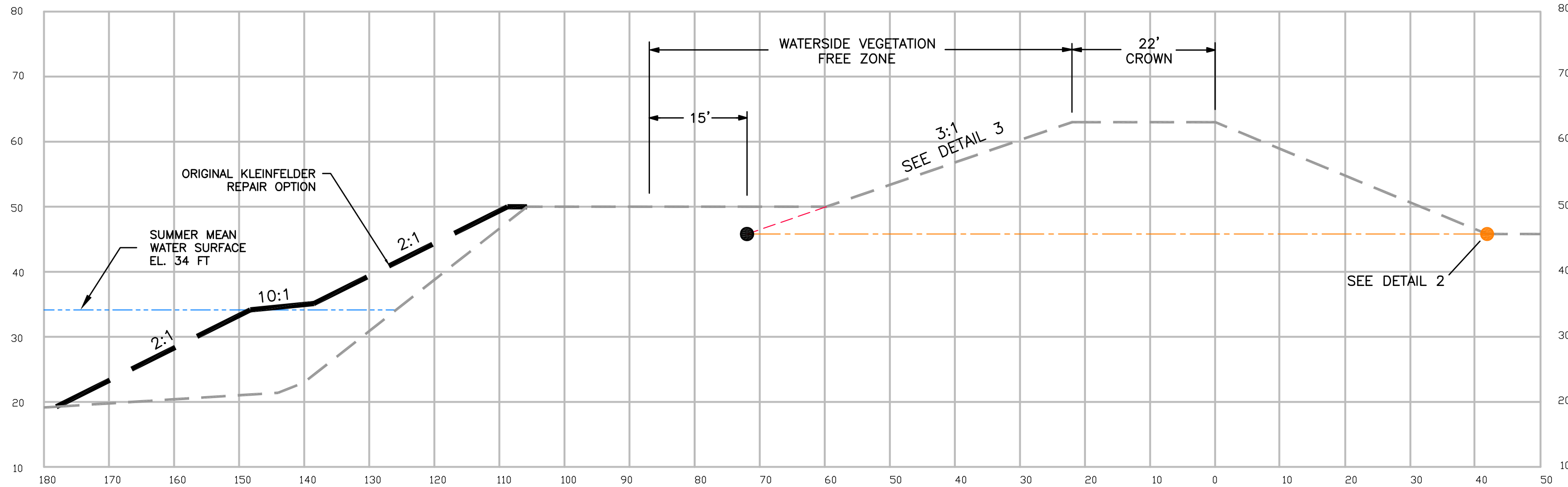
Figure SAC 116.5L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

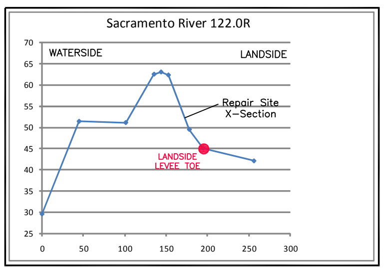
SACRAMENTO RIVER 122.0R



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	29.7
44.8	51.5
101.1	51.2
135.4	62.6
143.8	63.1
152.7	62.4
178.0	49.6
194.5	45.2
256.0	42.2

KEY DATA FOR DETERMINING LANDSIDE TOE

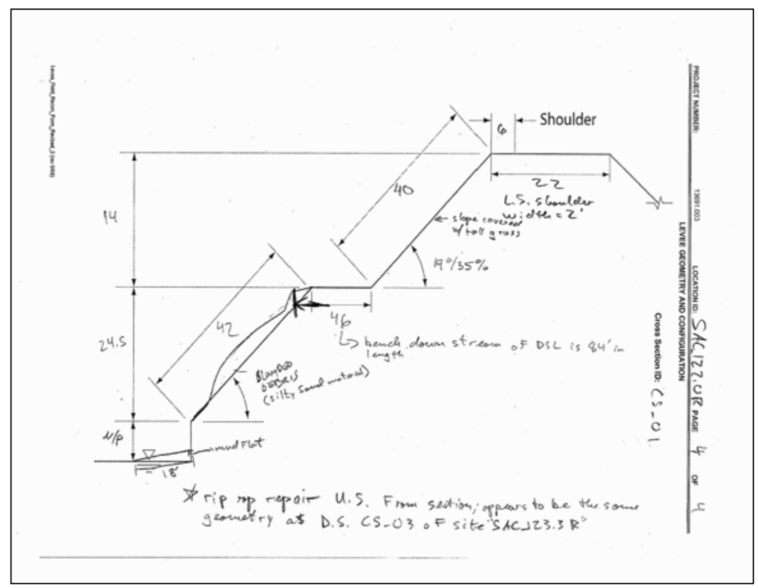
Comp Study Levee Crown Elevation	62.4
Comp Study Landside Toe Elevation	45.2
Comp Study Difference	17.2

Alternatives Report Analysis Levee Crown Elevation	63.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	45.8

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

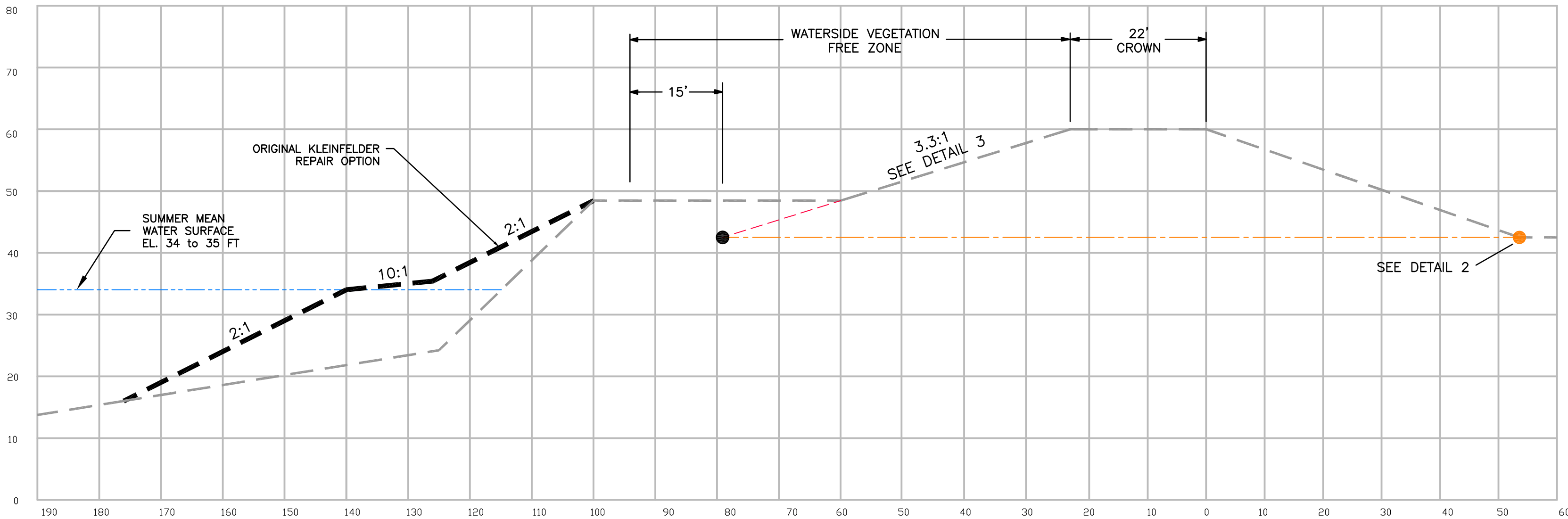
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 122.0R

Date	3/31/11
Figure	SAC 122.0R

DETAIL 1

SACRAMENTO RIVER 122.3R



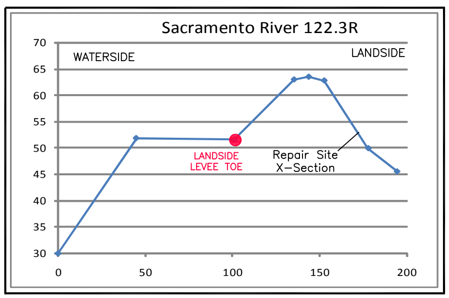
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	30.1
44.8	51.9
101.1	51.6
135.4	63.0
143.8	63.5
152.7	62.8
178.0	50.0
194.5	45.6

KEY DATA FOR DETERMINING LANDSIDE TOE

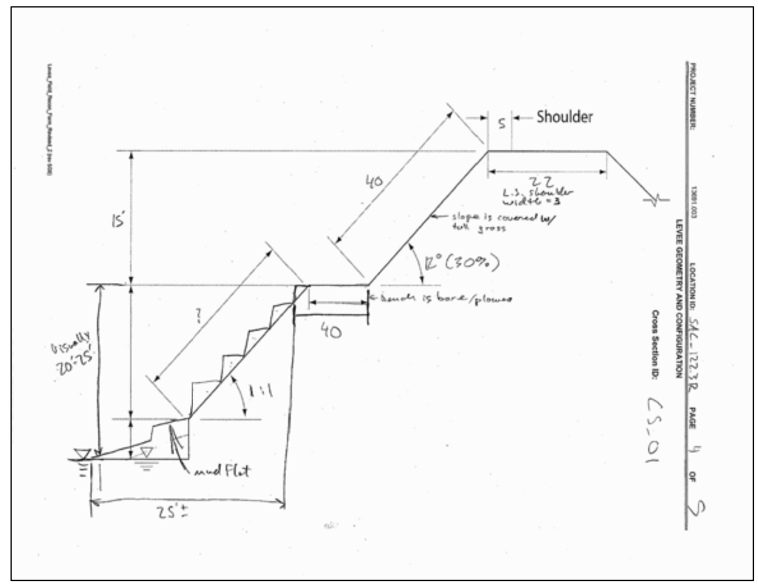
Comp Study Levee Crown Elevation	63.1
Comp Study Landside Toe Elevation	45.6
Comp Study Difference	17.5

Alternatives Report Analysis Levee Crown Elevation	60.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	42.5

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

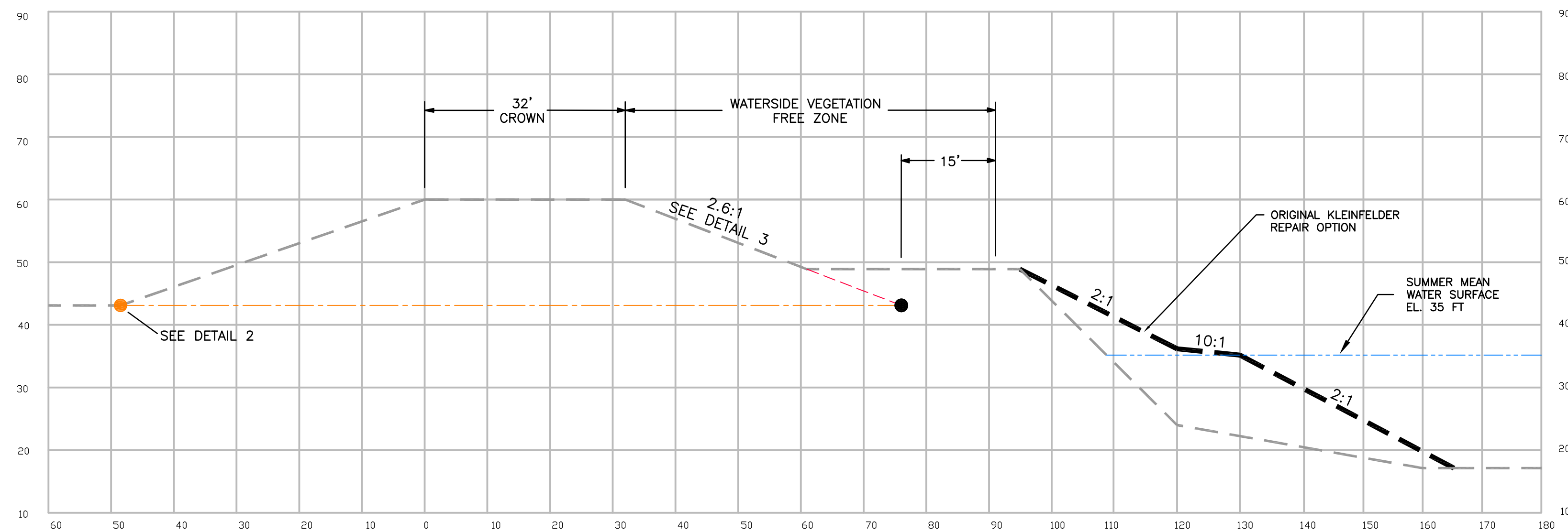
SACRAMENTO RIVER ~ 122.3R

Date: 3/31/11

Figure: SAC 122.3R

DETAIL 1

SACRAMENTO RIVER 123.3L



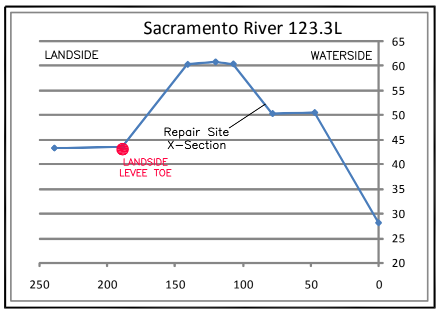
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	28.1
47.0	50.5
78.1	50.3
107.0	60.4
120.1	60.8
140.6	60.3
189.1	43.4
238.9	43.3

KEY DATA FOR DETERMINING LANDSIDE TOE

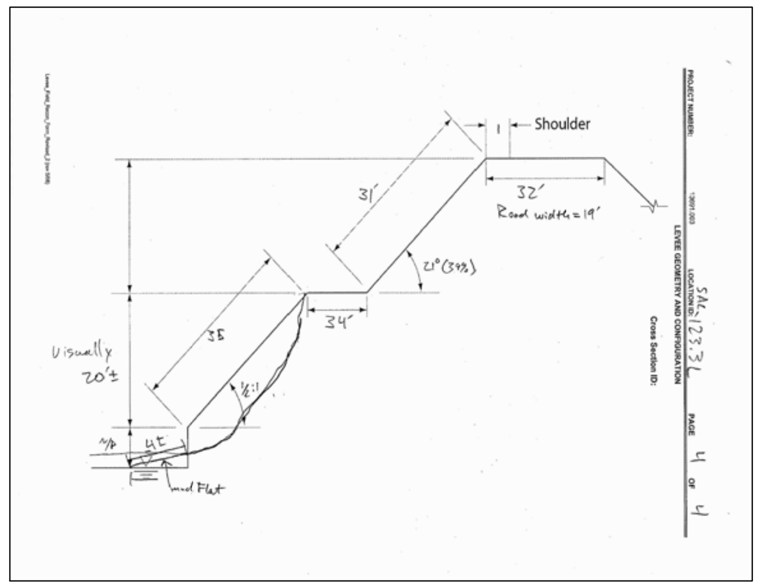
Comp Study Levee Crown Elevation	60.3
Comp Study Landside Toe Elevation	43.4
Comp Study Difference	16.9

Alternatives Report Analysis Levee Crown Elevation	60.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	43.1

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

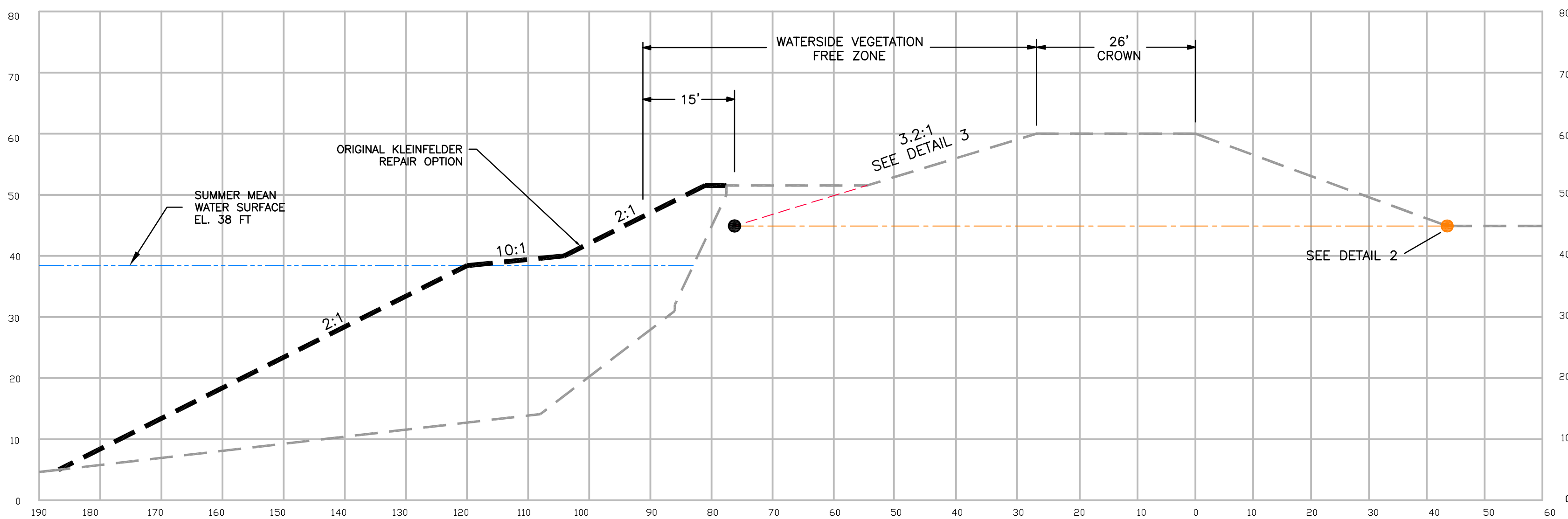
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 123.3L

Date	3/31/11
Figure	SAC 123.3L

DETAIL 1

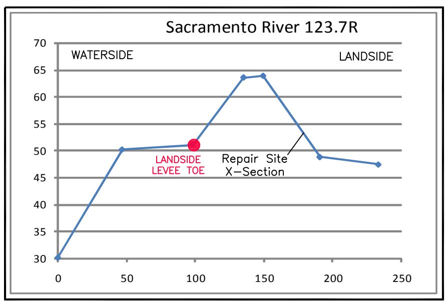
SACRAMENTO RIVER 123.7R



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	30.2
46.7	50.3
97.9	51.2
135.1	63.7
149.3	64.0
190.5	48.9
233.1	47.5

KEY DATA FOR DETERMINING LANDSIDE TOE

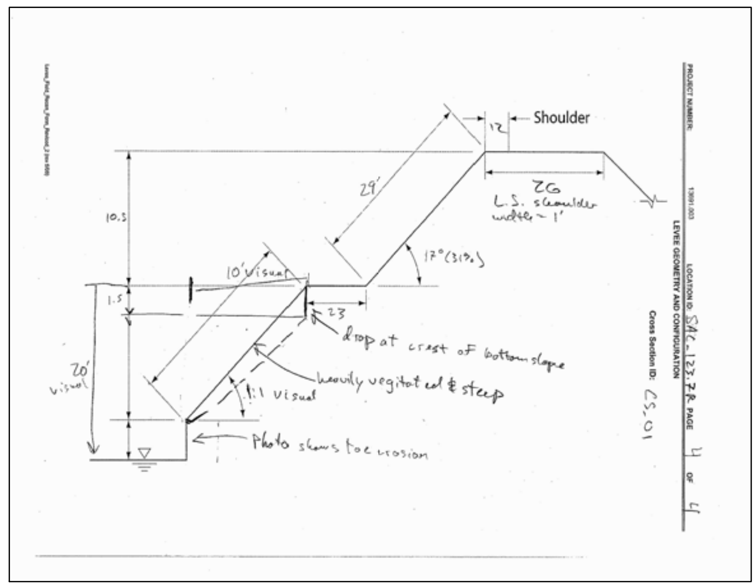
Comp Study Levee Crown Elevation	64.0
Comp Study Landside Toe Elevation	48.9
Comp Study Difference	15.1

Alternatives Report Analysis Levee Crown Elevation	60.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	44.9

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

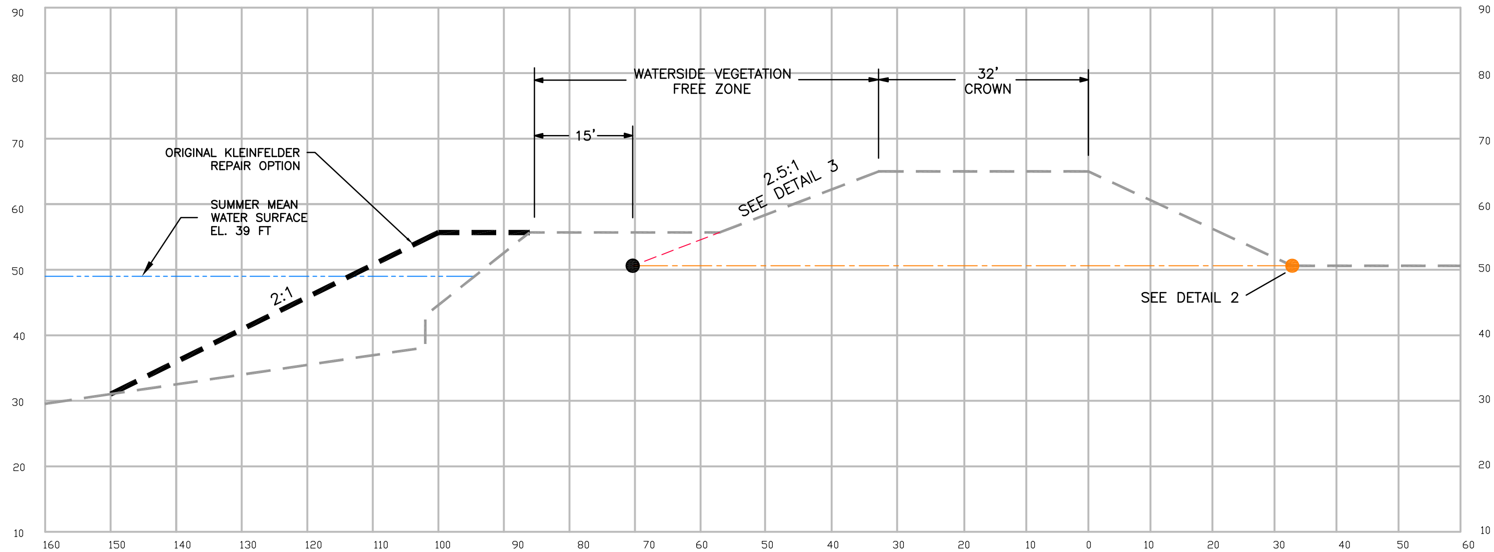
SACRAMENTO RIVER ~ 123.7R

Date 3/31/11

Figure SAC 123.7R

DETAIL 1

SACRAMENTO RIVER 127.9R



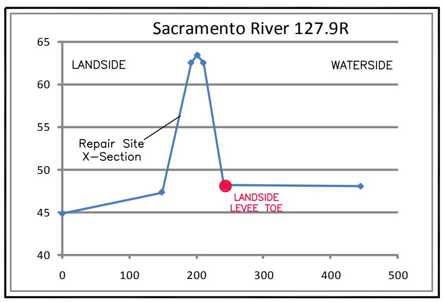
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	44.9
148.2	47.4
191.6	62.6
201.1	63.5
210.1	62.6
241.2	48.2
444.6	48.1

KEY DATA FOR DETERMINING LANDSIDE TOE

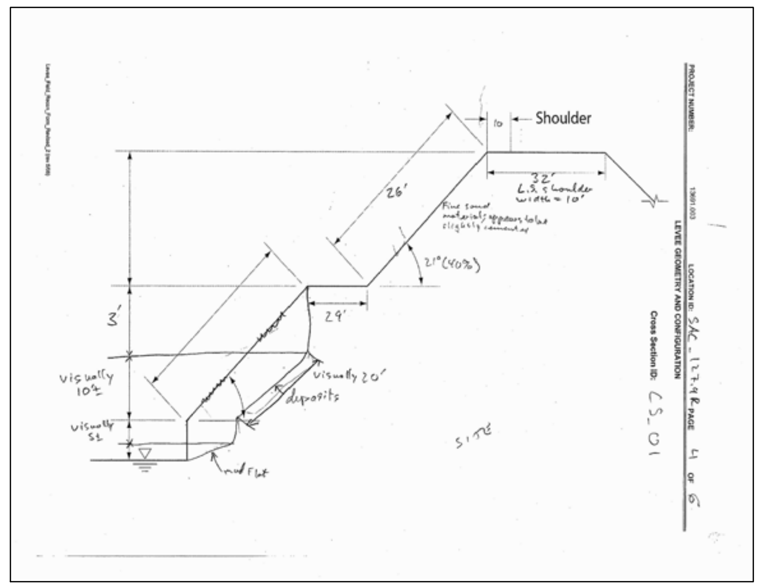
Comp Study Levee Crown Elevation	62.6
Comp Study Landside Toe Elevation	48.2
Comp Study Difference	14.4

Alternatives Report Analysis Levee Crown Elevation	65.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	50.6

DETAIL 3

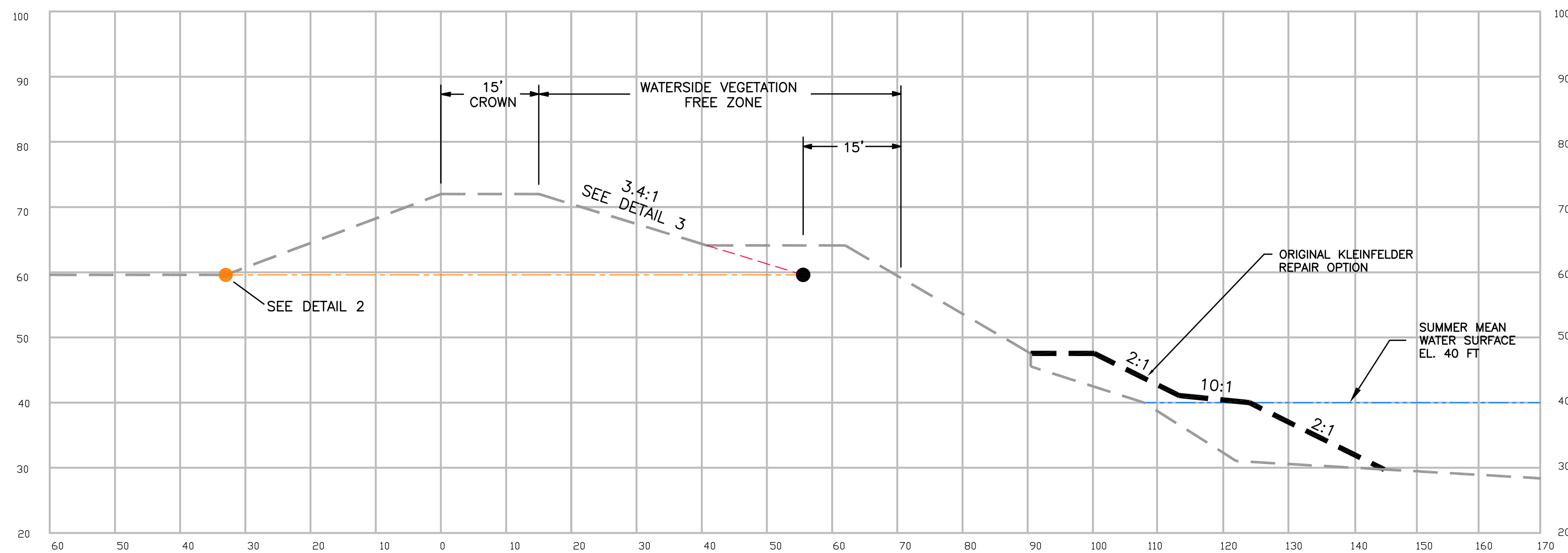
ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR Engineering, Inc.	Drawn By: E. Smith Checked By: R. Little QA/QC Review: D. Teak	SACRAMENTO RIVER ~ 127.9R
		Date	3/31/11
		Figure	SAC 127.9R

DETAIL 1

SACRAMENTO RIVER 131.8L



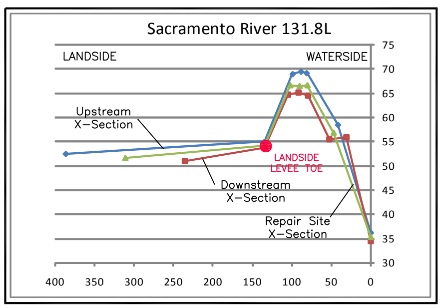
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	35.4
47.1	57.0
80.1	66.8
90.3	66.7
102.0	66.8
135.0	54.4
310.9	51.7

KEY DATA FOR DETERMINING LANDSIDE TOE

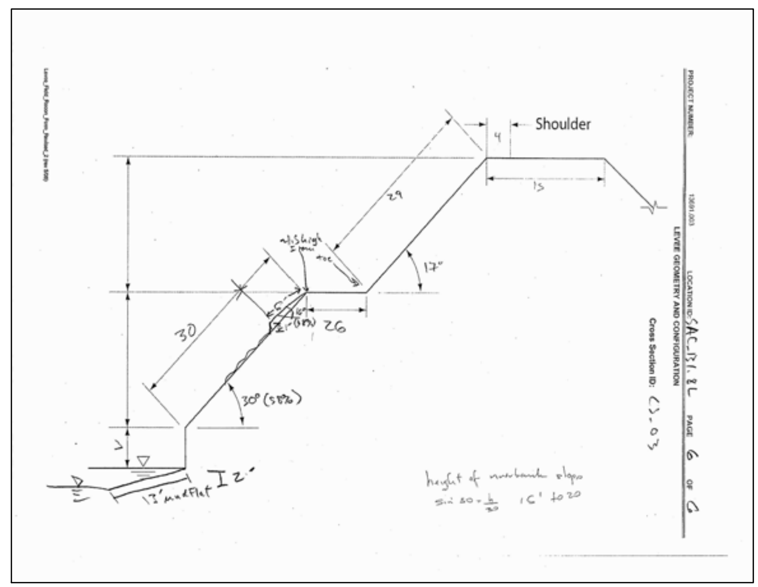
Comp Study Levee Crown Elevation	66.8
Comp Study Landside Toe Elevation	54.4
Comp Study Difference	12.4

Alternatives Report Analysis Levee Crown Elevation	72.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	59.6

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

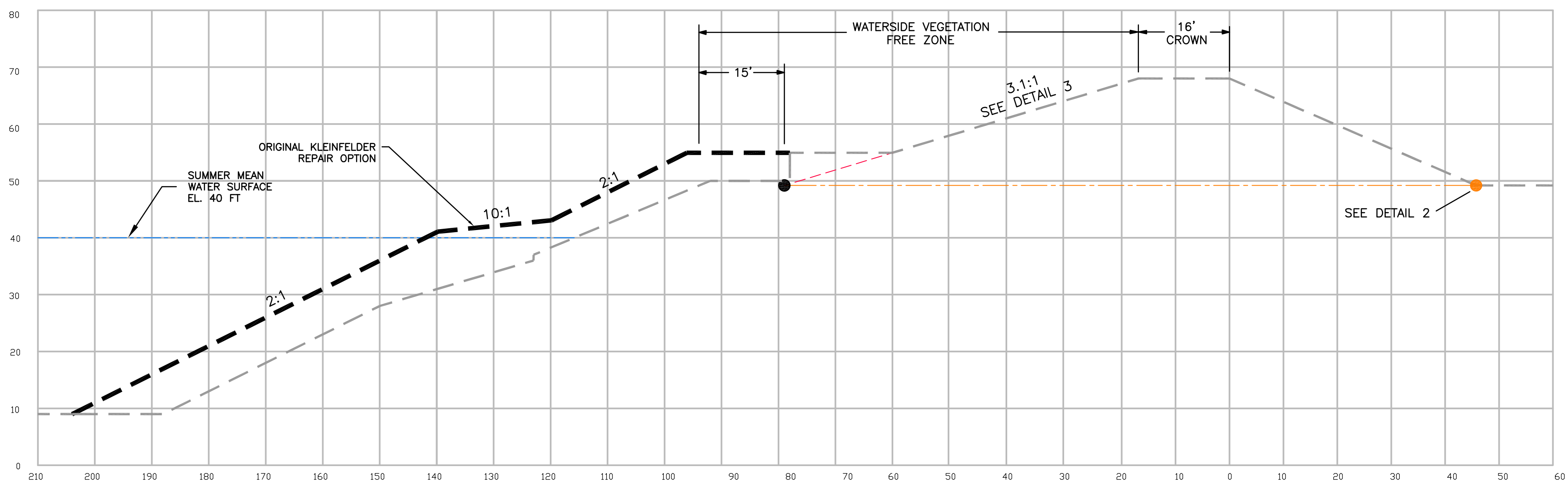
Drawn By: E. Smith	SACRAMENTO RIVER ~ 131.8L	Date	3/31/11
Checked By: R. Little		Figure	SAC 131.8L
QA/QC Review: D. Teak			

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

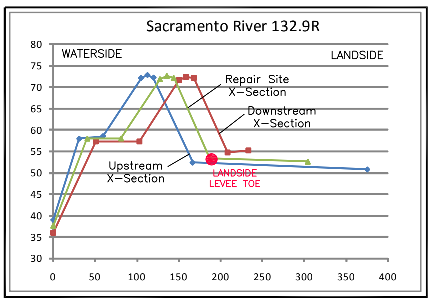
SACRAMENTO RIVER 132.9R



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	37.8
40.7	58.1
81.0	58.1
127.5	72.0
135.7	72.7
144.0	72.3
187.3	53.4
304.1	52.7

KEY DATA FOR DETERMINING LANDSIDE TOE

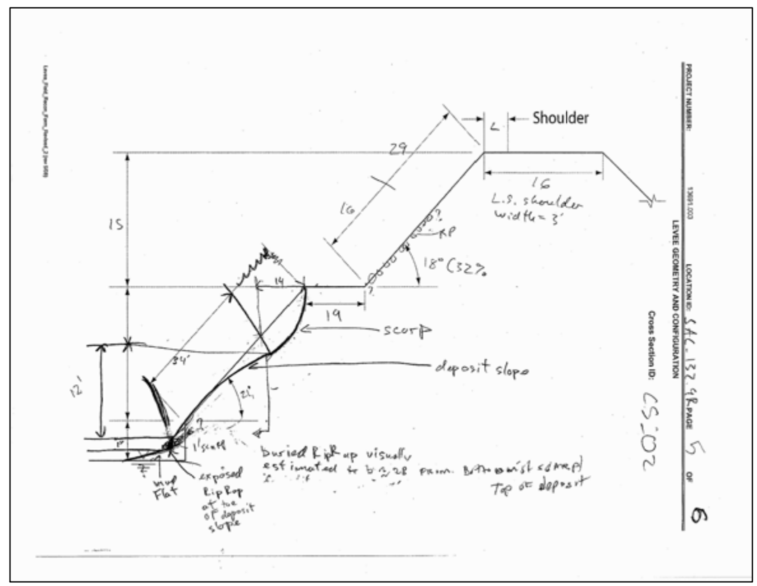
Comp Study Levee Crown Elevation	72.3
Comp Study Landside Toe Elevation	53.4
Comp Study Difference	18.8

Alternatives Report Analysis Levee Crown Elevation	68.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	49.2

DETAIL 3

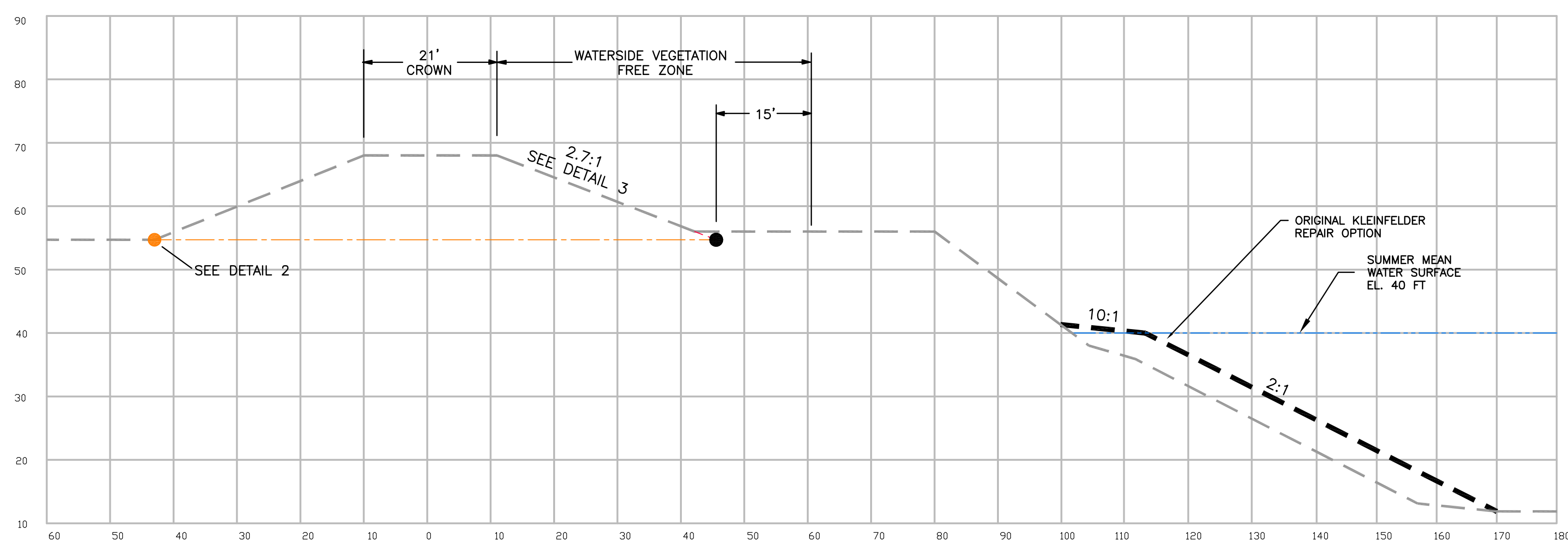
ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR Engineering, Inc.	Drawn By: E. Smith Checked By: R. Little QA/QC Review: D. Teak	SACRAMENTO RIVER ~ 132.9R	Date 3/31/11 Figure SAC 132.9R

DETAIL 1

SACRAMENTO RIVER 133.0L



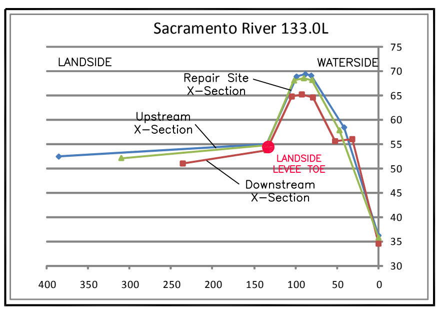
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	35.8
47.1	57.8
80.1	68.2
90.3	68.5
102.0	68.1
135.0	54.7
310.9	52.2

KEY DATA FOR DETERMINING
LANDSIDE TOE

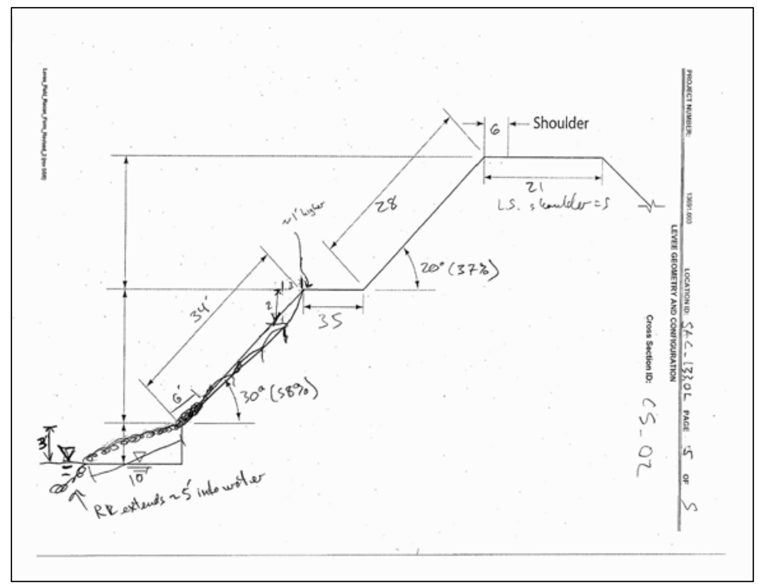
Comp Study Levee Crown Elevation	68.1
Comp Study Landside Toe Elevation	54.7
Comp Study Difference	13.3

Alternatives Report Analysis Levee Crown Elevation	68.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	54.7

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

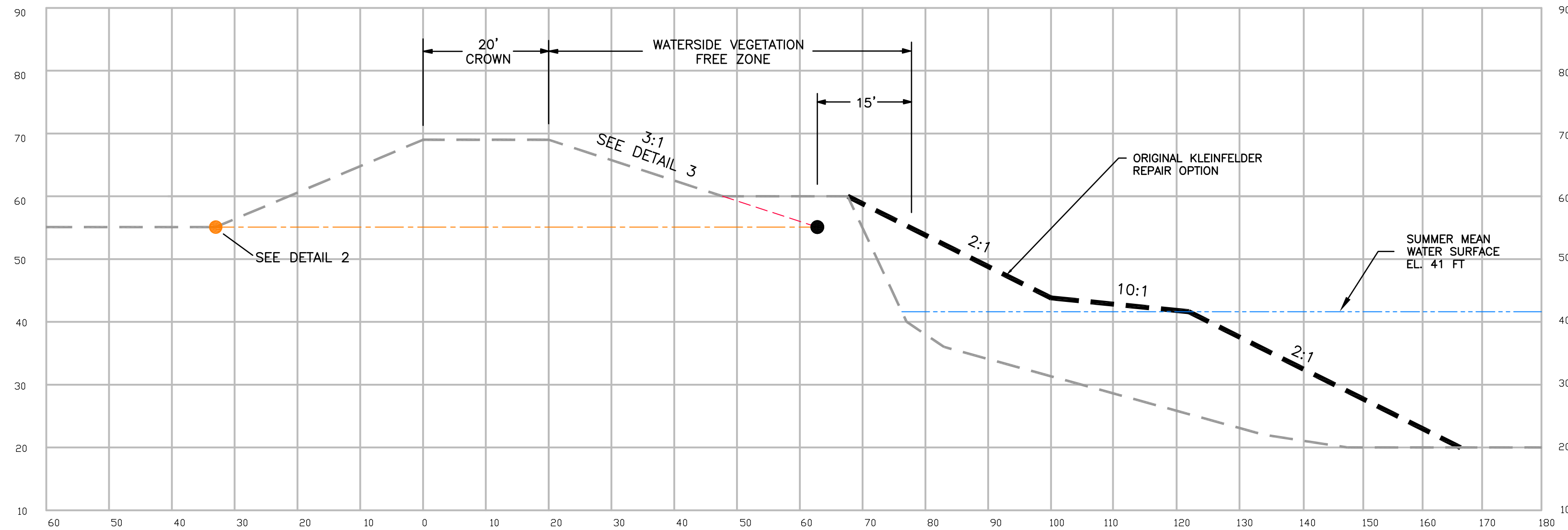
SACRAMENTO RIVER ~ 133.0L

Date 3/31/11

Figure SAC 133.0L

DETAIL 1

SACRAMENTO RIVER 133.8L



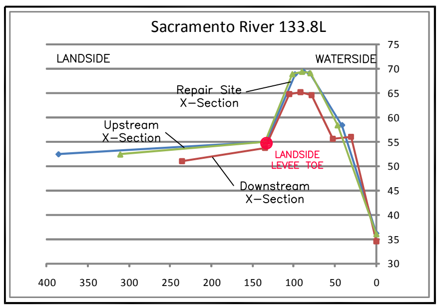
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	36.2
47.1	58.4
80.1	69.1
90.3	69.4
102.0	68.9
135.0	55.0
310.9	52.5

KEY DATA FOR DETERMINING LANDSIDE TOE

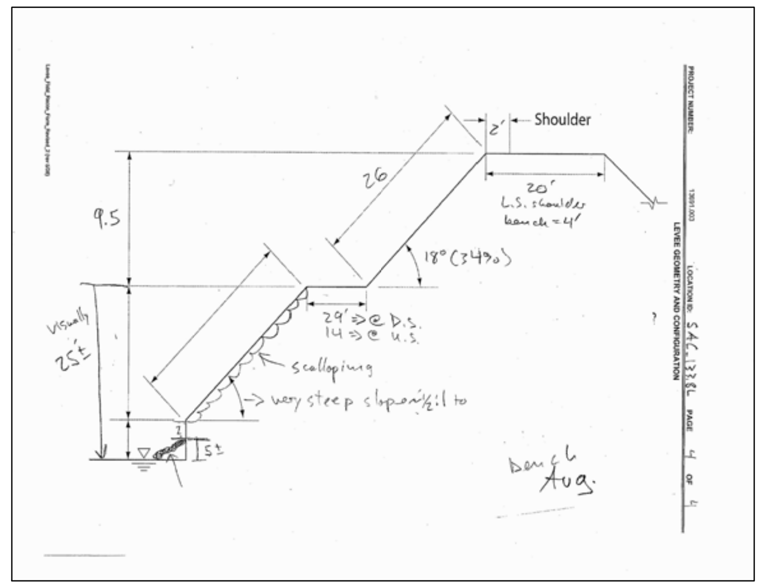
Comp Study Levee Crown Elevation	68.9
Comp Study Landside Toe Elevation	55.0
Comp Study Difference	13.9

Alternatives Report Analysis Levee Crown Elevation	69.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	55.1

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

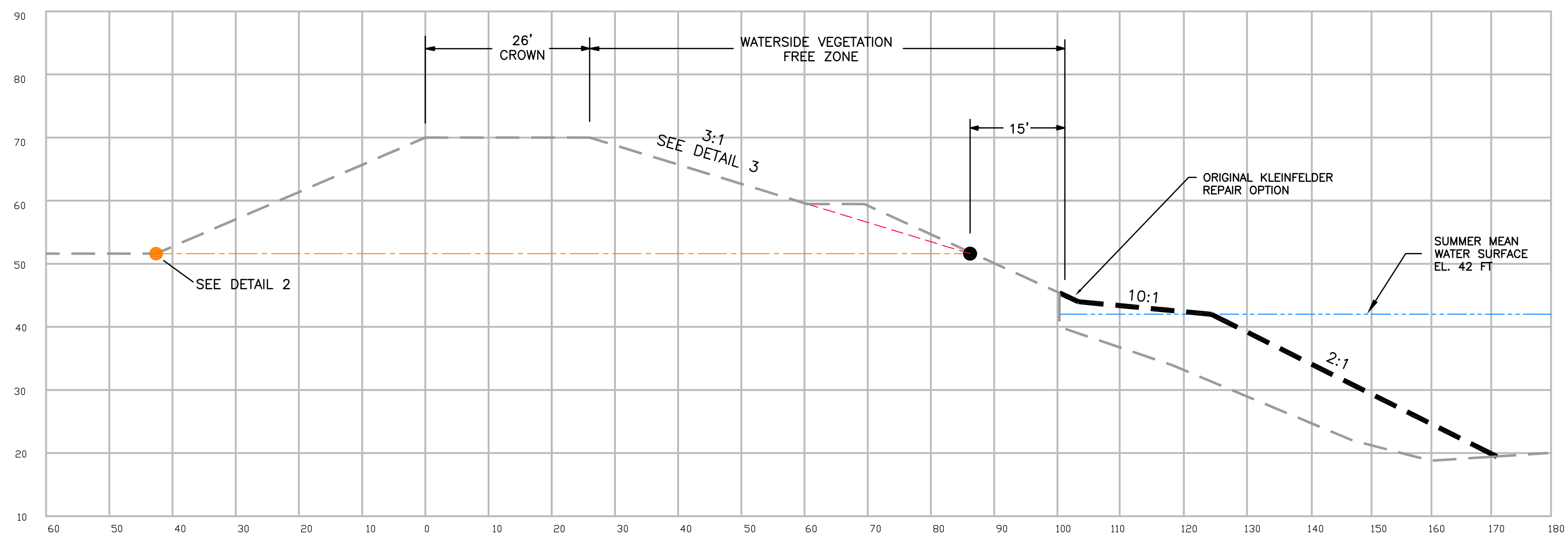
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 133.8L

Date	3/31/11
Figure	SAC 133.8L

DETAIL 1

SACRAMENTO RIVER 136.6L

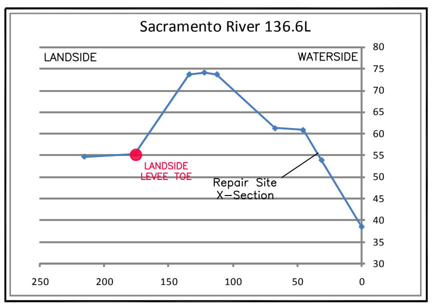


LEGEND
- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	38.6
31.1	53.9
45.6	60.8
67.3	61.2
112.5	73.6
122.3	74.0
134.0	73.6
176.6	55.3
215.7	54.7

KEY DATA FOR DETERMINING LANDSIDE TOE

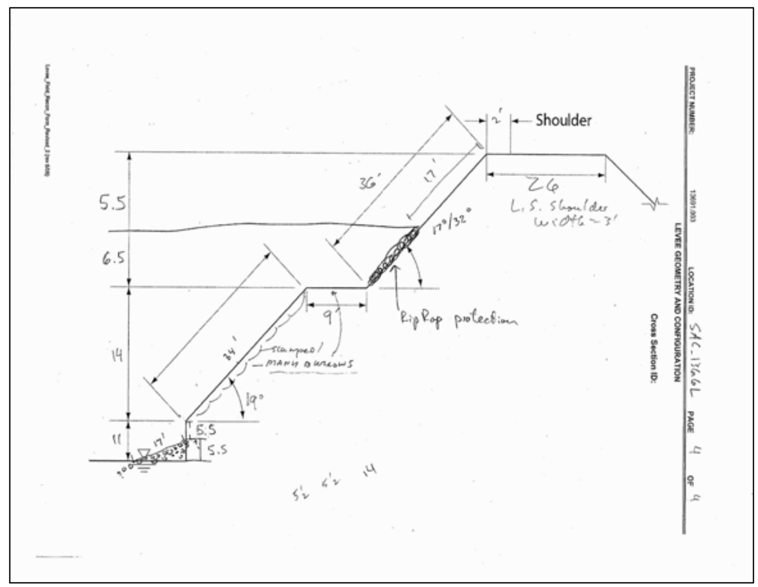
Comp Study Levee Crown Elevation	73.7
Comp Study Landside Toe Elevation	55.3
Comp Study Difference	18.4

Alternatives Report Analysis Levee Crown Elevation	70.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	51.6

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

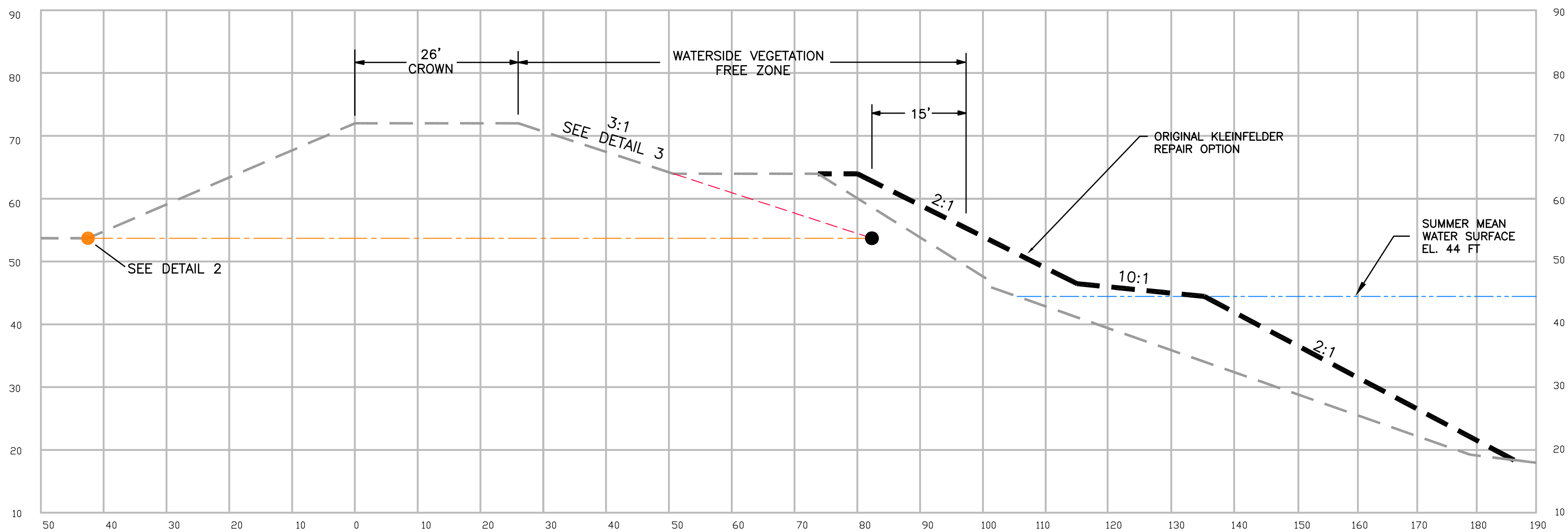
SACRAMENTO RIVER ~ 136.6L

Date 3/31/11

Figure SAC 136.6L

DETAIL 1

SACRAMENTO RIVER 138.1L



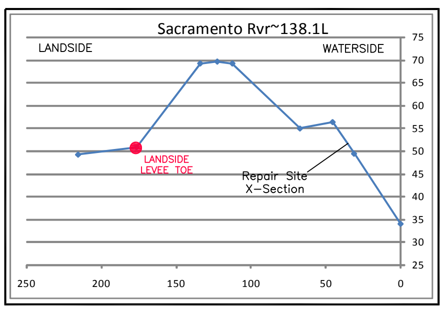
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	34.1
31.1	49.4
45.4	56.3
67.2	54.9
112.3	69.1
122.4	69.5
133.8	69.1
176.3	50.8
215.3	49.2

KEY DATA FOR DETERMINING
LANDSIDE TOE

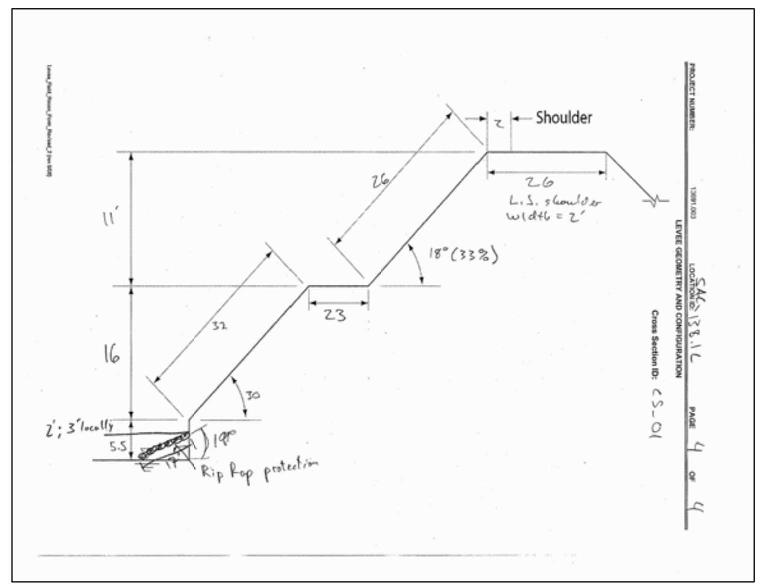
Comp Study Levee Crown Elevation	69.1
Comp Study Landside Toe Elevation	50.8
Comp Study Difference	18.3

Alternatives Report Analysis Levee Crown Elevation	72.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	53.7

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

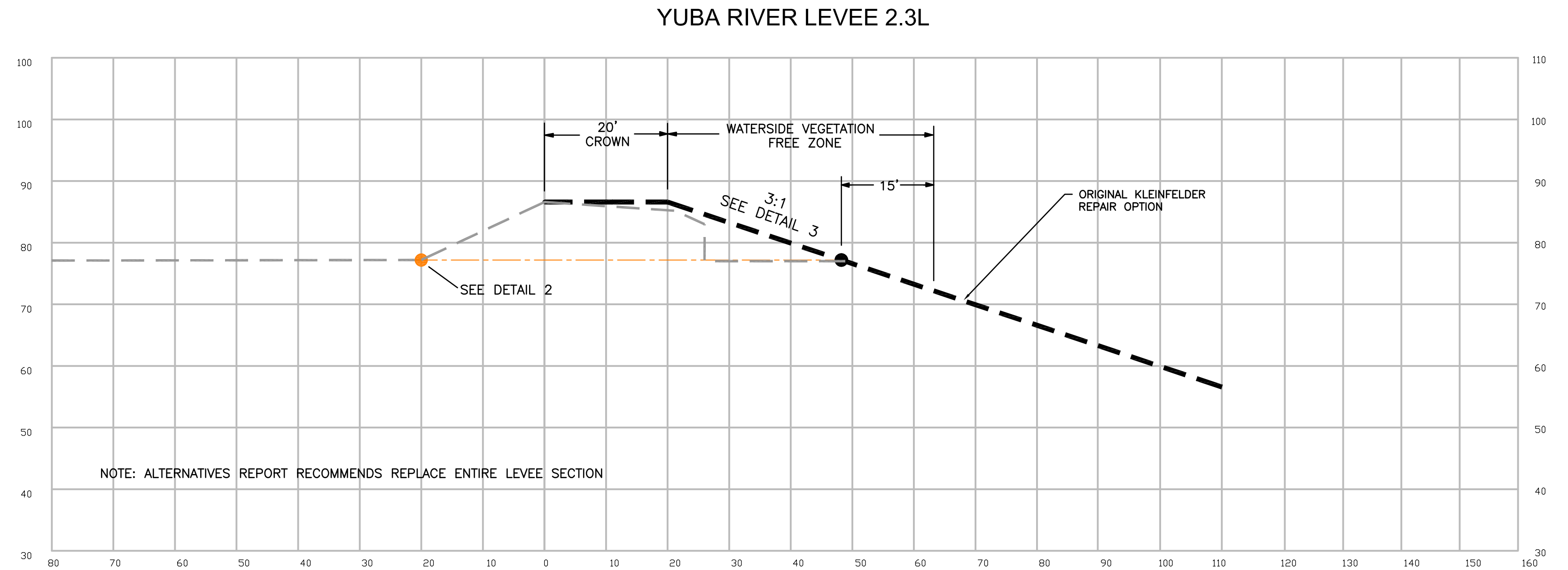
SACRAMENTO RIVER ~ 138.1L

Date 3/31/11
Figure SAC 138.1L

DETAIL 1

LEGEND

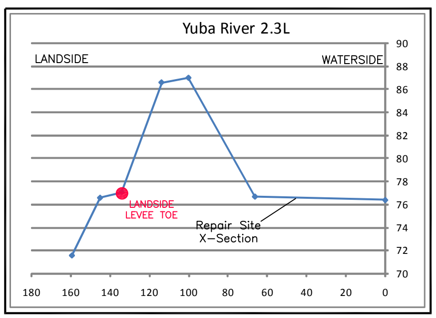
- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	76.4
66.2	76.7
100.0	87.0
113.7	86.6
133.9	77.1
144.9	76.6
159.2	71.6

KEY DATA FOR DETERMINING LANDSIDE TOE

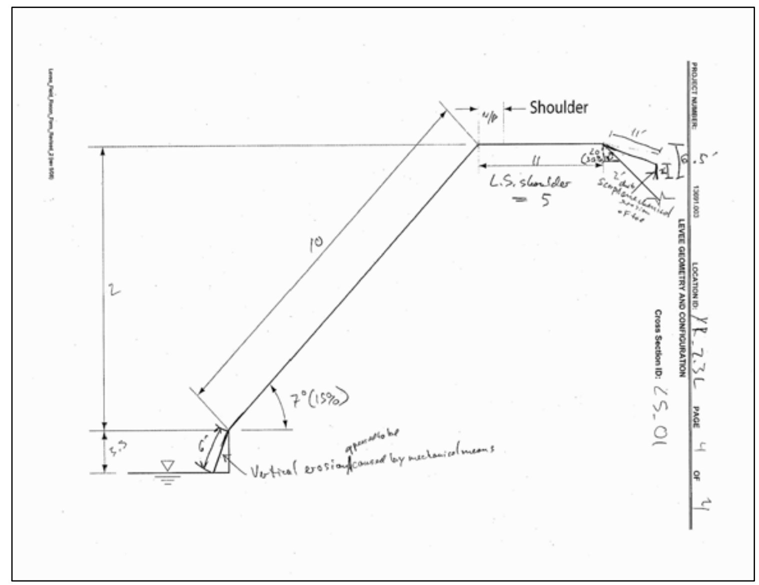
Comp Study Levee Crown Elevation	86.6
Comp Study Landside Toe Elevation	77.1
Comp Study Difference	9.5

Alternatives Report Analysis Levee Crown Elevation	87.5

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	78.0

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith
Checked By: R. Little
QA/QC Review: D. Teak

YUBA RIVER ~ 2.3L

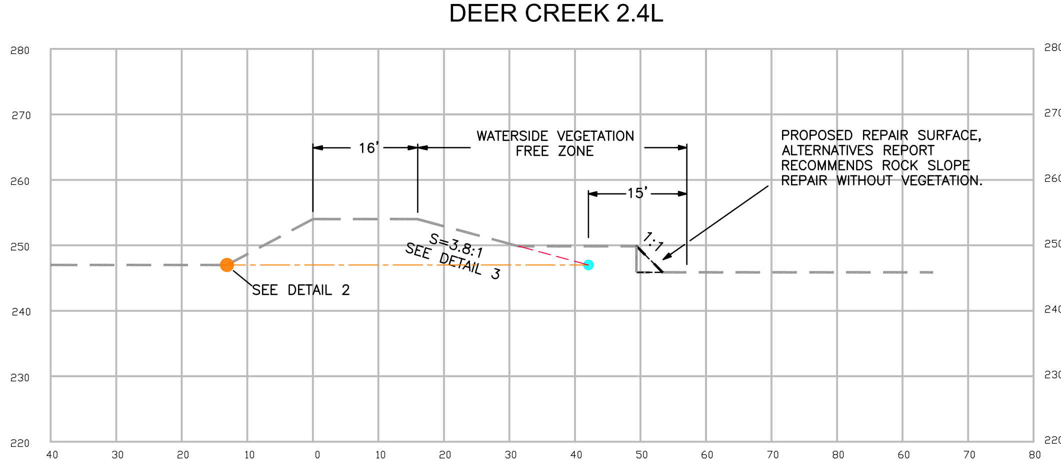
Date 3/31/11

Figure YR 2.3L

DETAIL 1

LEGEND

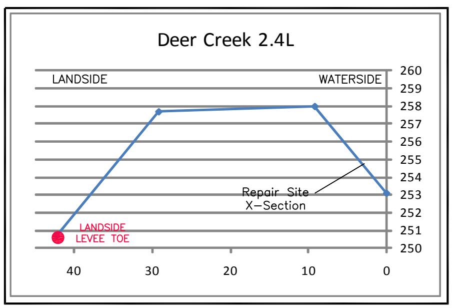
- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	253.1
9.2	258.0
29.2	257.7
42.3	250.7

KEY DATA FOR DETERMINING LANDSIDE TOE

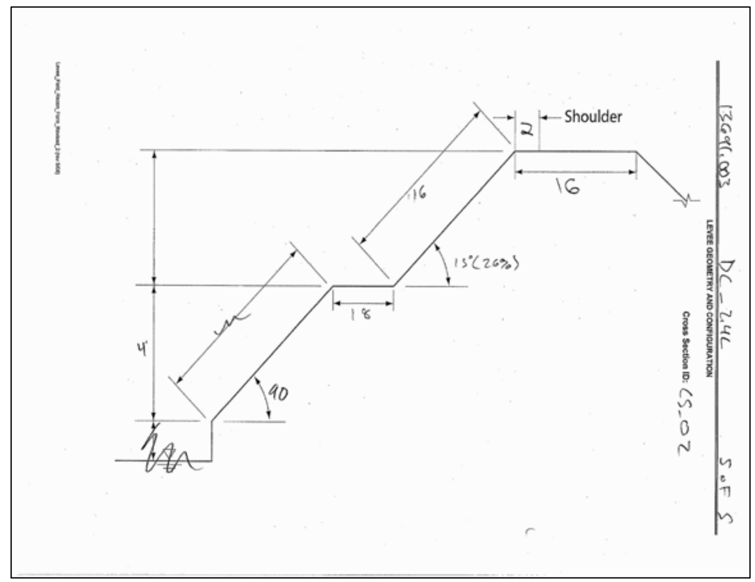
Comp Study Levee Crown Elevation	257.7
Comp Study Landside Toe Elevation	250.7
Comp Study Difference	7.0

Alternatives Report Analysis Levee Crown Elevation	254.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	247.0

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

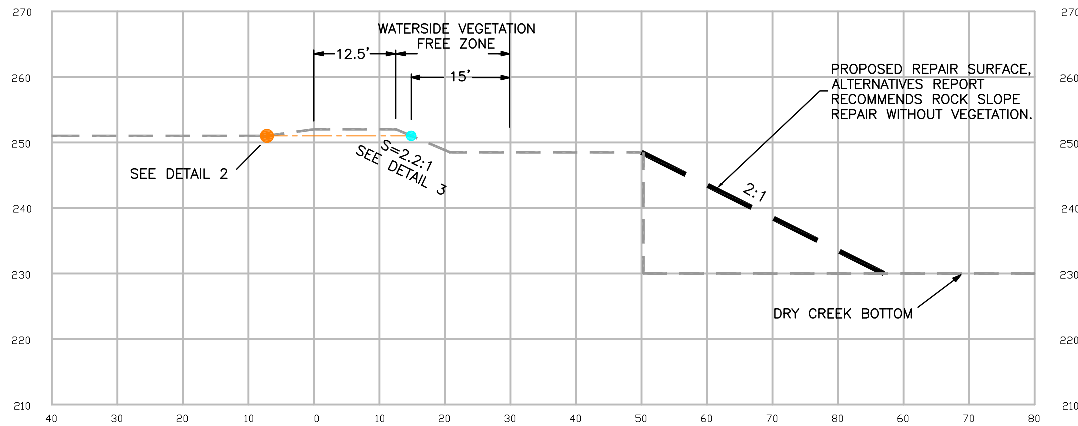
Drawn By: E. Smith		DEER CREEK LEVEE ~ 2.4L	Date	3/31/11
Checked By: R. Little			Figure	DC 2.4L
QA/QC Review: D. Teak				

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

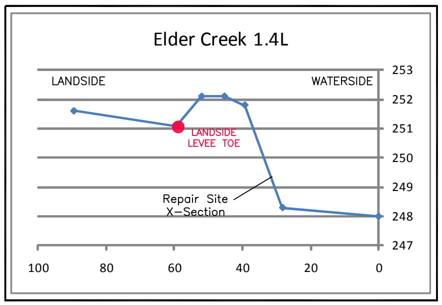
ELDER CREEK 1.44L



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



KEY DATA FOR DETERMINING LANDSIDE TOE

Comp Study Levee Crown Elevation	252.1
Comp Study Landside Toe Elevation	251.1
Comp Study Difference	1.0

Alternatives Report Analysis Levee Crown Elevation	252.0

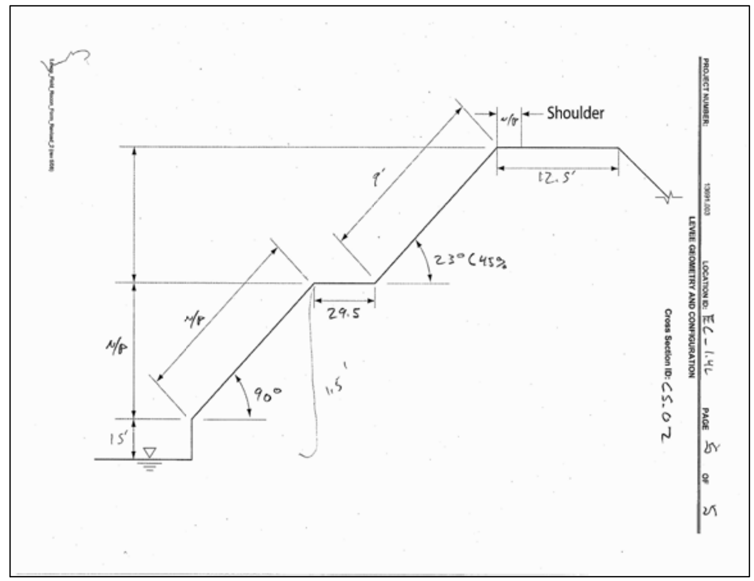
Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	251.0

Comp Study Data Points

Horizontal Distance to Data Point	Elevation at Data Point
0.0	248.0
28.2	248.3
39.3	251.8
45.3	252.1
52.0	252.1
59.3	251.1
89.6	251.6

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

ELDER CREEK ~ 1.4L

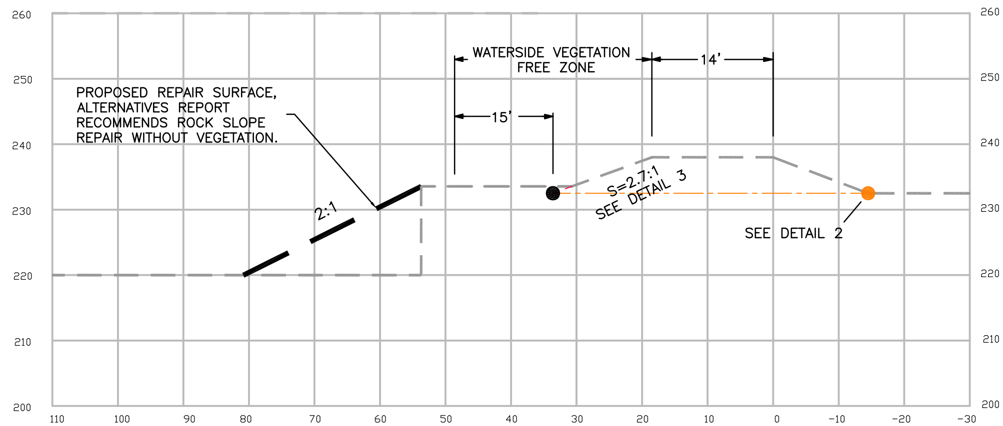
Date	3/31/11
Figure	EC 1.4L

DETAIL 1

LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

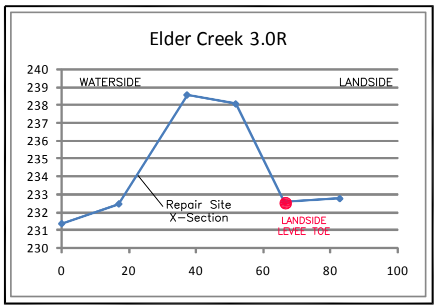
ELDER CREEK 3.0R



DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	231.4
17.0	232.5
37.3	238.6
51.8	238.1
66.3	232.6
82.8	232.8

KEY DATA FOR DETERMINING LANDSIDE TOE

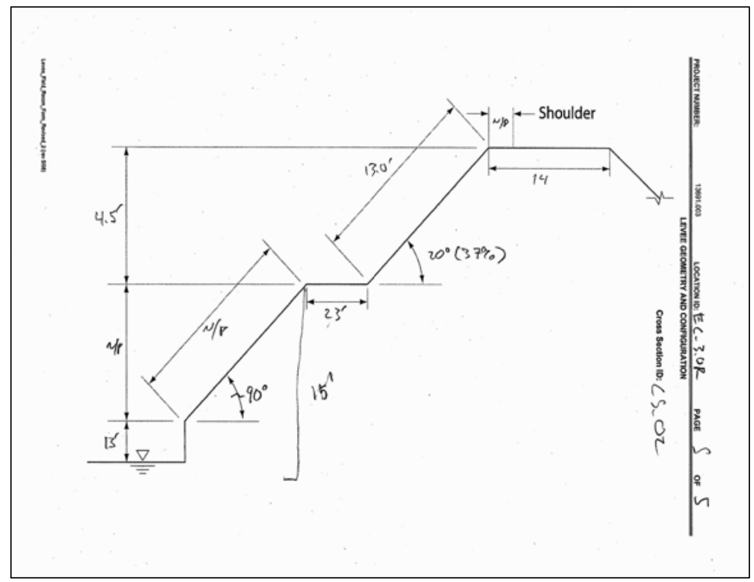
Comp Study Levee Crown Elevation	238.1
Comp Study Landside Toe Elevation	232.6
Comp Study Difference	5.5

Alternatives Report Analysis Levee Crown Elevation	238.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	232.5

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

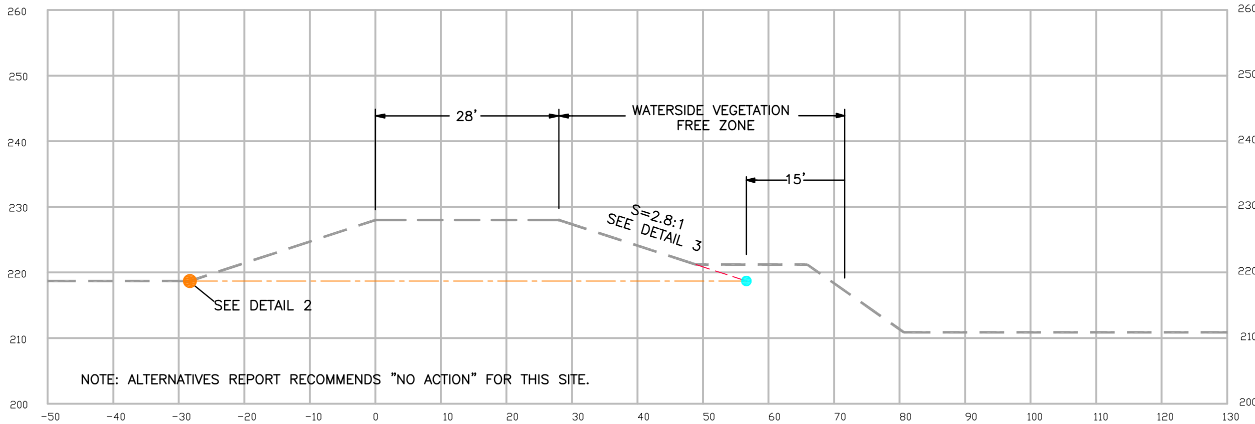
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

ELDER CREEK ~ 3.0R

Date	3/31/11
Figure	EC 3.0R

DETAIL 1

ELDER CREEK 4.1L



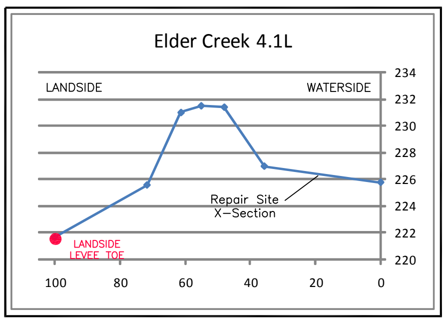
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	225.8
35.8	227.0
48.1	231.4
55.2	231.5
61.4	231.0
71.9	225.6
100.3	221.7

KEY DATA FOR DETERMINING LANDSIDE TOE

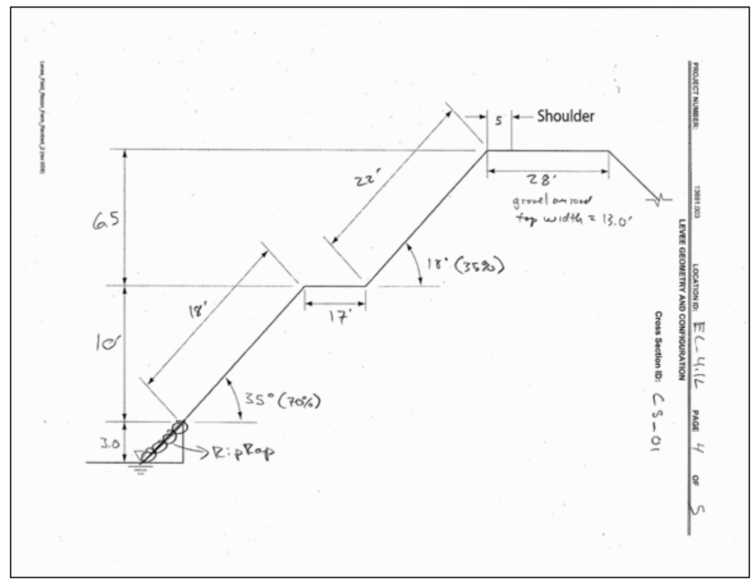
Comp Study Levee Crown Elevation	231.0
Comp Study Landside Toe Elevation	221.7
Comp Study Difference	9.3

Alternatives Report Analysis Levee Crown Elevation	228.0

Landside Toe Calculated by: (Alt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	218.7

DETAIL 3

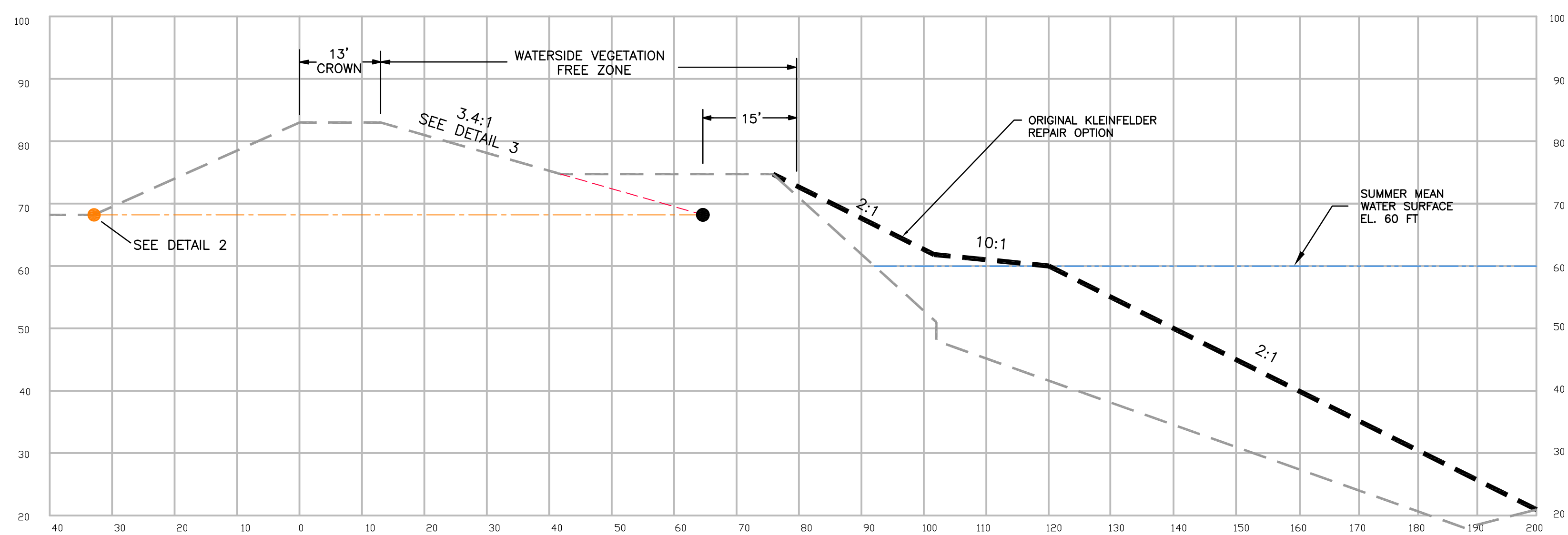
ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project Phase II Post Authorization Change Programmatic Plan Framework Memorandum	HDR Engineering, Inc.	Drawn By: E. Smith Checked By: R. Little QA/QC Review: D. Teak	ELDER CREEK ~ 4.1L	Date 3/31/11 Figure EC 4.1L

DETAIL 1

SACRAMENTO RIVER 152.8L



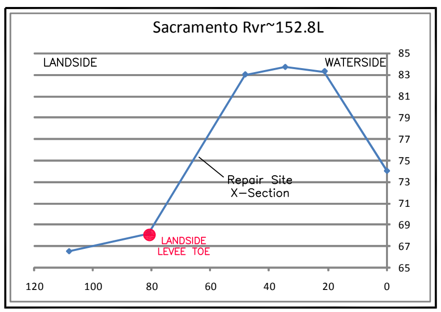
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	74.0
21.2	83.3
34.6	83.7
48.2	83.0
81.1	68.2
108.1	66.5

KEY DATA FOR DETERMINING LANDSIDE TOE

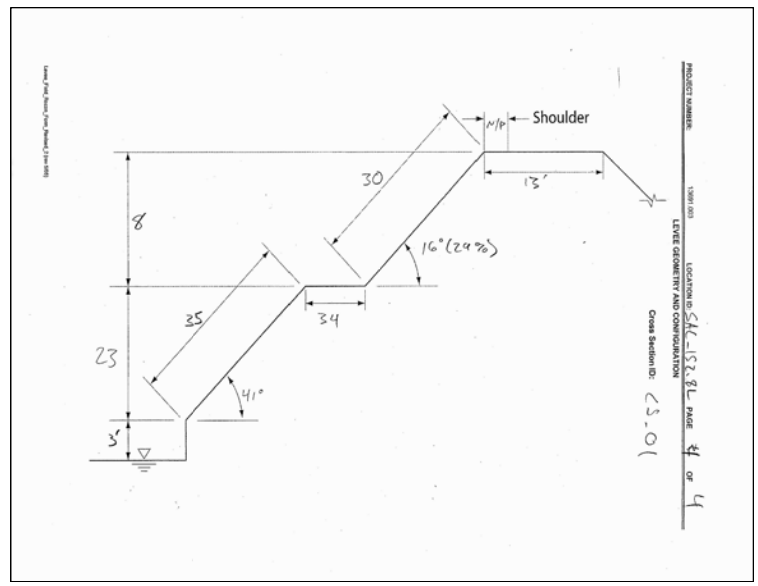
Comp Study Levee Crown Elevation	83.0
Comp Study Landside Toe Elevation	68.2
Comp Study Difference	14.8

Alternatives Report Analysis Levee Crown Elevation	83.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	68.2

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

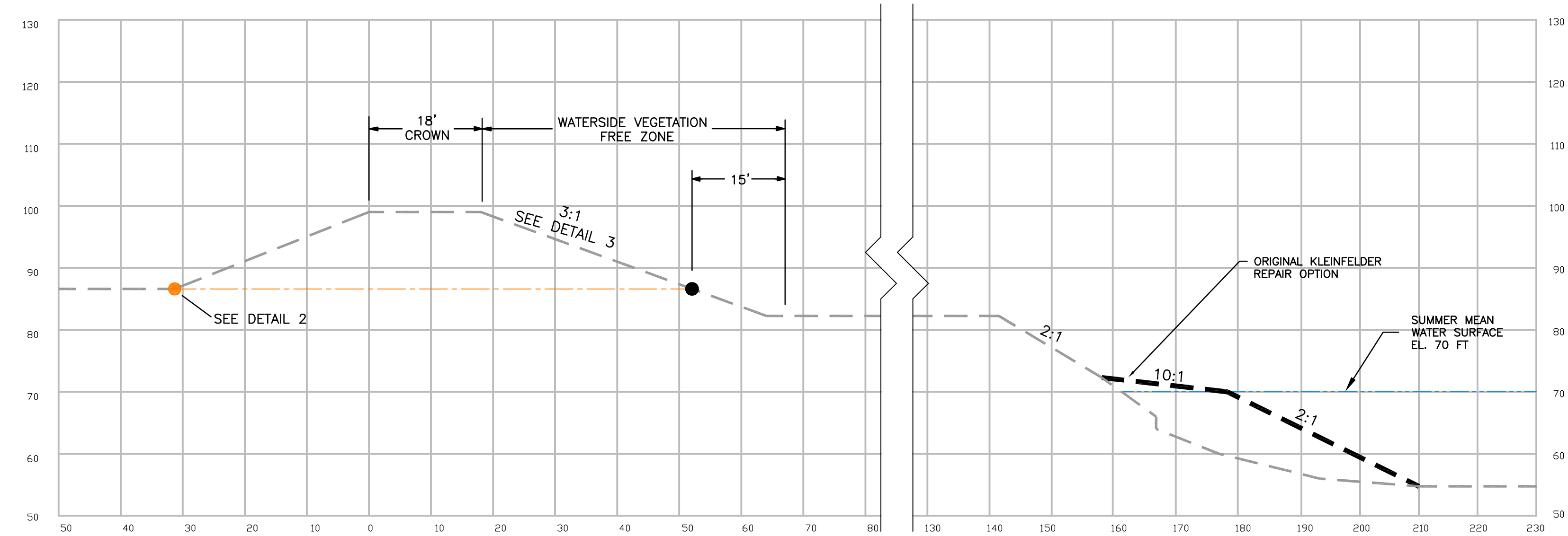
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 152.8L

Date	3/31/11
Figure	SAC 152.8L

DETAIL 1

SACRAMENTO RIVER 163.0L

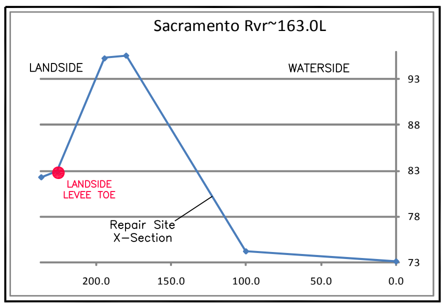


LEGEND
- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM
DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	73.1
100.2	74.2
180.2	95.6
194.6	95.3
225.9	82.9
236.8	82.3

KEY DATA FOR DETERMINING
LANDSIDE TOE

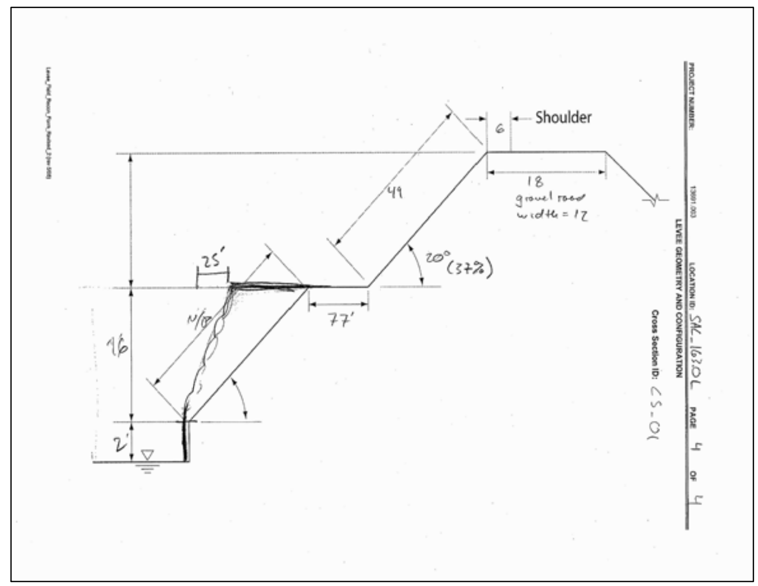
Comp Study Levee Crown Elevation	95.3
Comp Study Landside Toe Elevation	82.9
Comp Study Difference	12.4

Alternatives Report Analysis Levee Crown Elevation	99.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	86.6

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

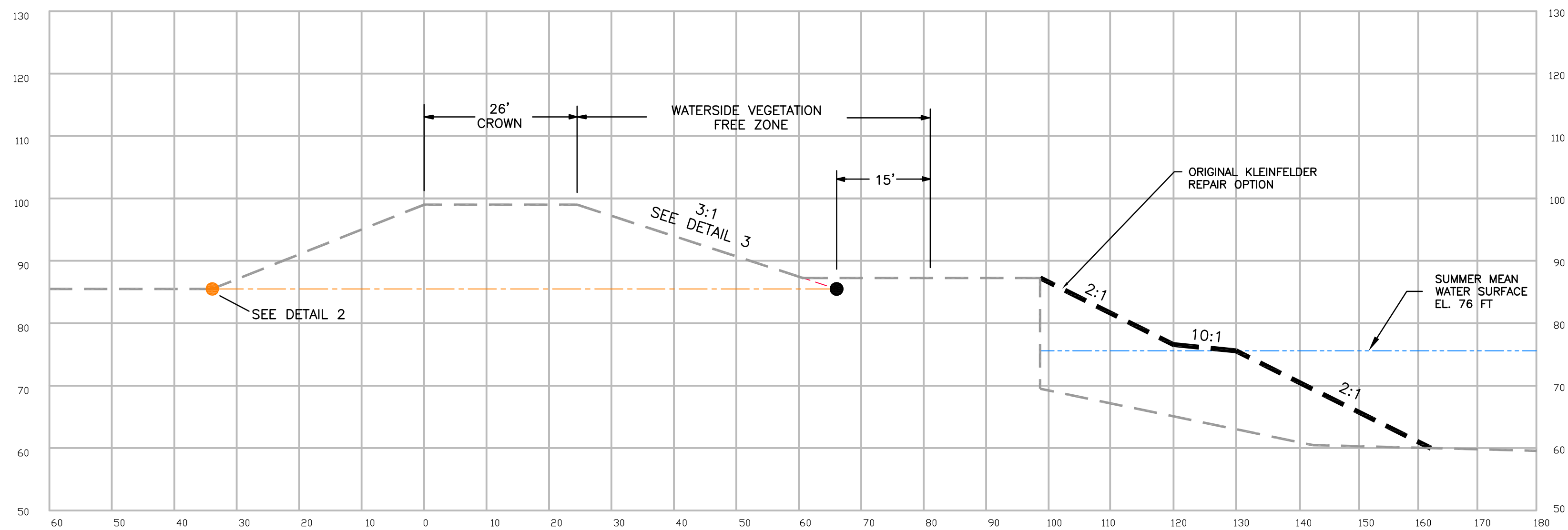
Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 163.0L

Date	3/31/11
Figure	SAC 163.0L

DETAIL 1

SACRAMENTO RIVER 168.3L



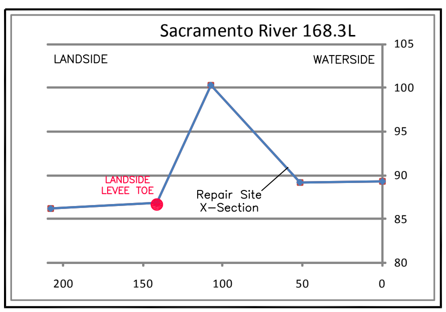
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	89.3
51.7	89.2
55.8	100.3
33.9	86.8
66.3	86.2

KEY DATA FOR DETERMINING LANDSIDE TOE

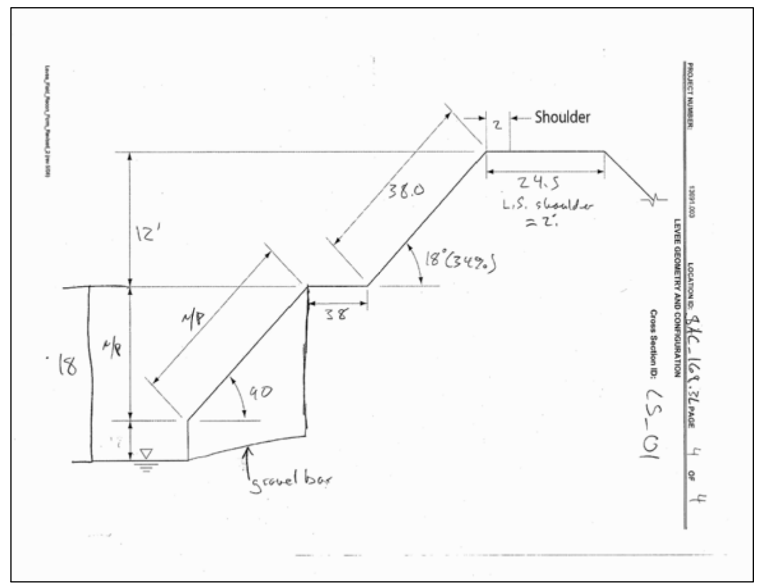
Comp Study Levee Crown Elevation	100.3
Comp Study Landside Toe Elevation	86.8
Comp Study Difference	13.5

Alternatives Report Analysis Levee Crown Elevation	99.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	85.5

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



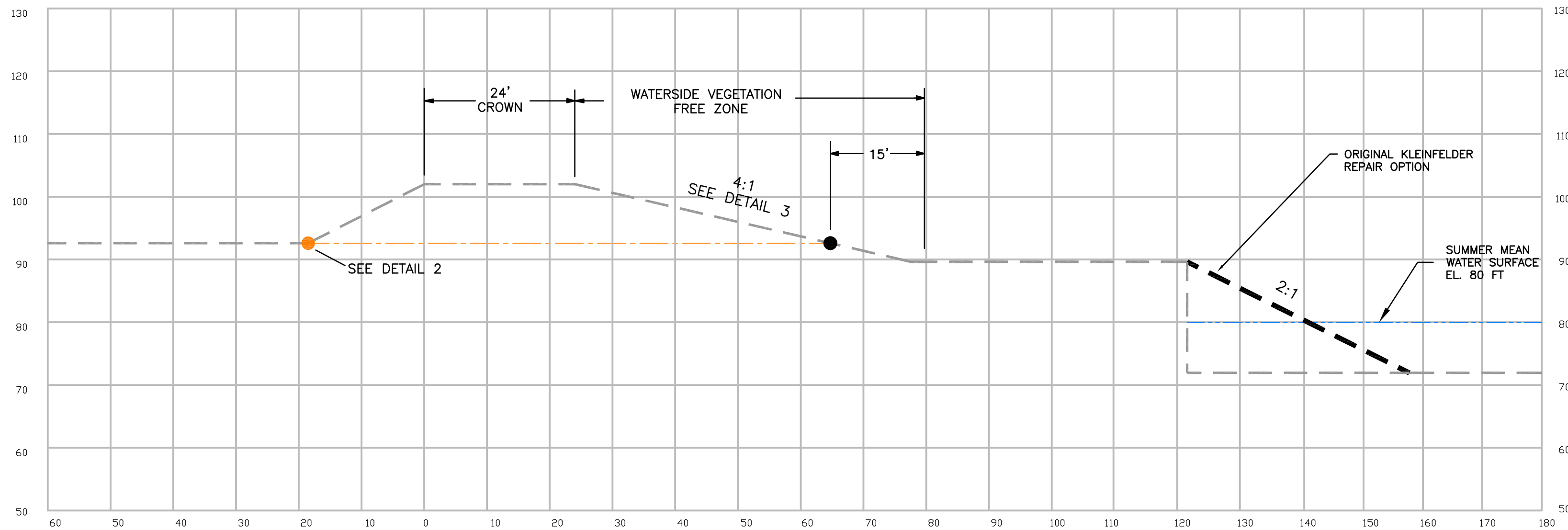
Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By: E. Smith	SACRAMENTO RIVER ~ 168.3L	Date	3/31/11
Checked By: R. Little		Figure	SAC 168.3L
QA/QC Review: D. Teak			

DETAIL 1

SACRAMENTO RIVER 172.0L



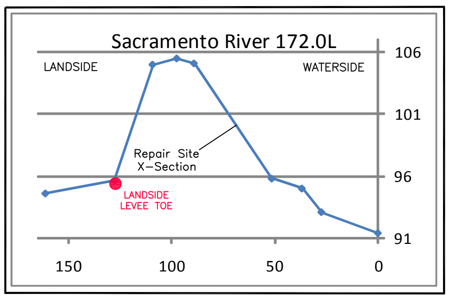
LEGEND

- EXISTING LEVEE SURFACE
- ALTERNATIVES REPORT PROPOSED REPAIR SURFACE
- PROJECTED LANDSIDE LEVEE TOE
- WATER SURFACE
- PROJECTED WATERSIDE SLOPE

DETAIL 2

ESTABLISH LANDSIDE TOE

CROSS-SECTION FROM DWR COMPREHENSIVE STUDY



Comp Study Data Points	
Horizontal Distance to Data Point	Elevation at Data Point
0.0	91.4
27.4	93.1
36.8	95.0
51.4	95.8
89.0	105.1
97.3	105.5
109.0	105.0
127.5	95.6
160.8	94.6

KEY DATA FOR DETERMINING LANDSIDE TOE

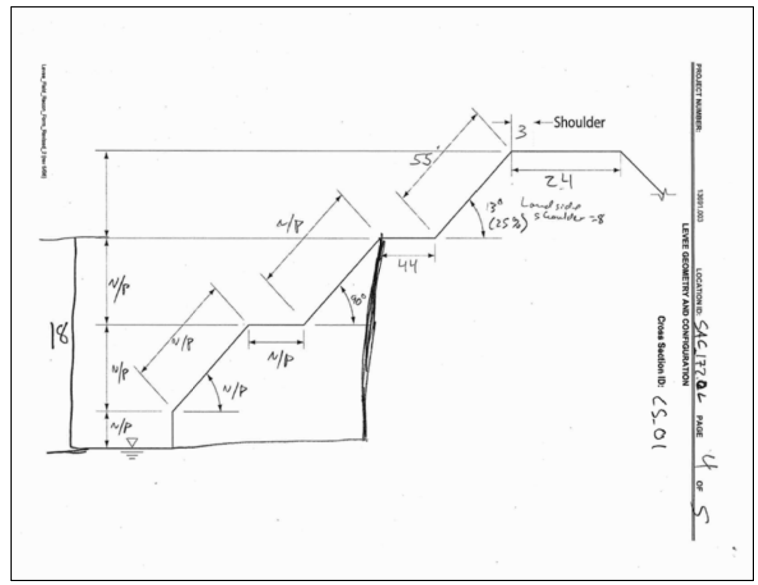
Comp Study Levee Crown Elevation	105.0
Comp Study Landside Toe Elevation	95.6
Comp Study Difference	9.4

Alternatives Report Analysis Levee Crown Elevation	102.0

Landside Toe Calculated by: (ALt. Report Crown Elev.) - (Comp Study Difference)	
Determined Landside Toe	92.6

DETAIL 3

ALTERNATIVES REPORT (80,000LF) FIELD NOTES



Sacramento River Bank Protection Project
Phase II Post Authorization Change
Programmatic Plan Framework Memorandum

HDR Engineering, Inc.

Drawn By:	E. Smith
Checked By:	R. Little
QA/QC Review:	D. Teak

SACRAMENTO RIVER ~ 172.0L

Date	3/31/11
Figure	SAC 172.0L

SACRAMENTO RIVER BANK PROTECTION PROJECT

Hydraulics Evaluation Technical Memo ***March, 2011***

The purpose of this technical memorandum (TM) is to provide hydraulics information for the Sacramento River Bank Protection Project. No hydraulic modeling was requested at this time; therefore, best available hydraulic modeling information from the US Army Corps of Engineers (USACE) Comprehensive Study dated 2002 was used for this evaluation. HEC-RAS models for the Sacramento and San Joaquin Rivers, obtained by conversion from the Comprehensive Study UNET models, were provided to HDR by the California Department of Water Resources (DWR) for use with the Central Valley Floodplain Evaluation and Delineation project. The Sacramento River HEC-RAS model based on the NGVD 29 vertical datum was used to obtain hydraulic modeling information from approximately half a mile upstream to approximately half a mile downstream of each of the following river mile (RM) locations:

- Cache Slough - RM 15.9, RN 23.6
- Georgiana Slough - RM 3.6, RM 3.7, RM 4.0
- Sacramento River - RM 21.5, RM 22.5, RM 22.7, 23.2

Table 1 provides the HEC-RAS stationing information and variations in water surface elevations and channel velocities for the 100-year storm event at these locations obtained from the Comprehensive Study.

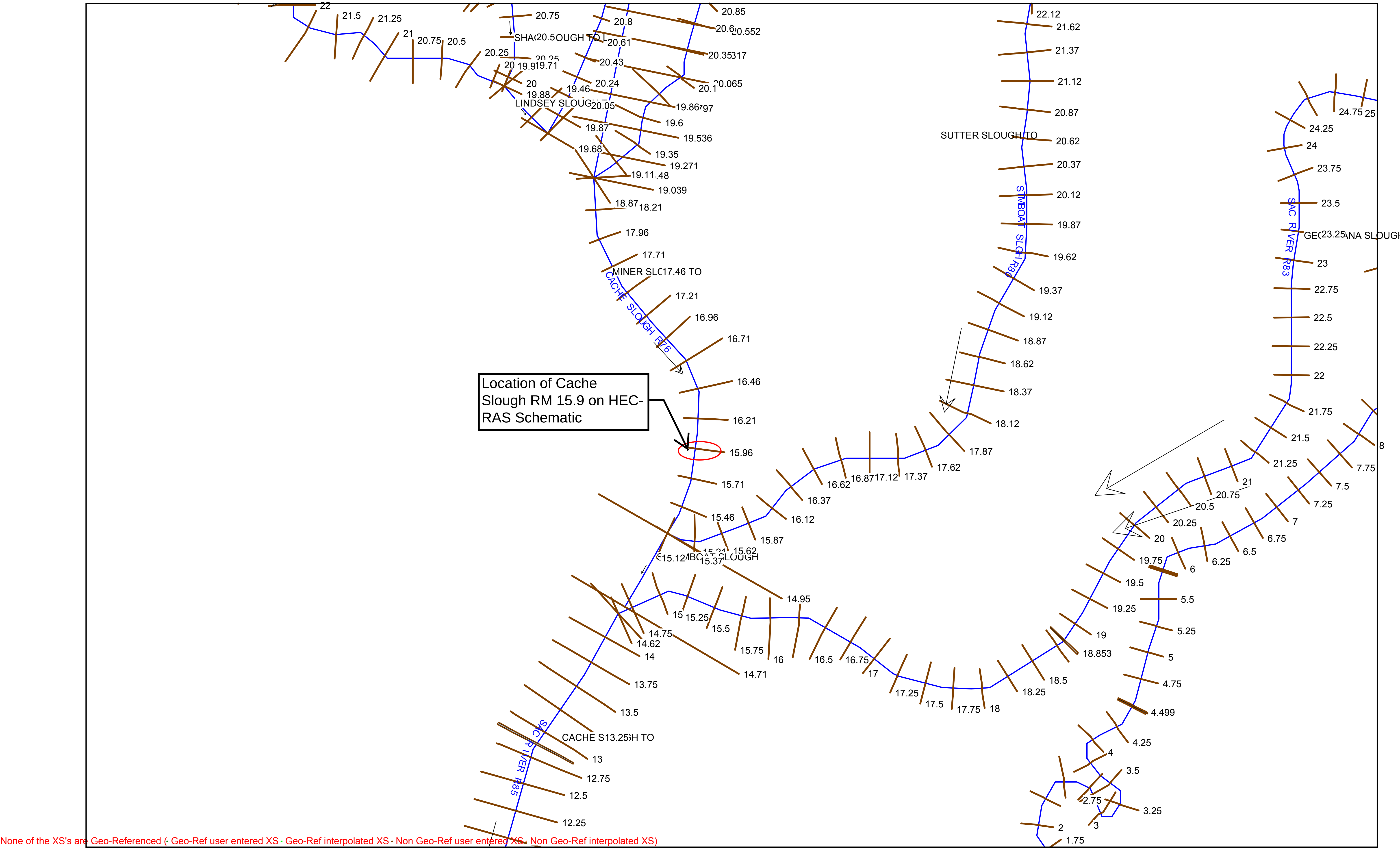
Table 1: Hydraulic Modeling Information from the 2002 Comprehensive Study

River Mile Location	Start and End HEC-RAS Stationing for Reach*	Variation in Water Surface Elevation for Reach (ft, NGVD 29)	Variation in Channel Velocity for Reach (ft/s)
Cache Slough RM 15.9	RM 15.46 - RM 16.46	13.2 to 16.0	6 to 11
Cache Slough RM 23.6	RM 23 - RM 24.25	21.3**	0.04 to 0.35
Georgiana Slough RM 3.6	RM 3.0 to RM 4.0	9.8 to 10.8	3.5 to 4.3
Georgiana Slough RM 3.7	RM 3.25 to RM 4.25	10.0 to 11.0	3.5 to 4.3
Georgiana Slough RM 4.0	RM 3.5 to RM 4.499	10.2 to 11.2	3.5 to 4.3
Sacramento River RM 21.5	RM 21.0 to 22.0	14.7 to 15.1	4.2 to 4.6
Sacramento River RM 22.5	RM 22.0 to 23.0	15.1 to 15.5	4.2 to 4.7
Sacramento River RM 22.7	RM 22.25 to RM 23.25	15.2 to 15.6	4.1 to 4.7
Sacramento River RM 23.2	RM 22.75 to RM 23.75	15.4 to 15.8	4.1 to 4.7

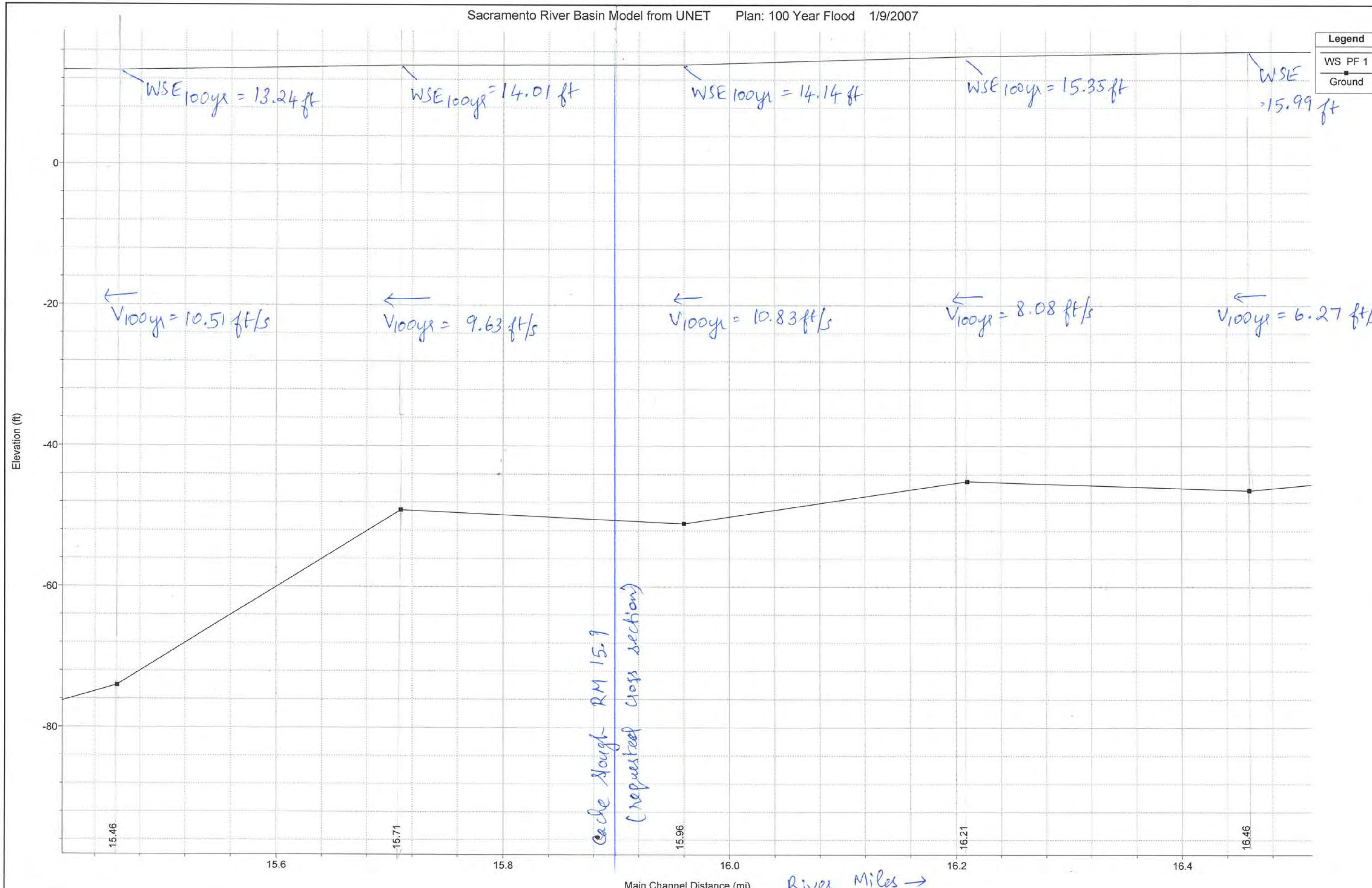
Notes: *The reach considered for each river mile location extends from approximately half a mile upstream to approximately half a mile downstream of that location.

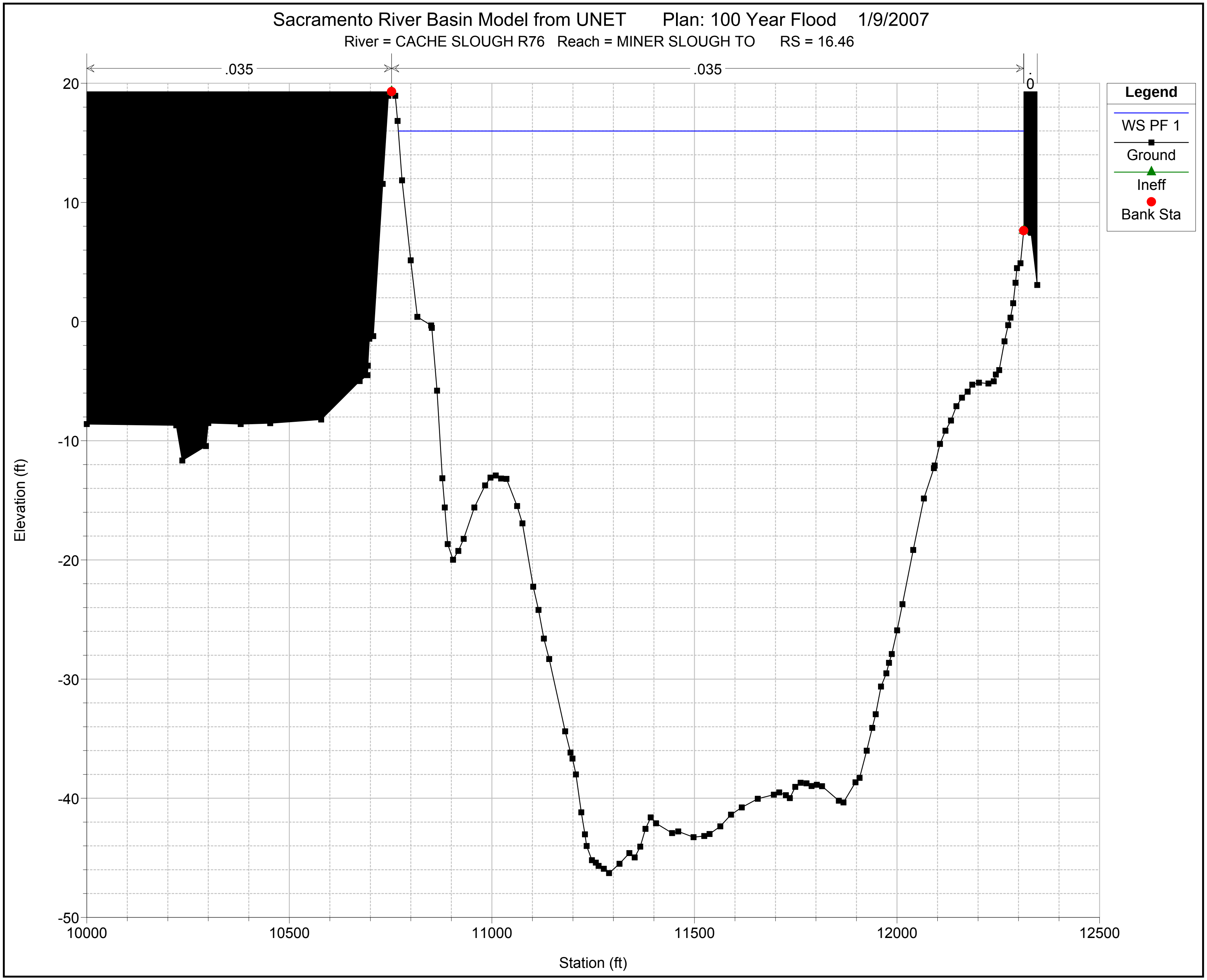
**For the reach from RM 23 to RM 24.25, the channel does not have adequate capacity to contain the 100-year flood.

The attachments to this TM include the following information for each of the RM locations: HEC-RAS schematic showing the location, output profile figures from HEC-RAS for each reach, and HEC-RAS plots for all cross sections within that reach. Water surface elevations and channel velocities for the 100-year storm event at each HEC-RAS cross section are provided in the output profile figures.

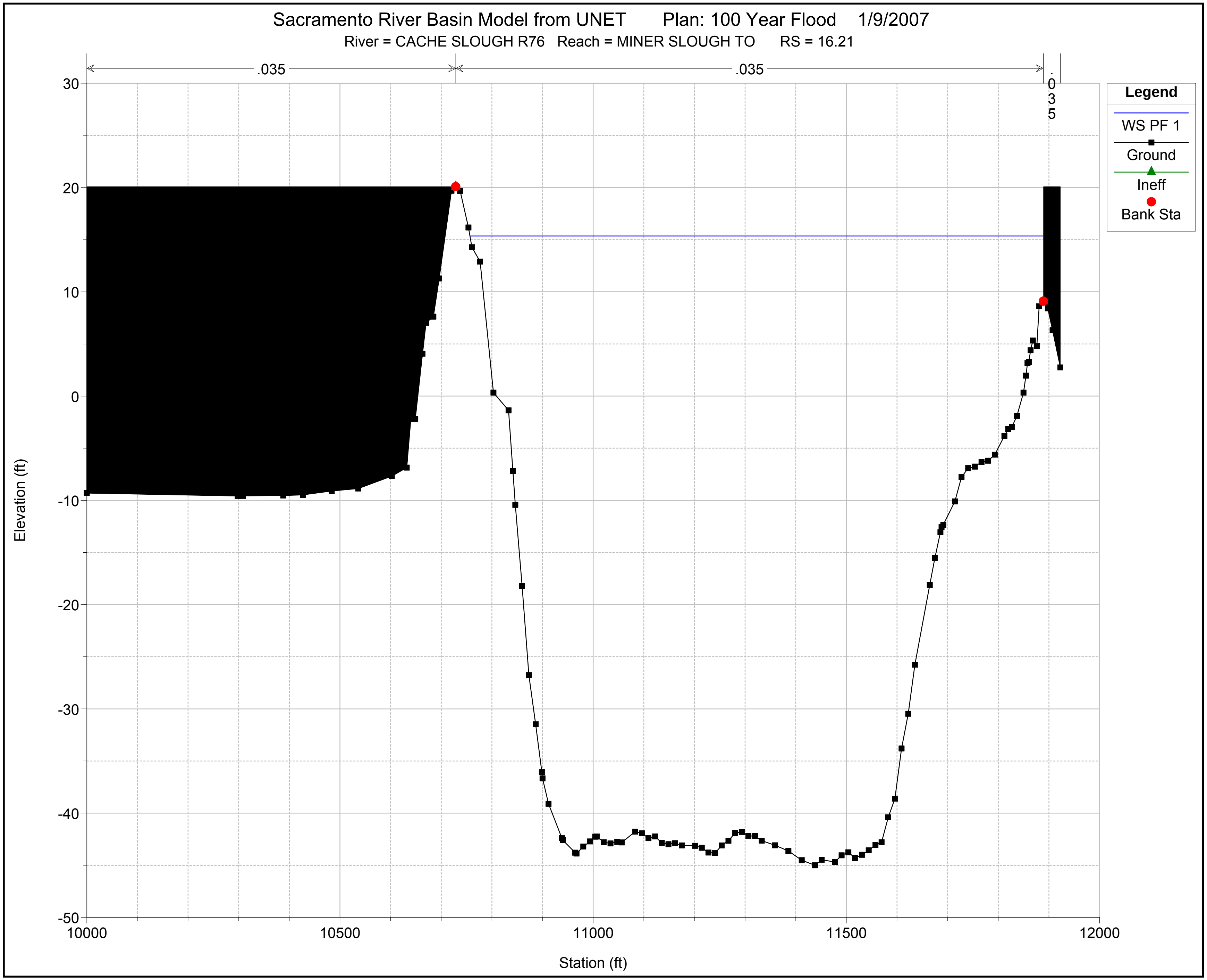

Location of Cache Slough RM 15.9 on HEC-RAS Schematic
15.96
CACHE SLOUGH R76
STMBOAT SLGH R80
SAC RIVER R83
SAC RIVER R85
SUTTER SLOUGH TO
LINDSEY SLOUG
MINER SLC17.46 TO
CACHE S13.25GH TO
None of the XS's are Geo-Referenced (· Geo-Ref user entered XS · Geo-Ref interpolated XS · Non Geo-Ref user entered XS · Non Geo-Ref interpolated XS)

HEC-RAS Profile Drawing for Cache Slough RM 15.9

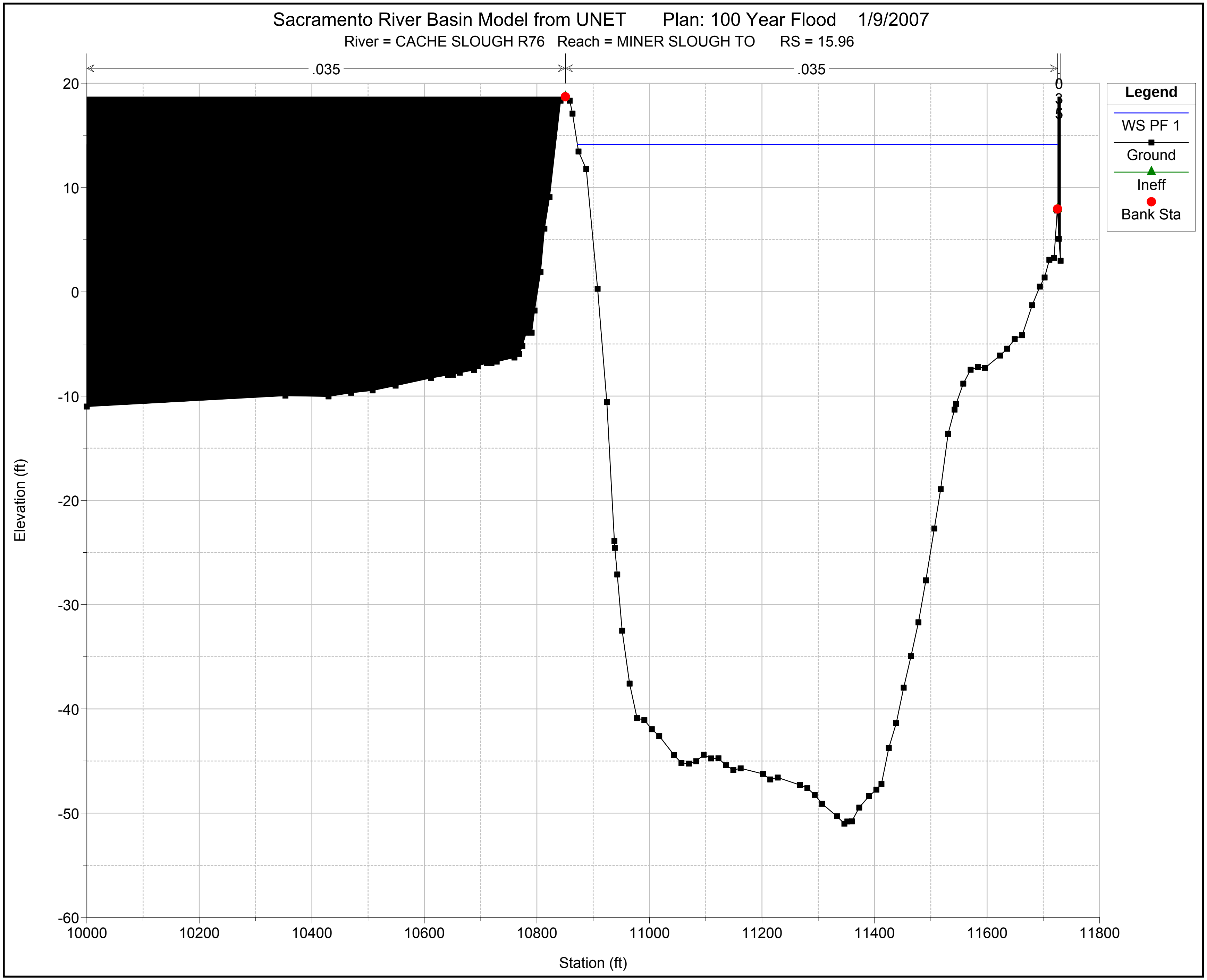




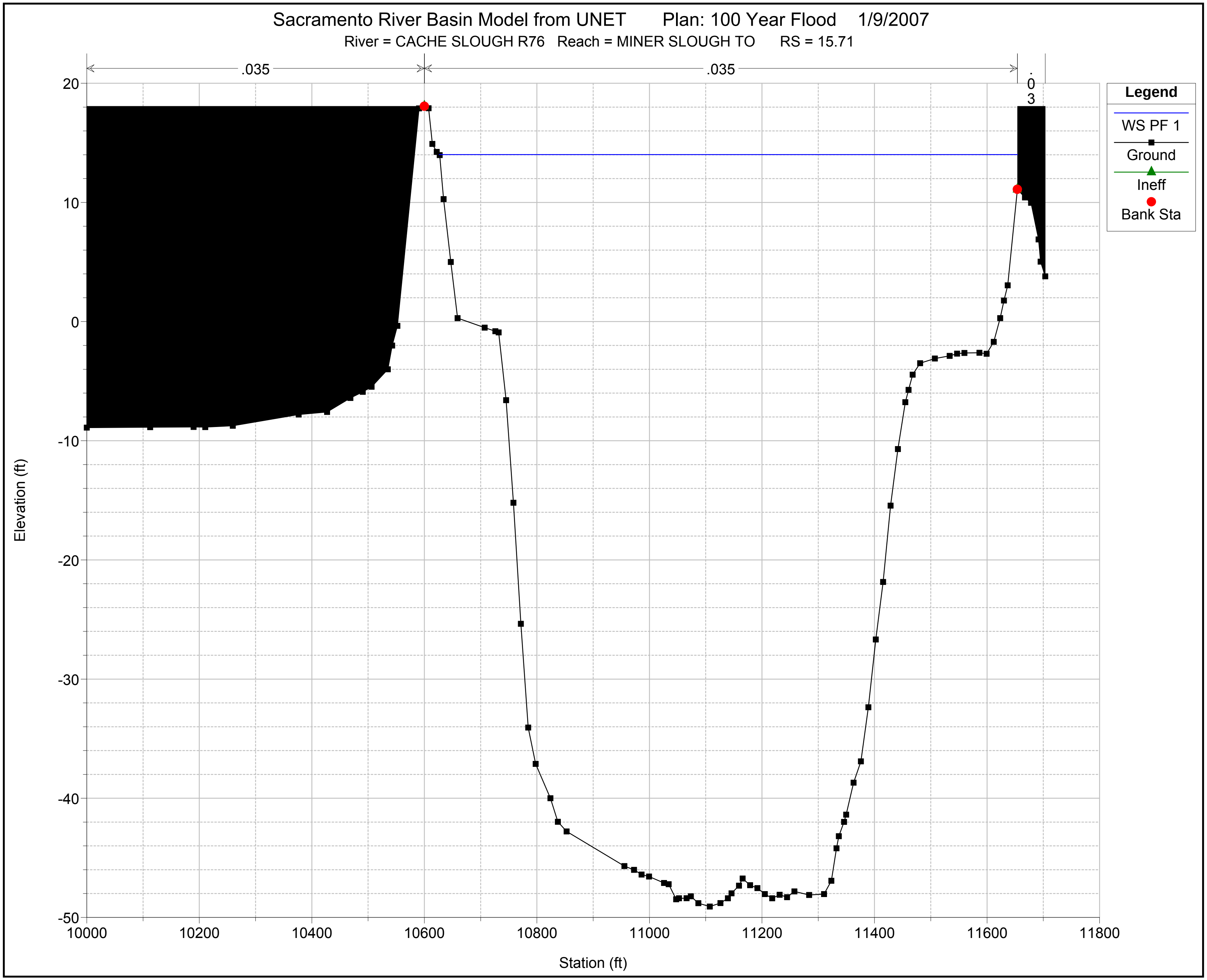
Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = CACHE SLOUGH R76 Reach = MINER SLOUGH TO RS = 16.46
.035
.035
.0
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
20
10
0
-10
-20
-30
-40
-50
10000
10500
11000
11500
12000
12500

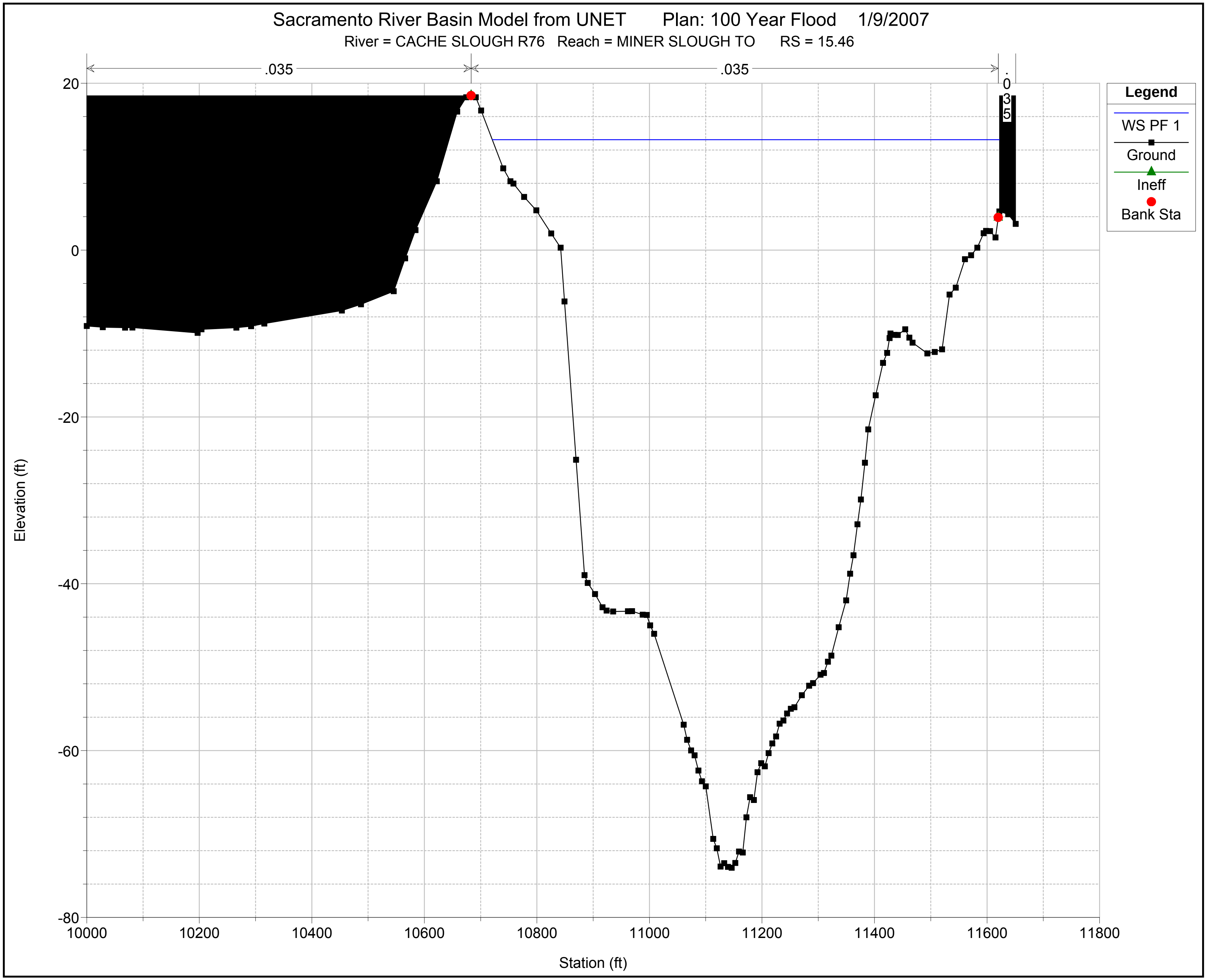


Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = CACHE SLOUGH R76 Reach = MINER SLOUGH TO RS = 16.21
.035
.035
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
30
20
10
0
-10
-20
-30
-40
-50
10000
10500
11000
11500
12000
Station (ft)

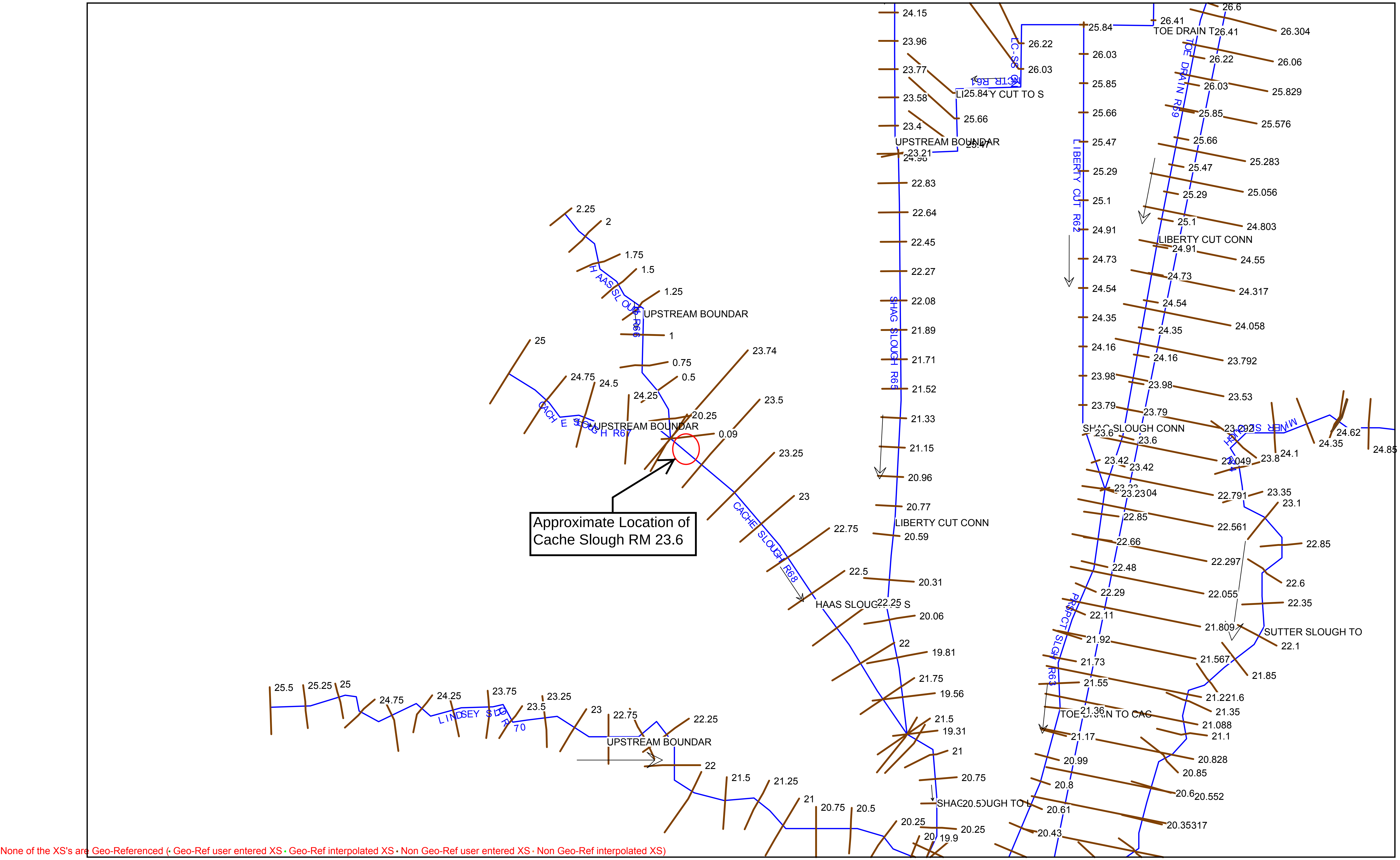


Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = CACHE SLOUGH R76 Reach = MINER SLOUGH TO RS = 15.96
.035
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
20
10
0
-10
-20
-30
-40
-50
-60
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800

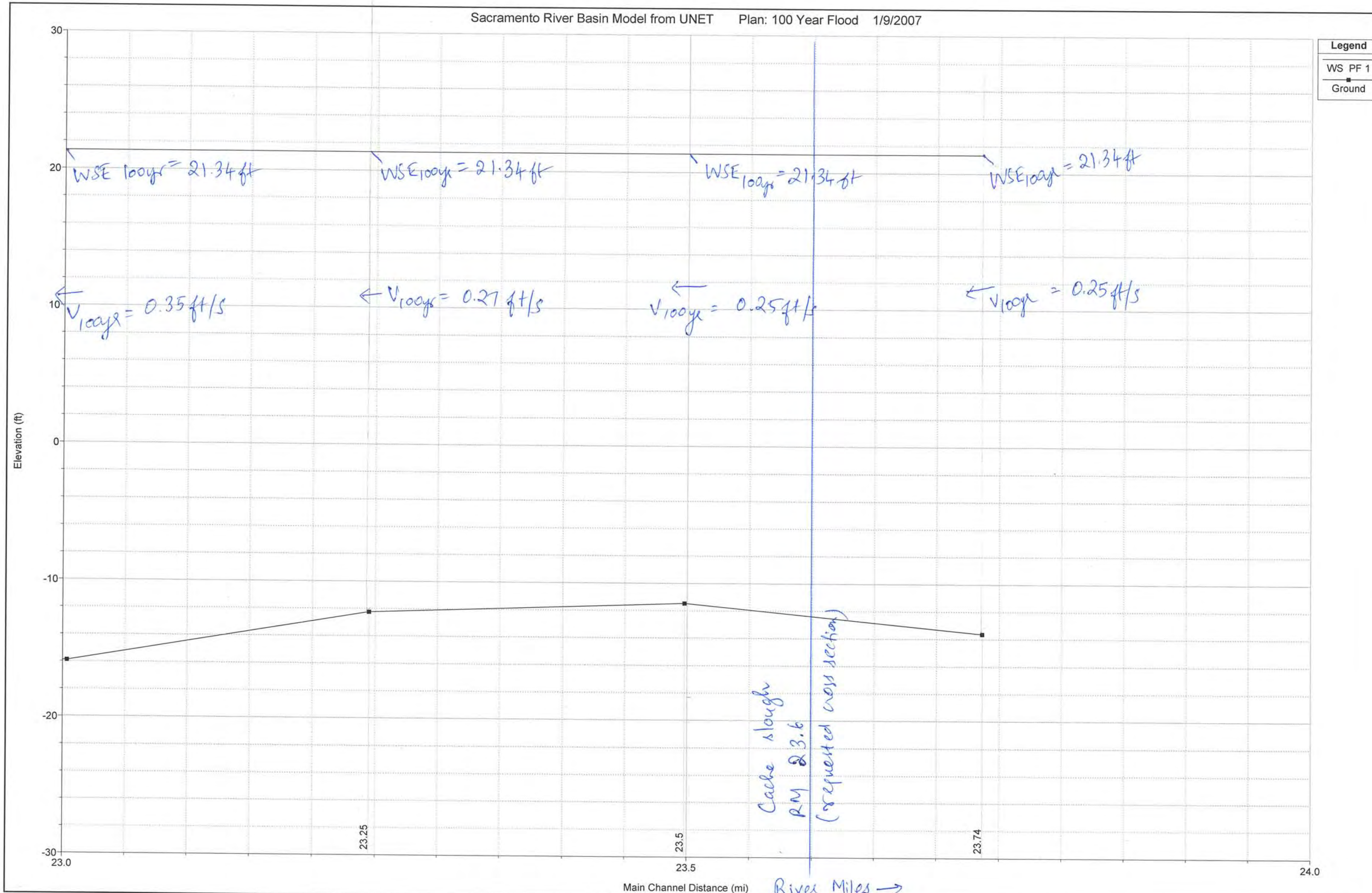

Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = CACHE SLOUGH R76 Reach = MINER SLOUGH TO RS = 15.71
.035
.035
.03
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
20
10
0
-10
-20
-30
-40
-50
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800



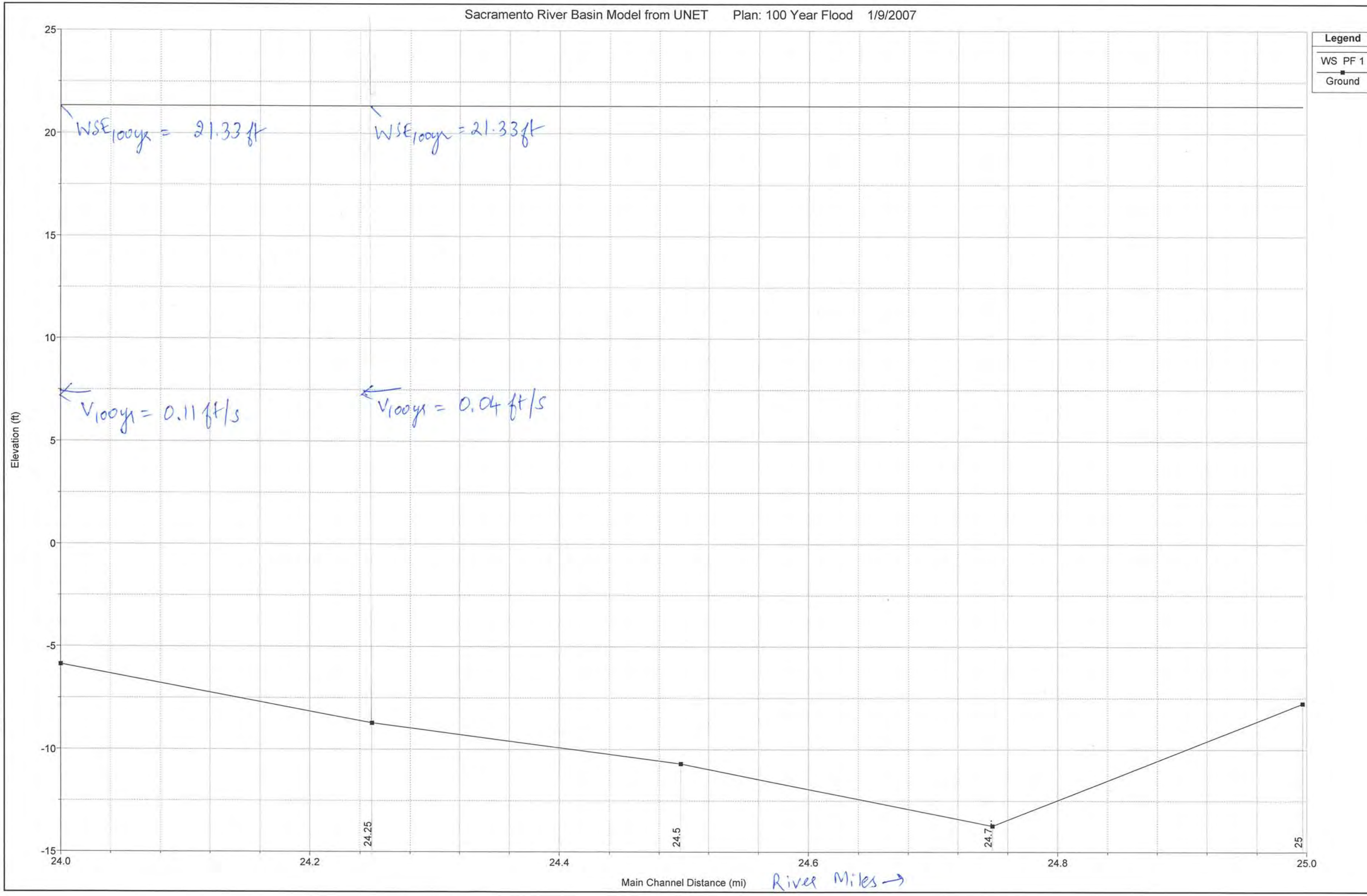
Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = CACHE SLOUGH R76 Reach = MINER SLOUGH TO RS = 15.46
.035
.035
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
20
0
-20
-40
-60
-80
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800

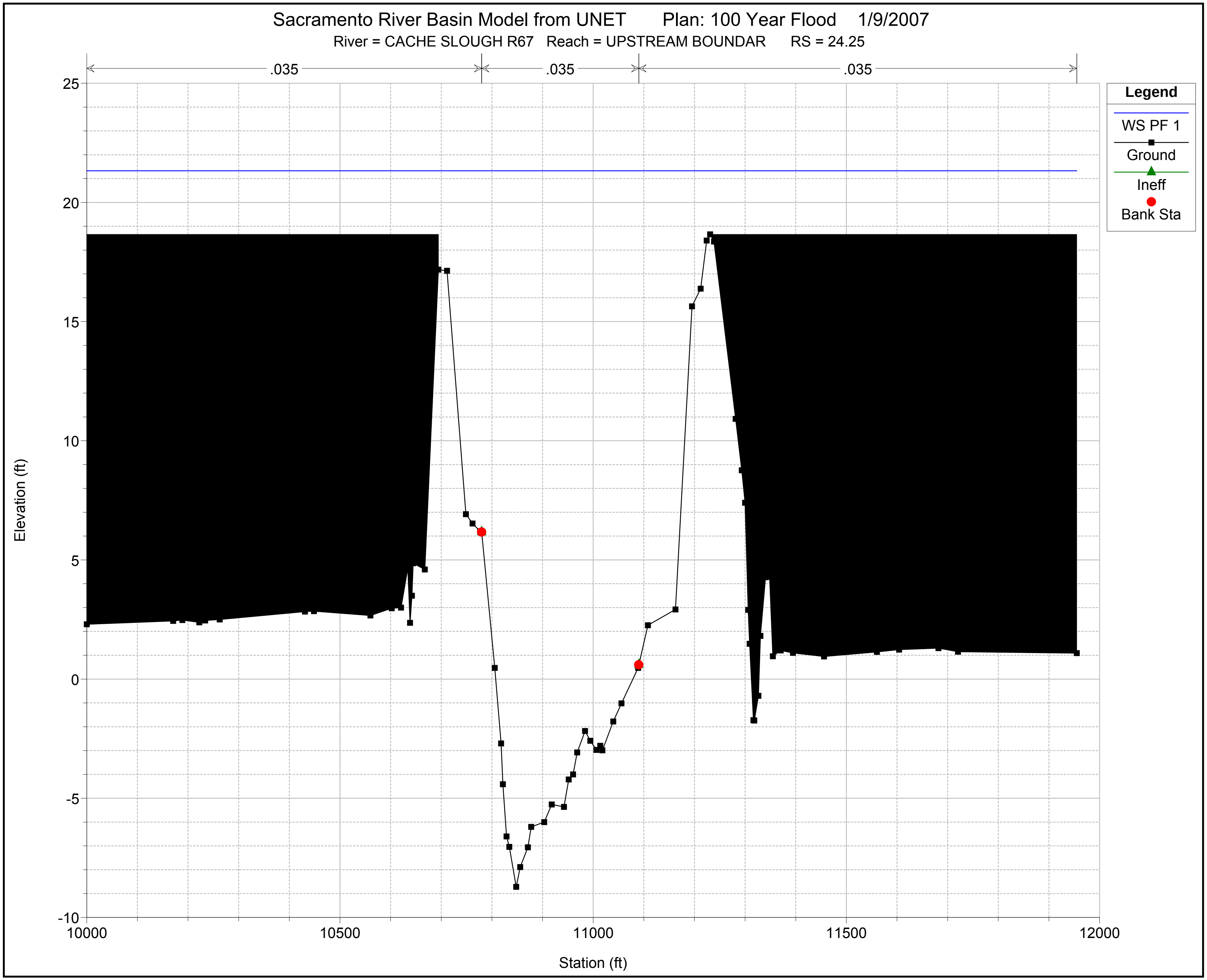

Approximate Location of
Cache Slough RM 23.6
UPSTREAM BOUNDARY
LIBERTY CUT CONN
SHAG SLOUGH CONN
HAAS SLOUGH TO S
SUTTER SLOUGH TO
TOE DRAIN TO CAC
SHAG SLOUGH TO L
CACHE SLOUGH R68
SHAG SLOUGH R65
LIBERTY CUT R62
None of the XS's are Geo-Referenced (· Geo-Ref user entered XS · Geo-Ref interpolated XS · Non Geo-Ref user entered XS · Non Geo-Ref interpolated XS)

HEC-RAS Profile Drawing for Cache Slough RM 23.6 Sheet 1

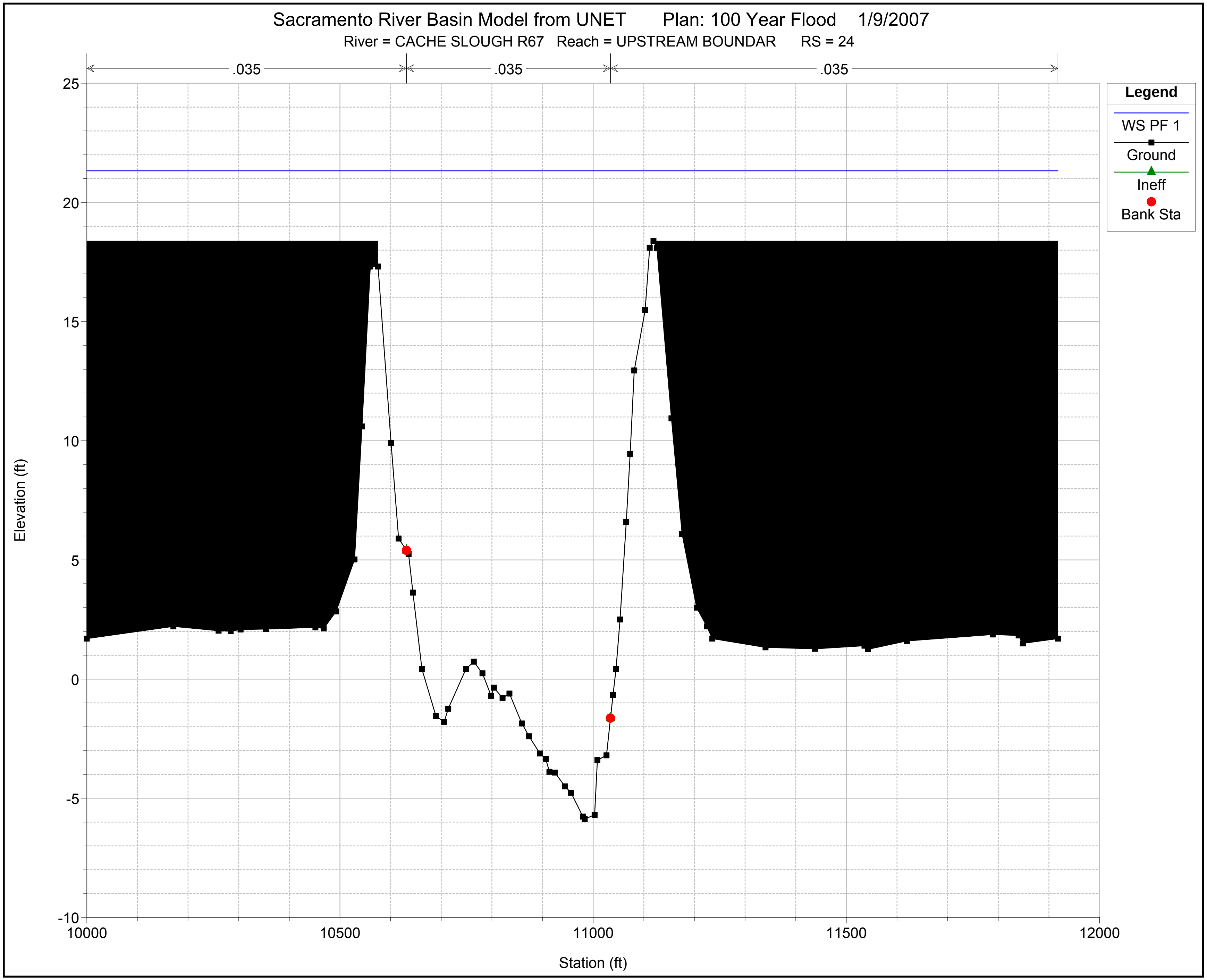


HEC-RAS Profile Drawing for Cache Slough RM 23.6 Sheet 2

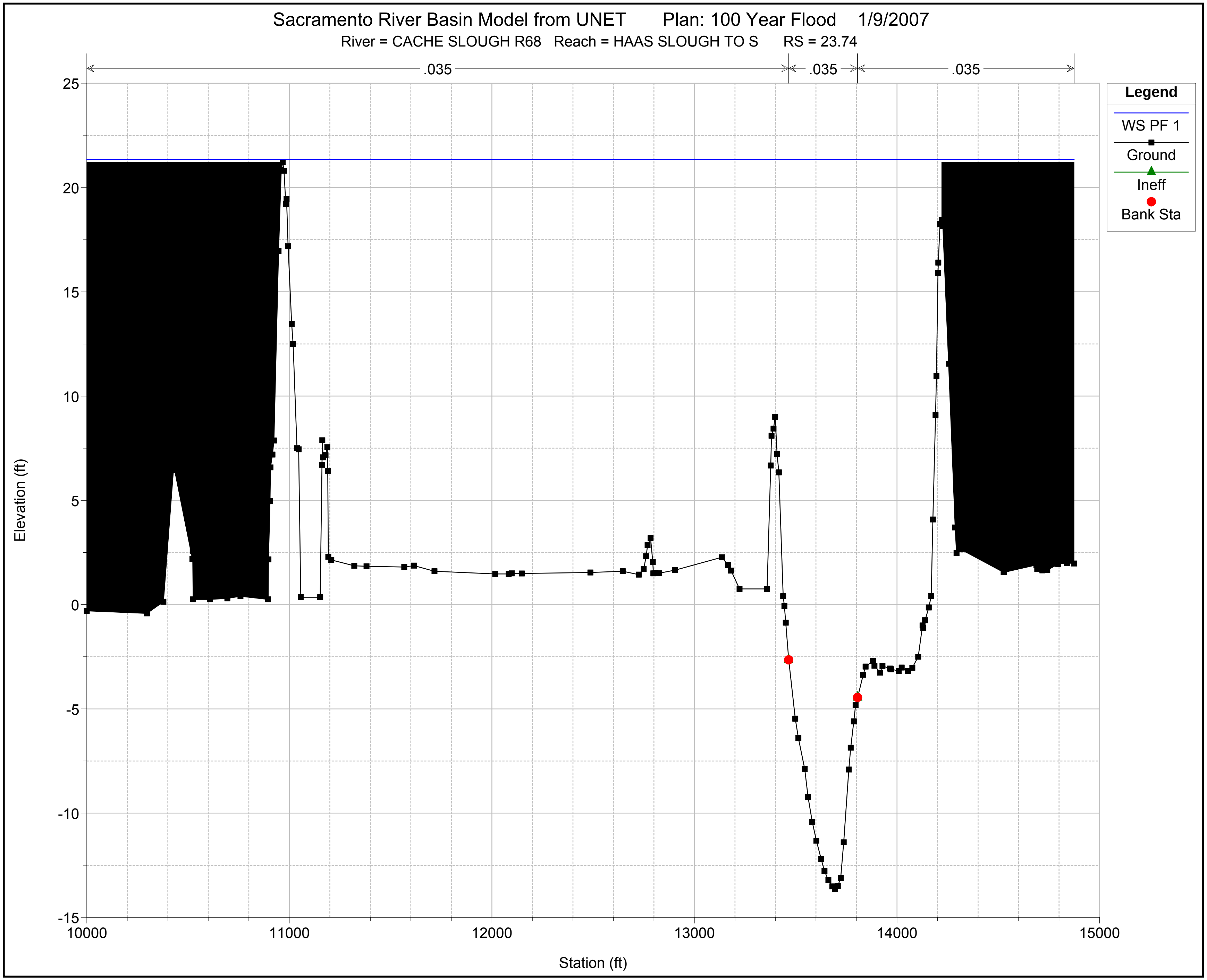




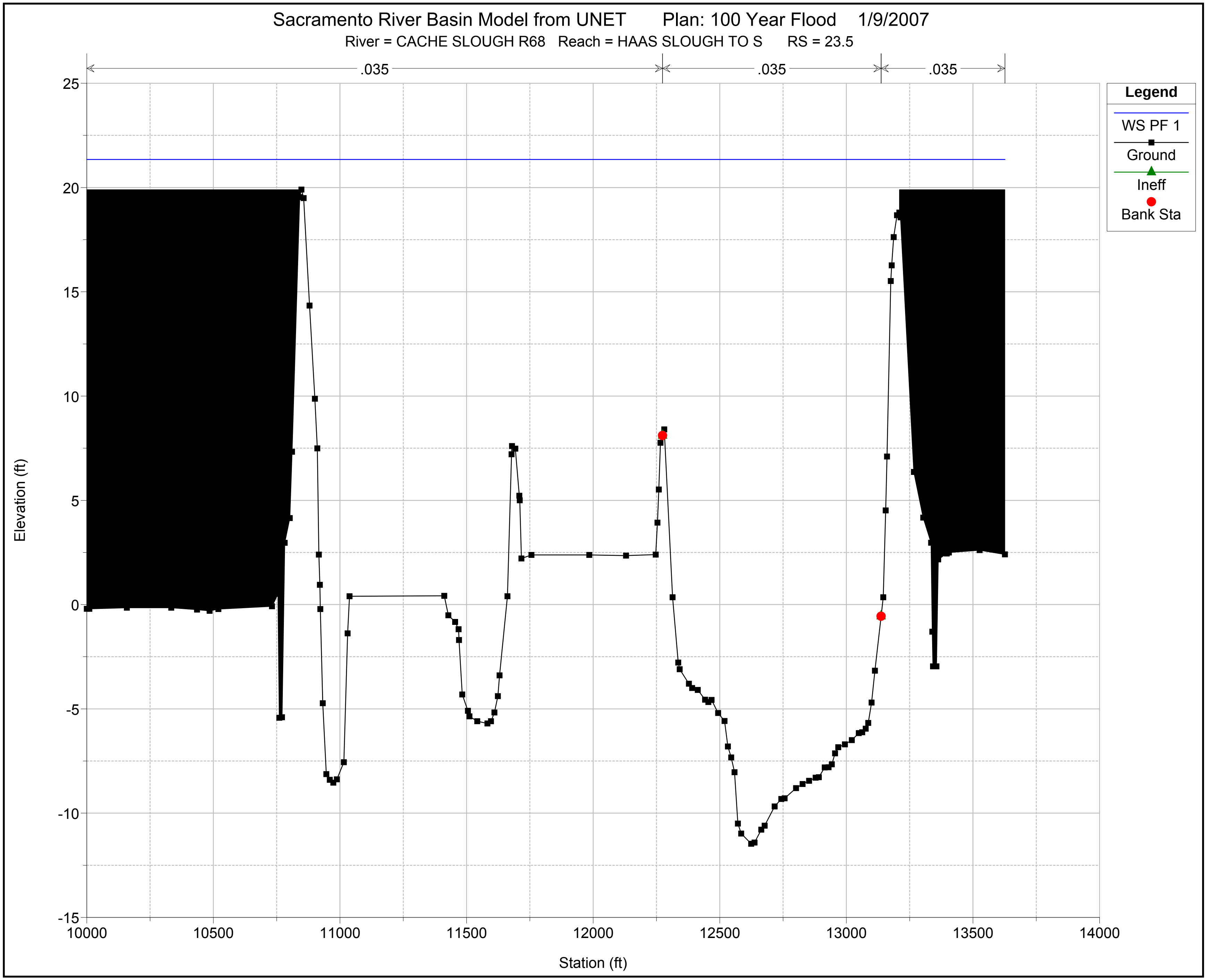
Note: The channel does not have adequate capacity at this location to contain the 100-year flood.



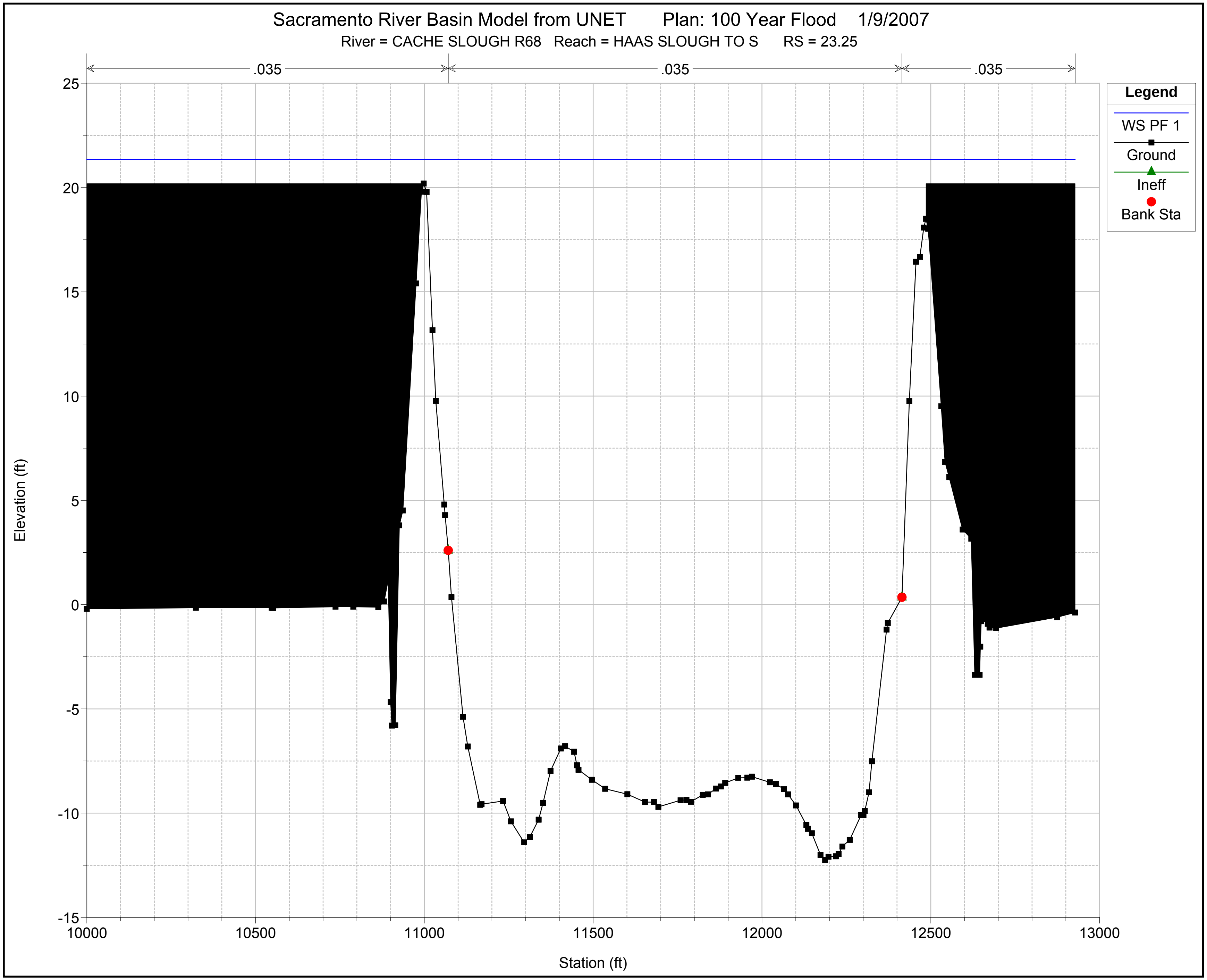
Note: The channel does not have adequate capacity at this location to contain the 100-year flood.



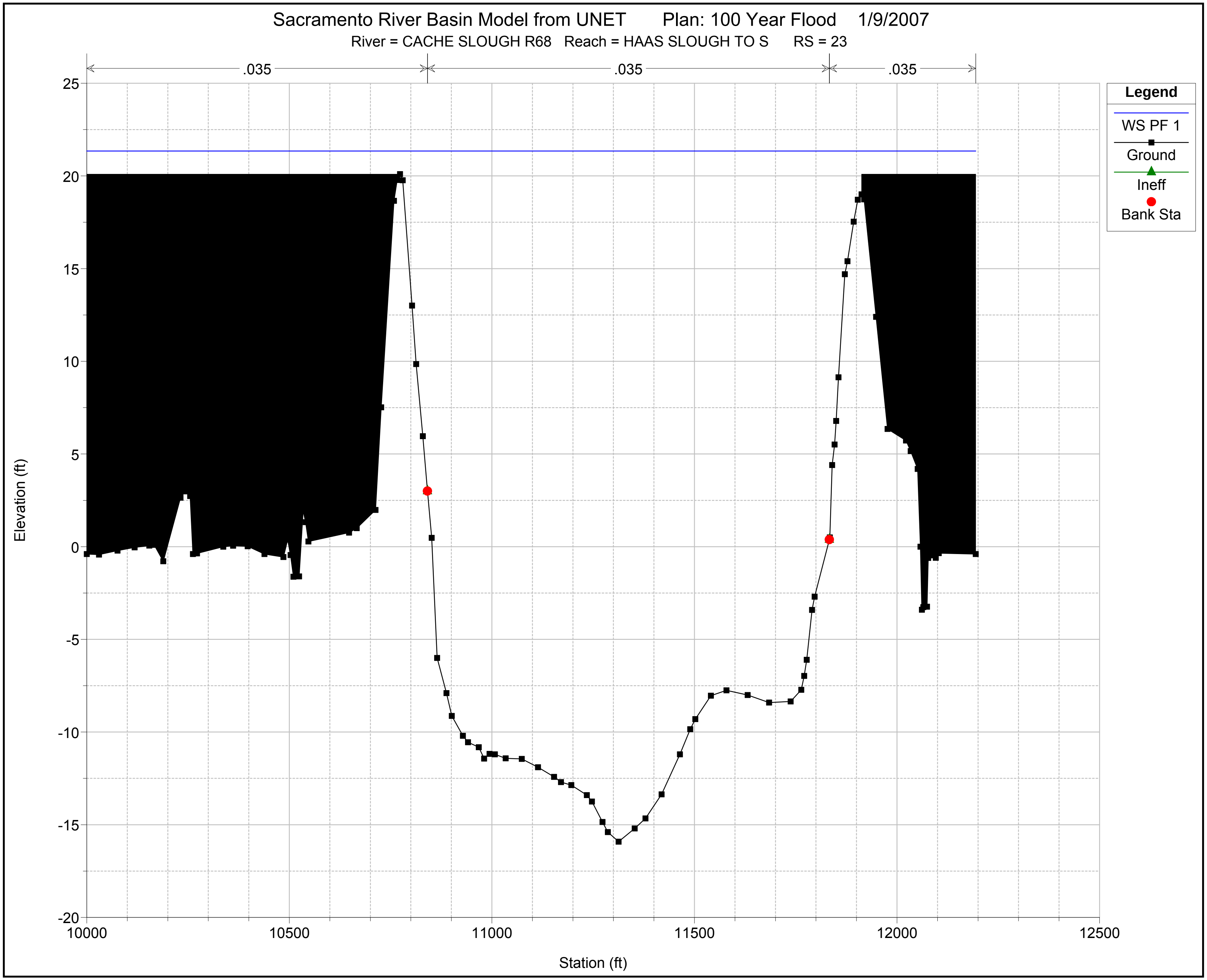
Note: The channel does not have adequate capacity at this location to contain the 100-year flood.



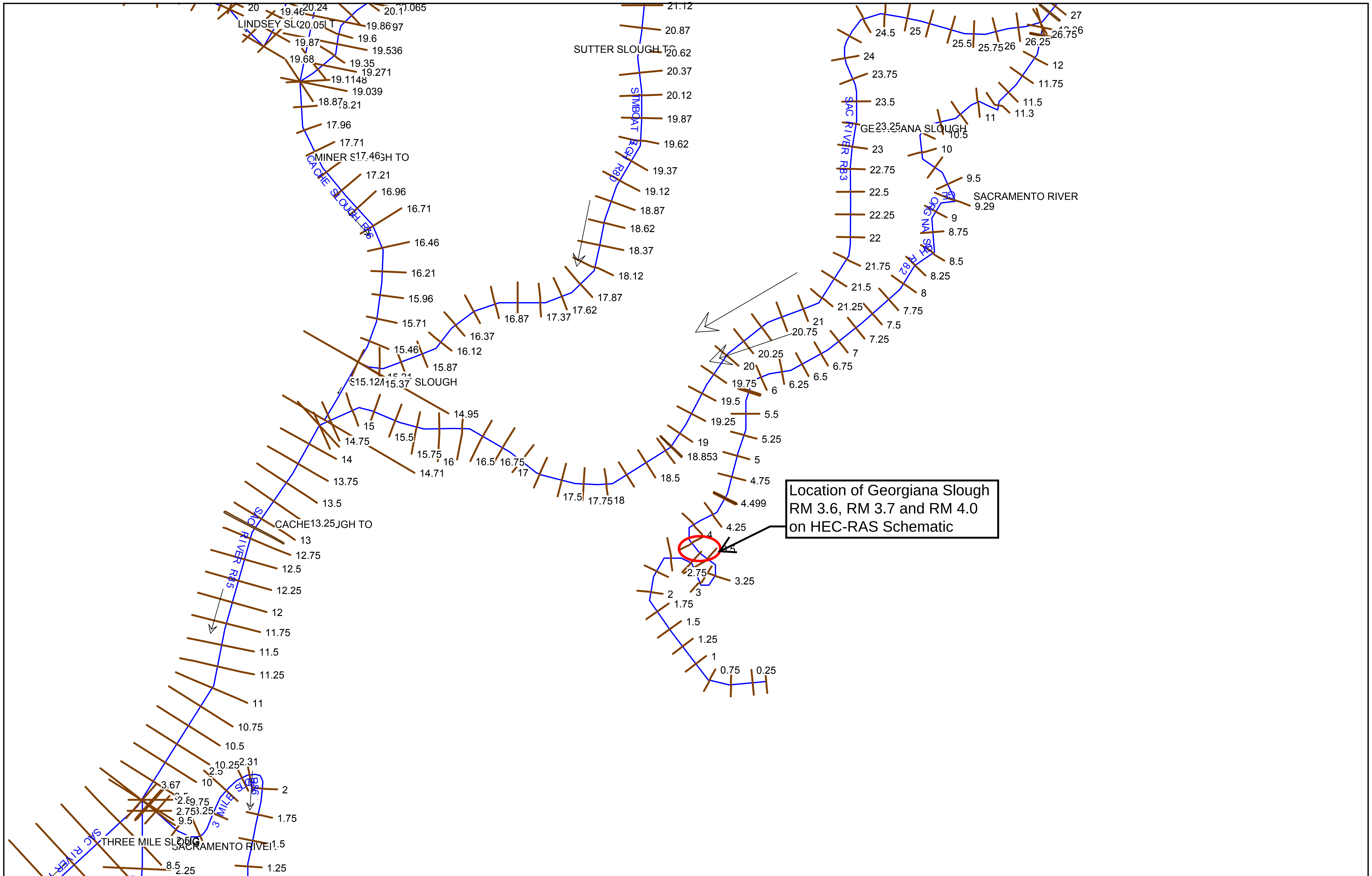
Note: The channel does not have adequate capacity at this location to contain the 100-year flood.



Note: The channel does not have adequate capacity at this location to contain the 100-year flood.

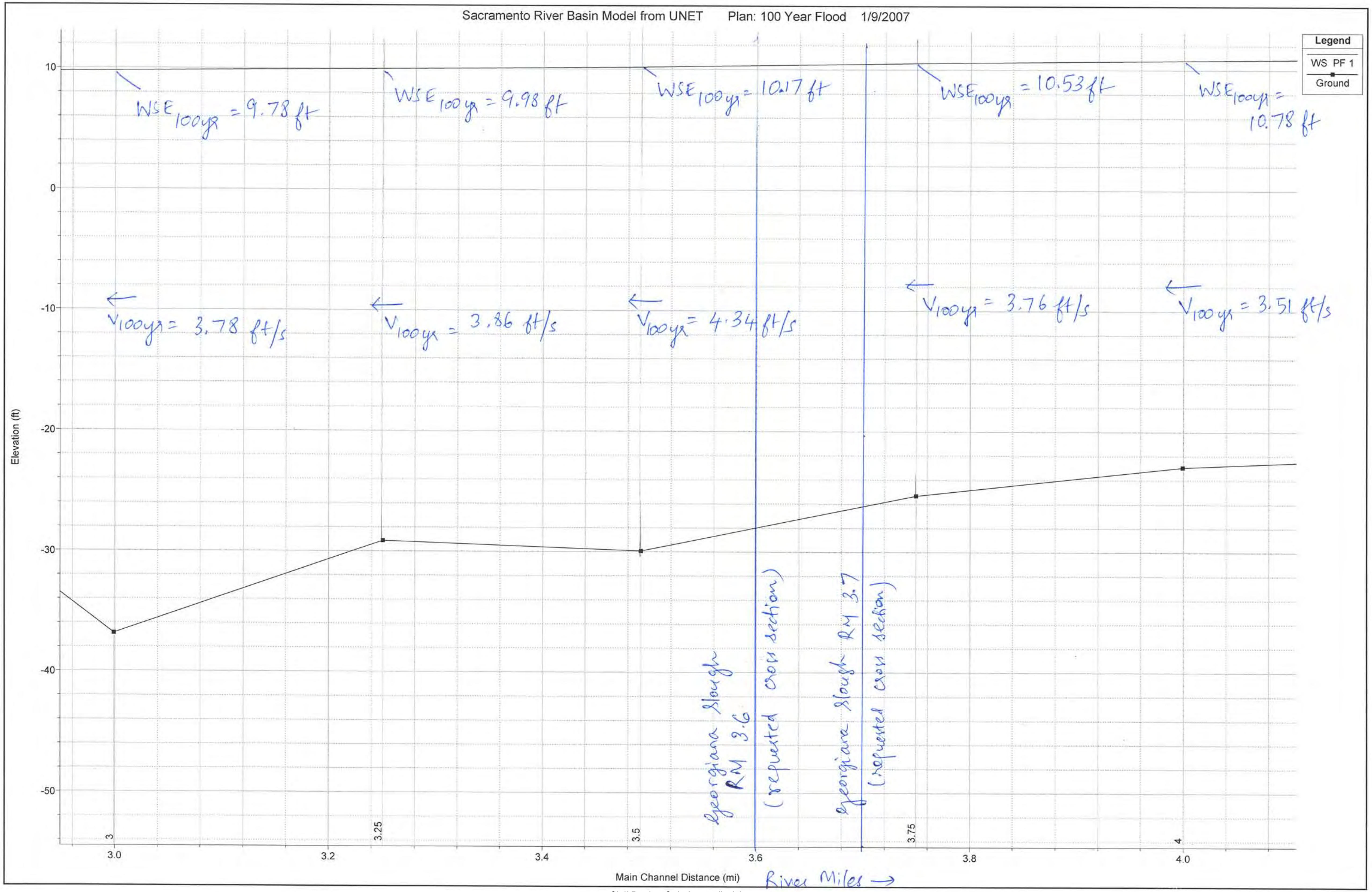


Note: The channel does not have adequate capacity at this location to contain the 100-year flood.

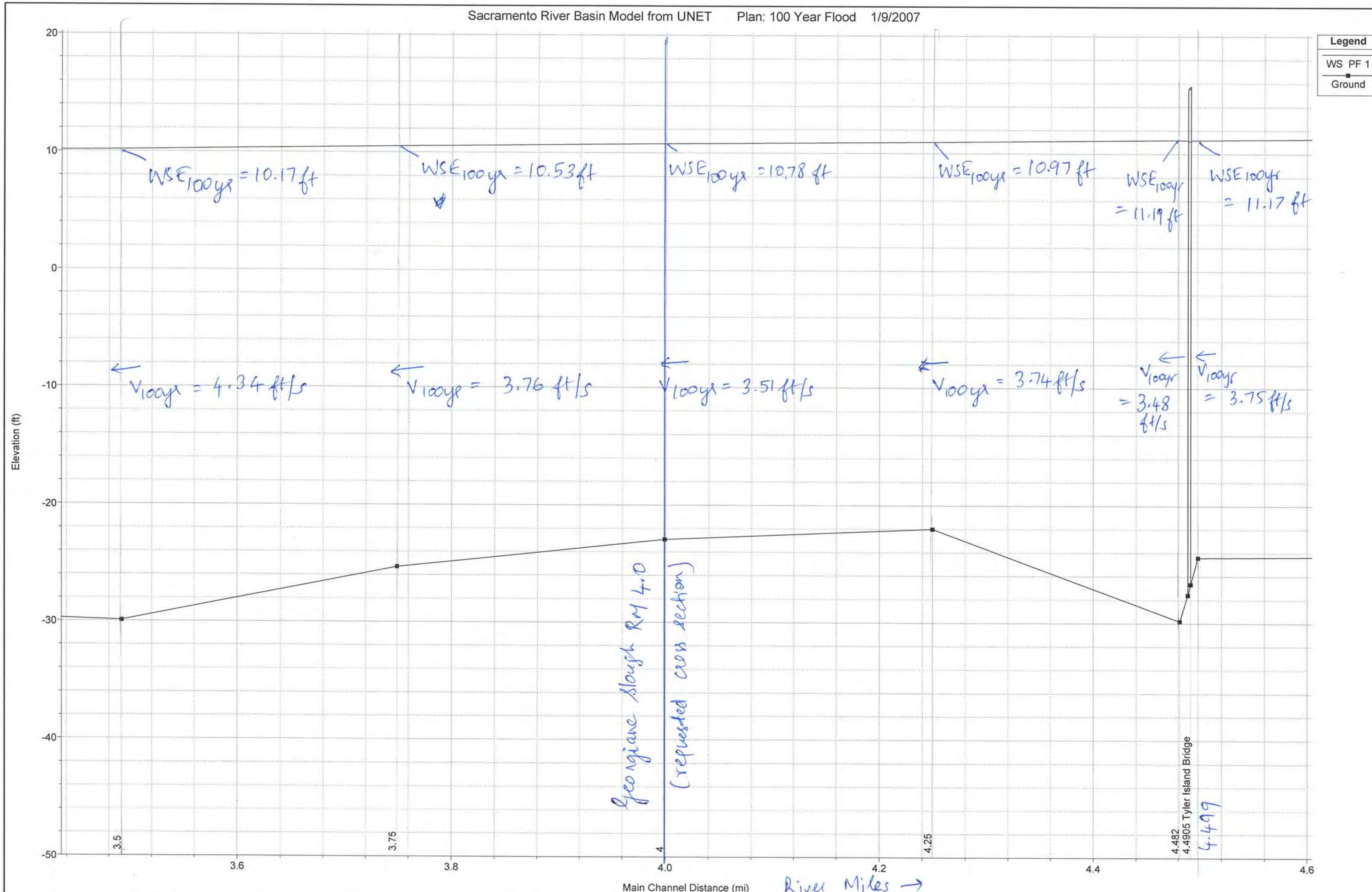


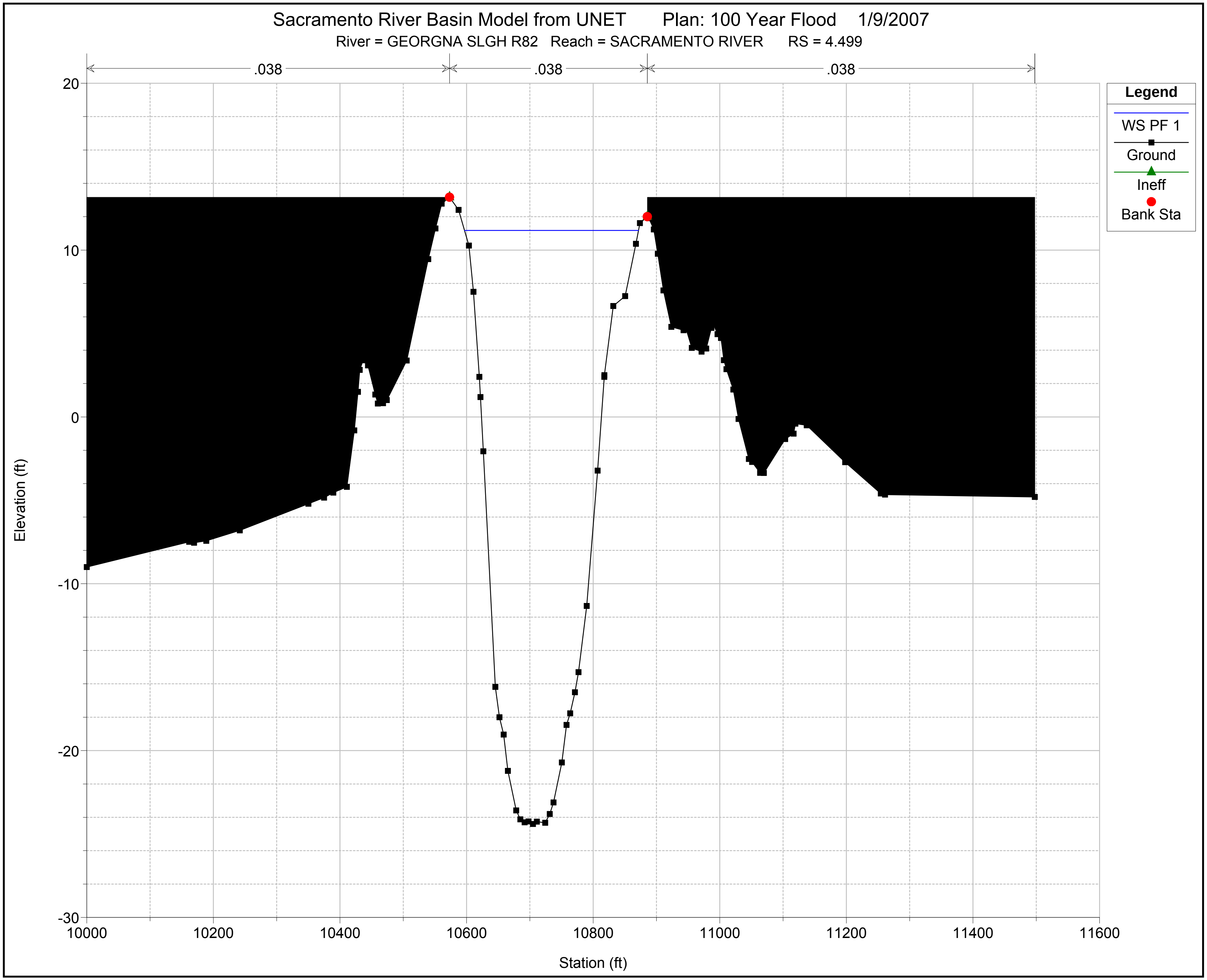
None of the XS's are Geo-Referenced (· Geo-Ref user entered XS · Geo-Ref interpolated XS · Non Geo-Ref user entered XS · Non Geo-Ref interpolated XS)

HEC-RAS Profile Drawing for Georgiana Slough RM 3.6 and 3.7

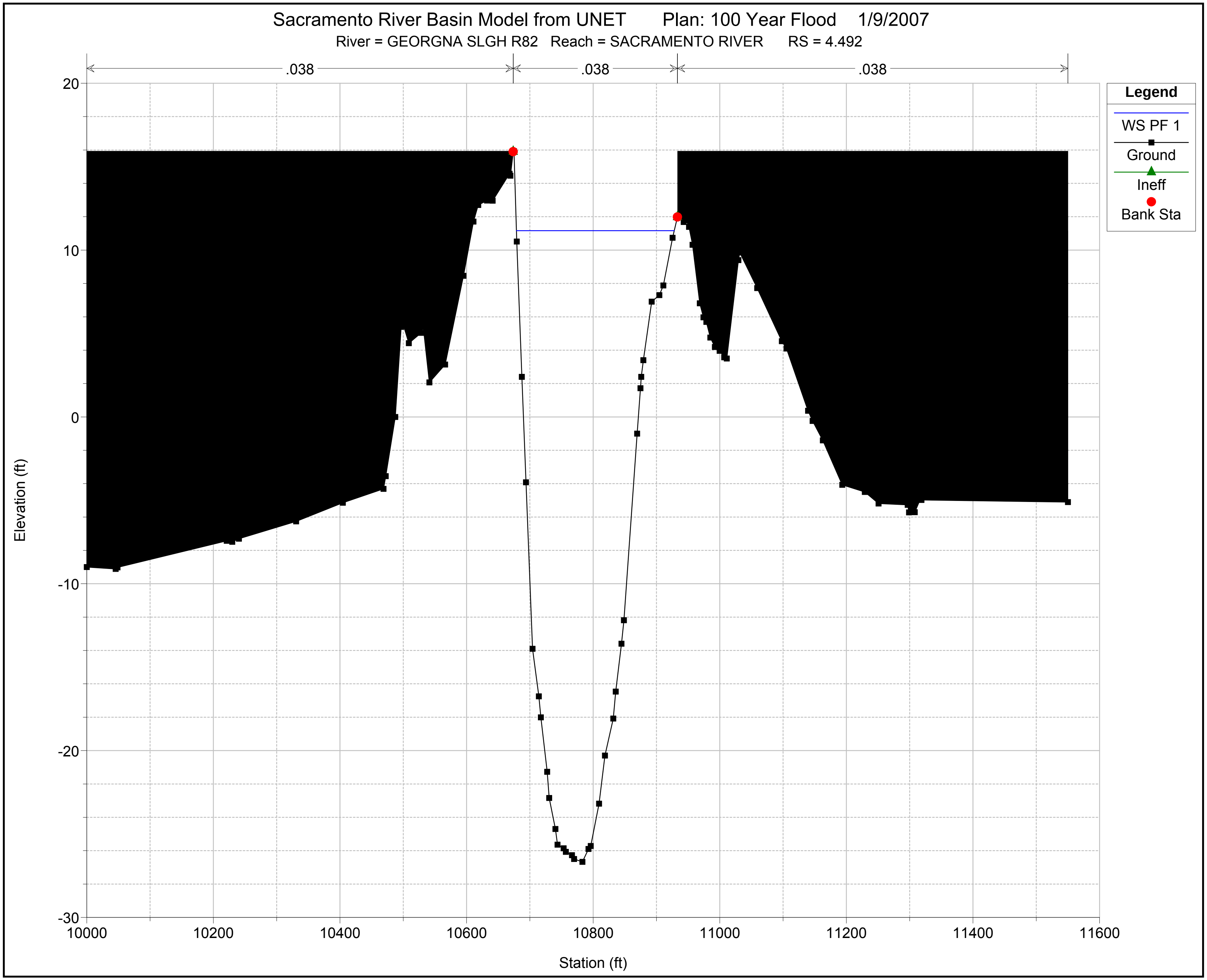


HEC-RAS Profile Drawing for Georgiana Slough RM 4.0

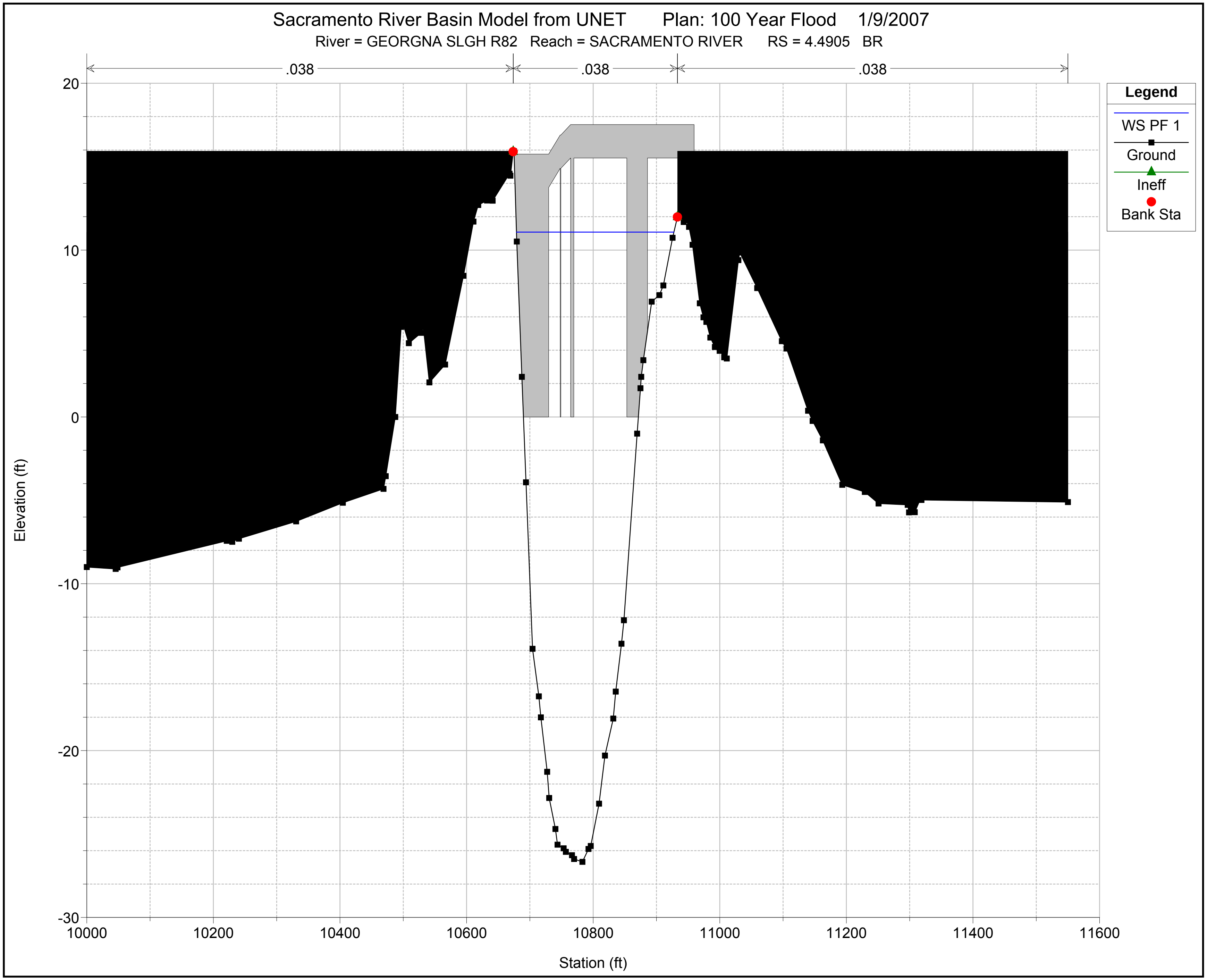


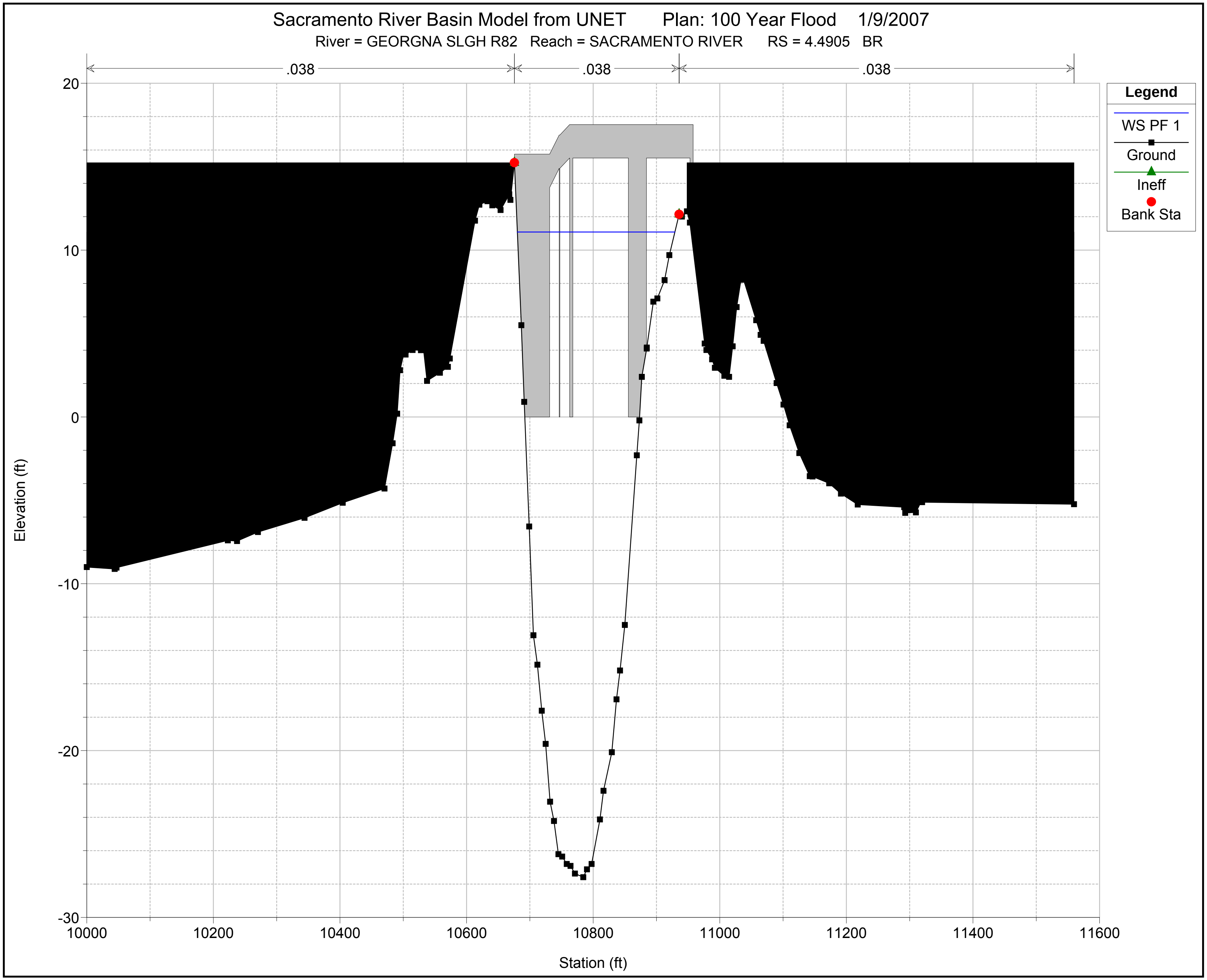


Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = GEORGNA SLGH R82 Reach = SACRAMENTO RIVER RS = 4.499
.038
.038
.038
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
Station (ft)

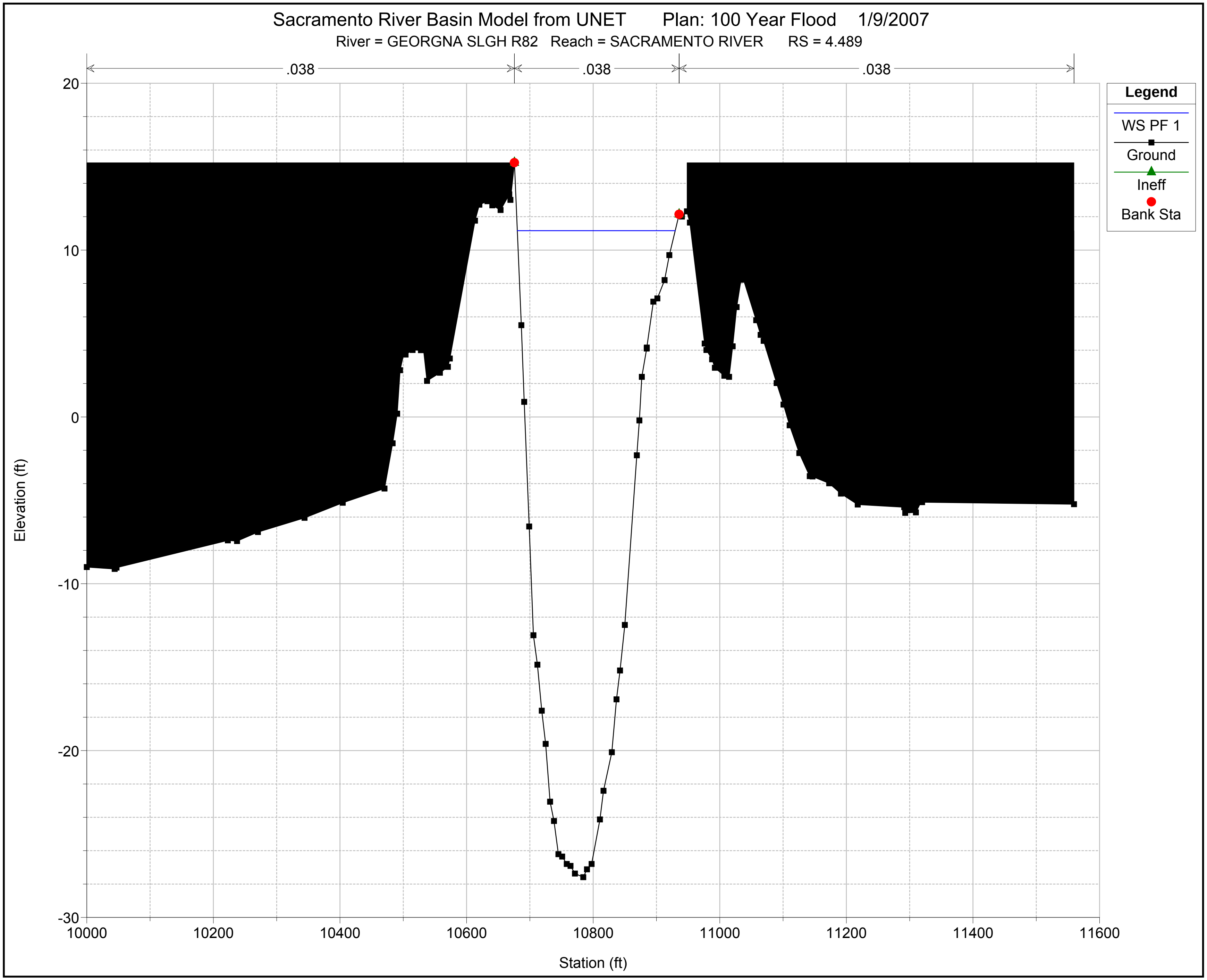


Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = GEORGNA SLGH R82 Reach = SACRAMENTO RIVER RS = 4.492
.038
.038
.038
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
Station (ft)

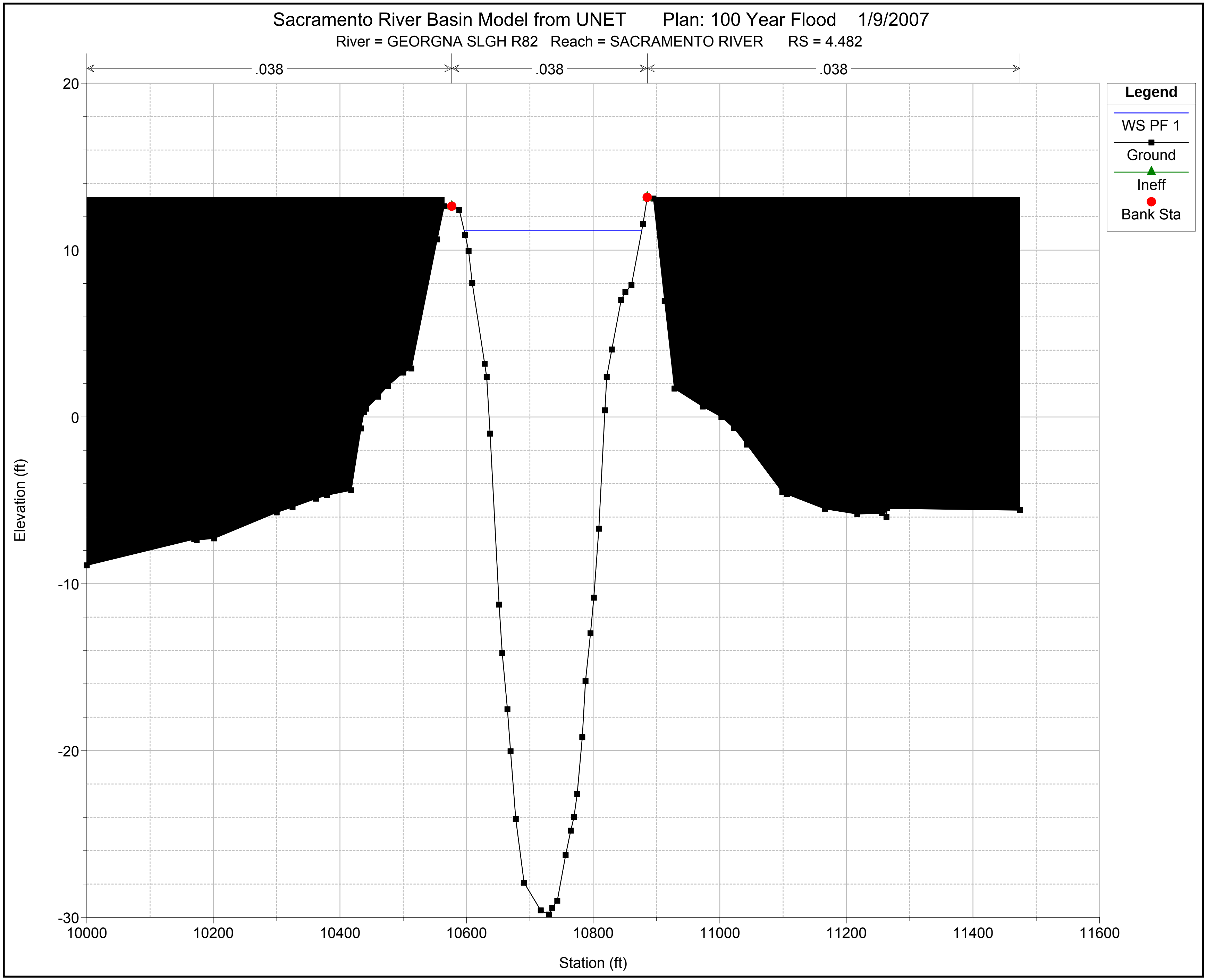

Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = GEORGNA SLGH R82 Reach = SACRAMENTO RIVER RS = 4.4905 BR
.038
.038
.038
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
Station (ft)

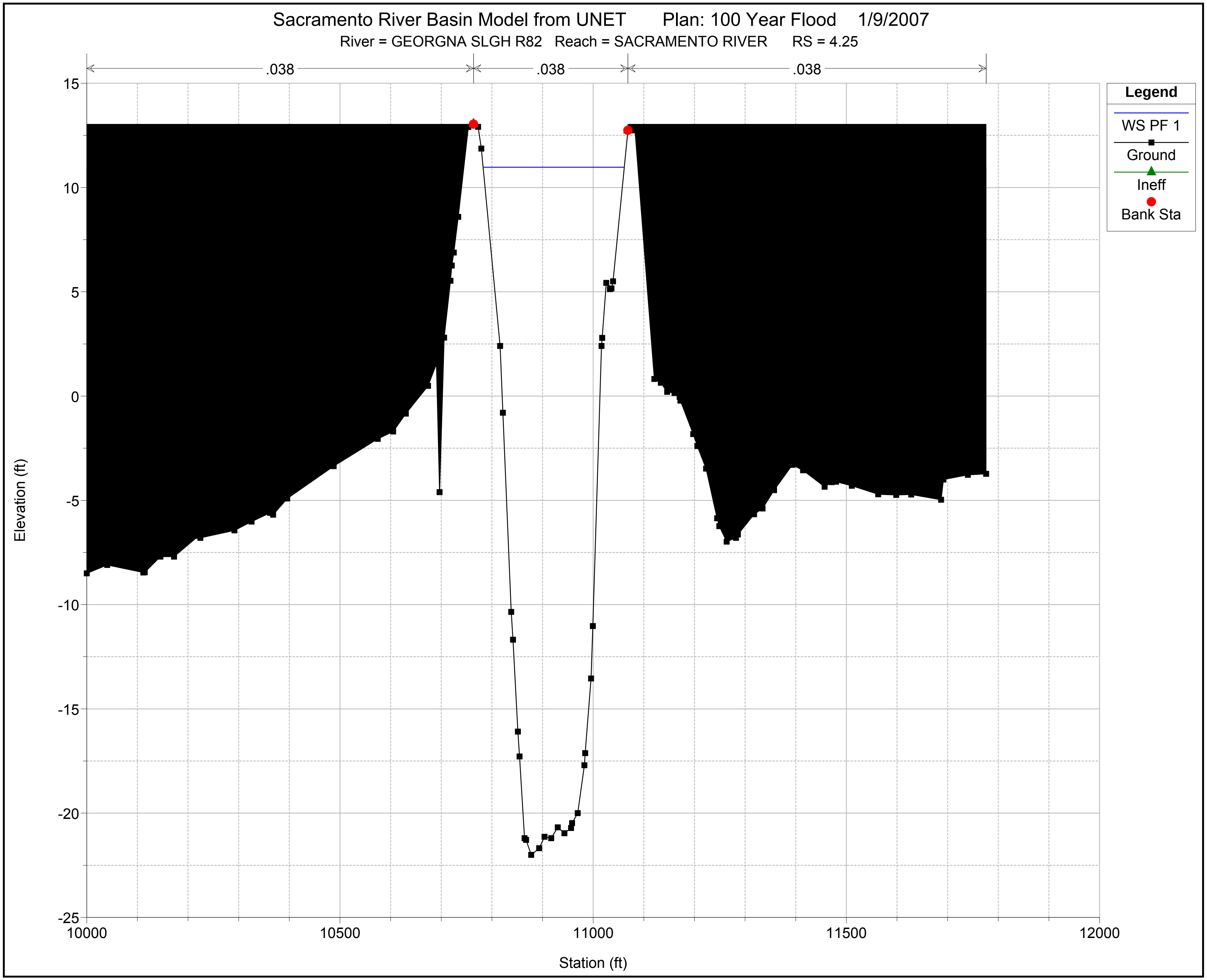


Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = GEORGNA SLGH R82 Reach = SACRAMENTO RIVER RS = 4.4905 BR
.038
.038
.038
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600

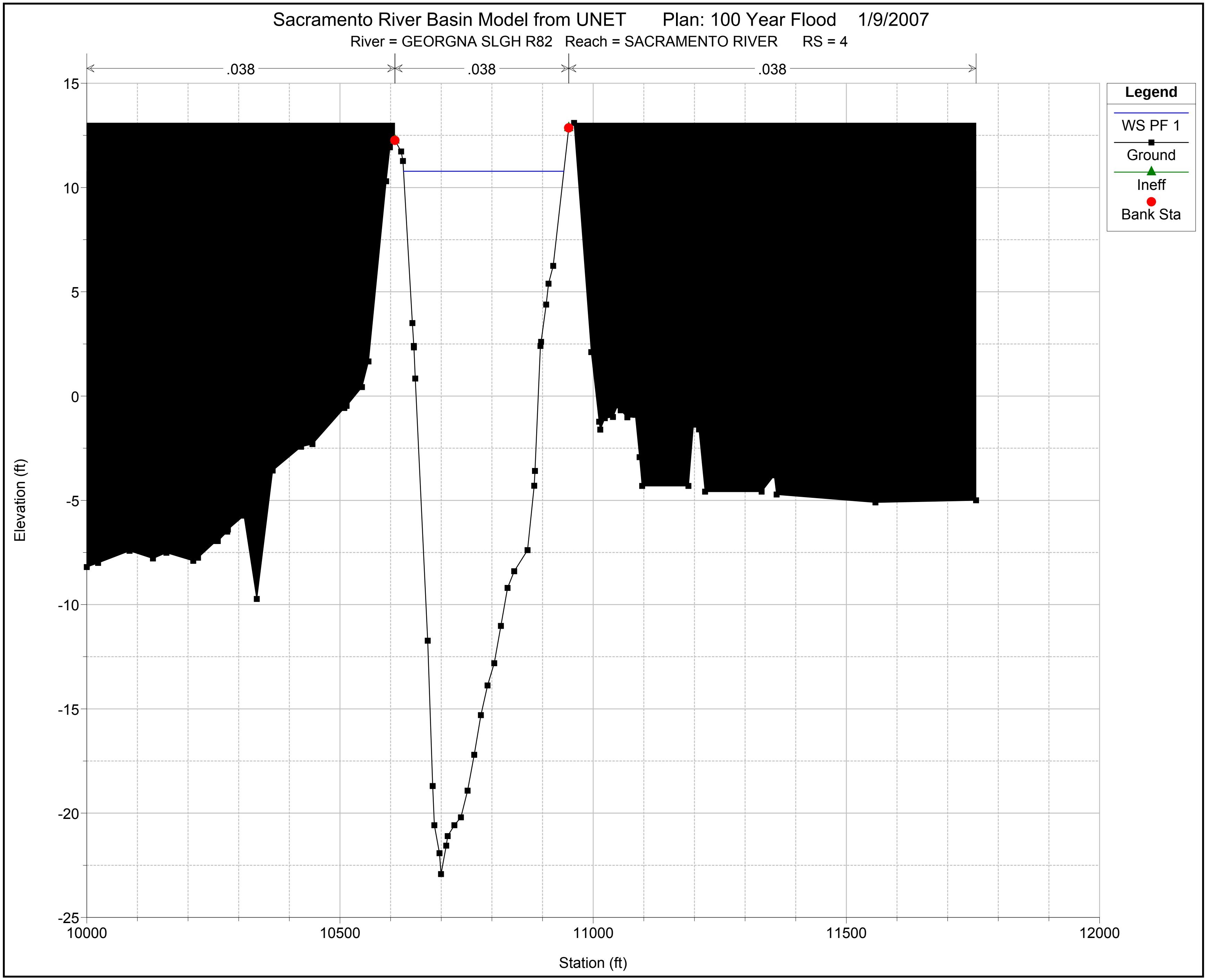


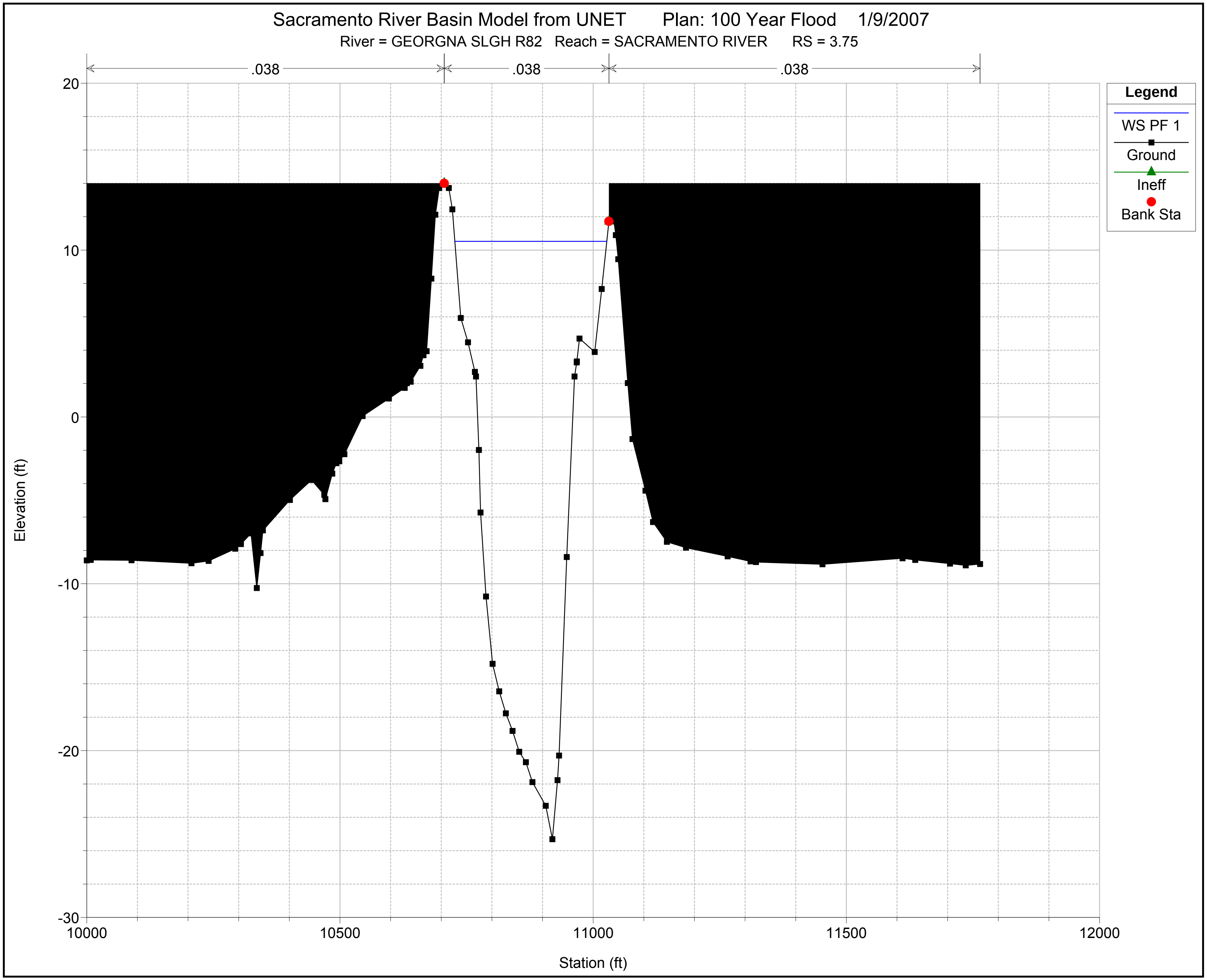
Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = GEORGNA SLGH R82 Reach = SACRAMENTO RIVER RS = 4.489
.038
.038
.038
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
Station (ft)


Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = GEORGNA SLGH R82 Reach = SACRAMENTO RIVER RS = 4.482
.038
.038
.038
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
Station (ft)

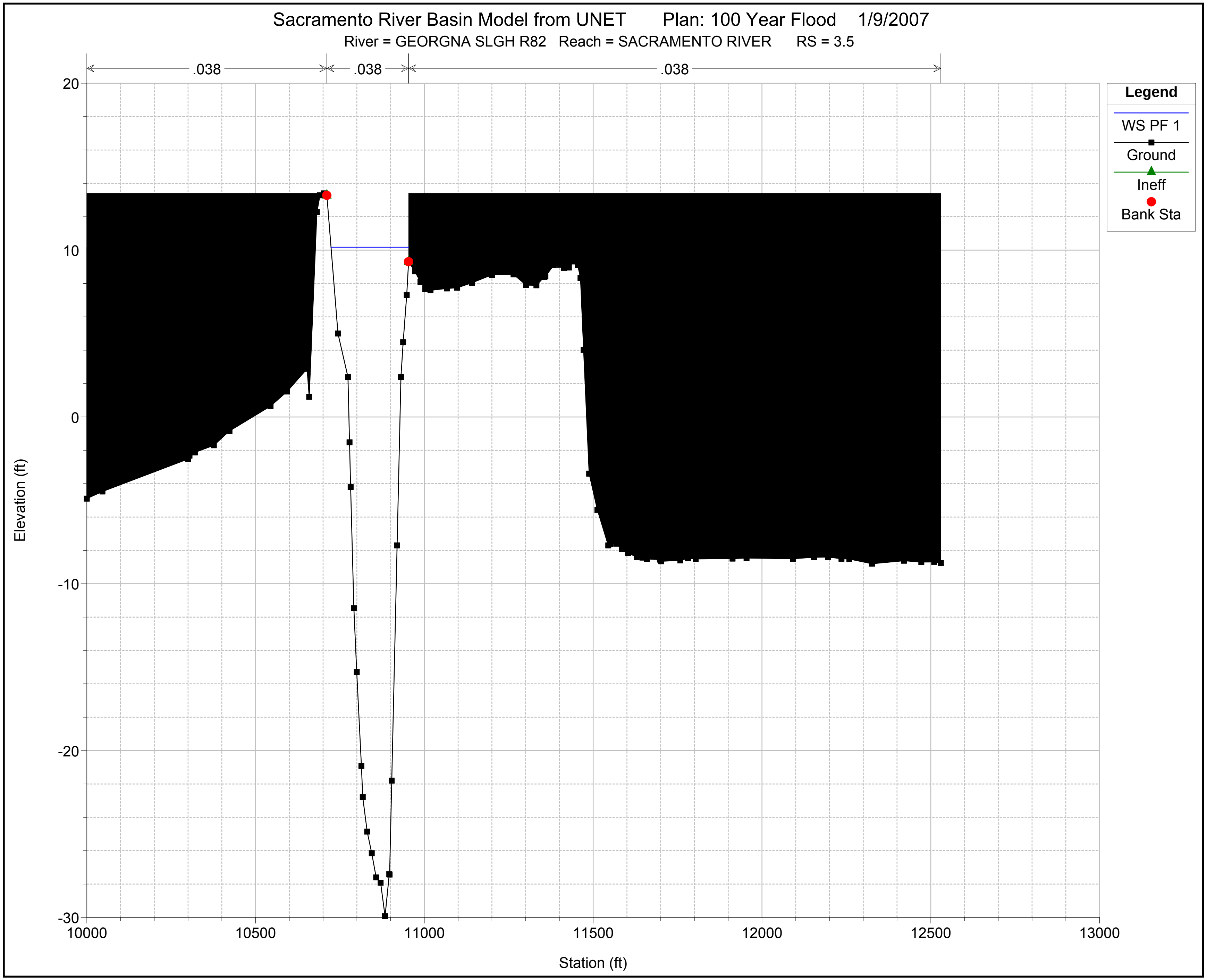


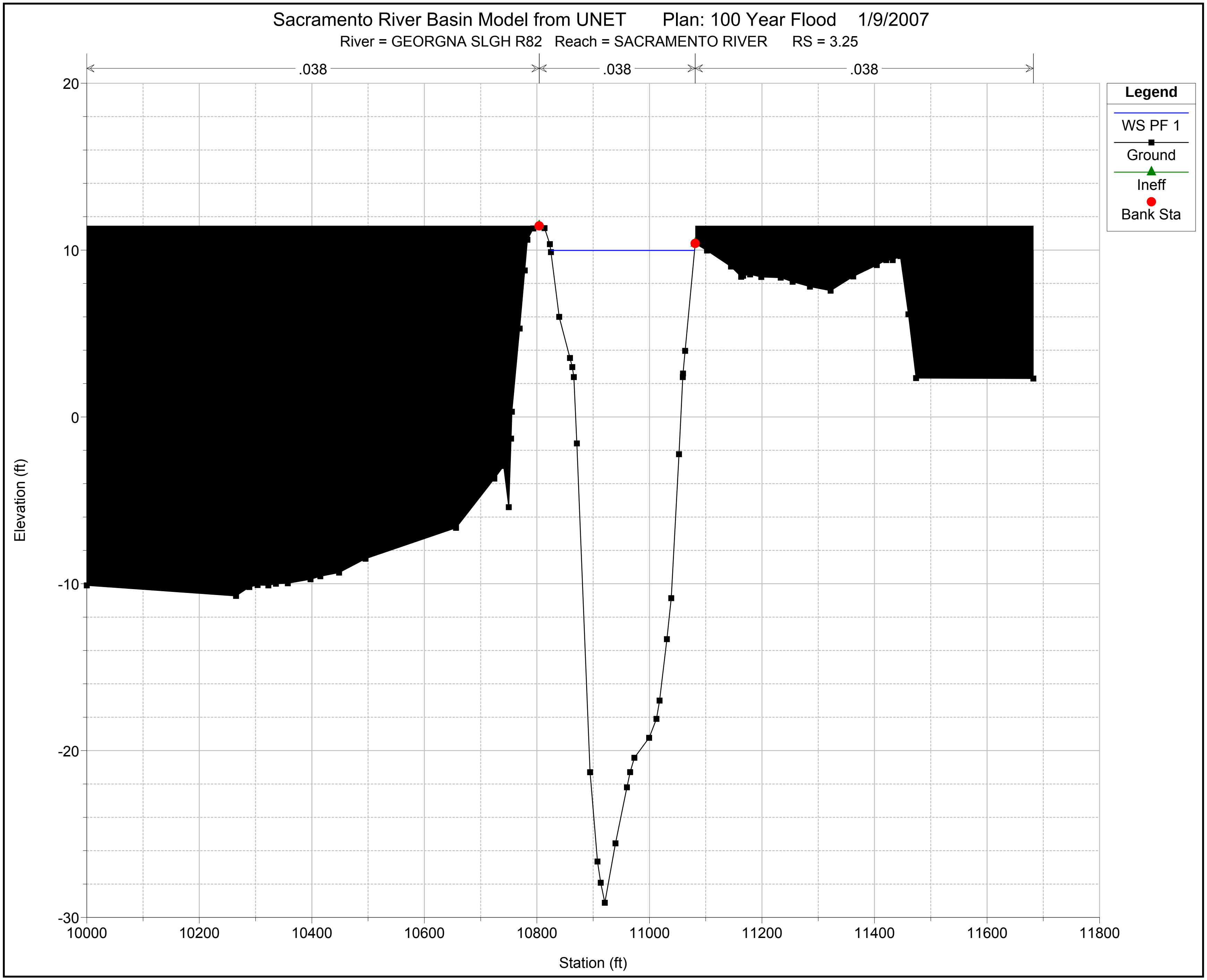
Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = GEORGNA SLGH R82 Reach = SACRAMENTO RIVER RS = 4.25
.038
.038
.038
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
15
10
5
0
-5
-10
-15
-20
-25
10000
10500
11000
11500
12000
Station (ft)


Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = GEORGNA SLGH R82 Reach = SACRAMENTO RIVER RS = 4
.038
.038
.038
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
15
10
5
0
-5
-10
-15
-20
-25
10000
10500
11000
11500
12000
Station (ft)

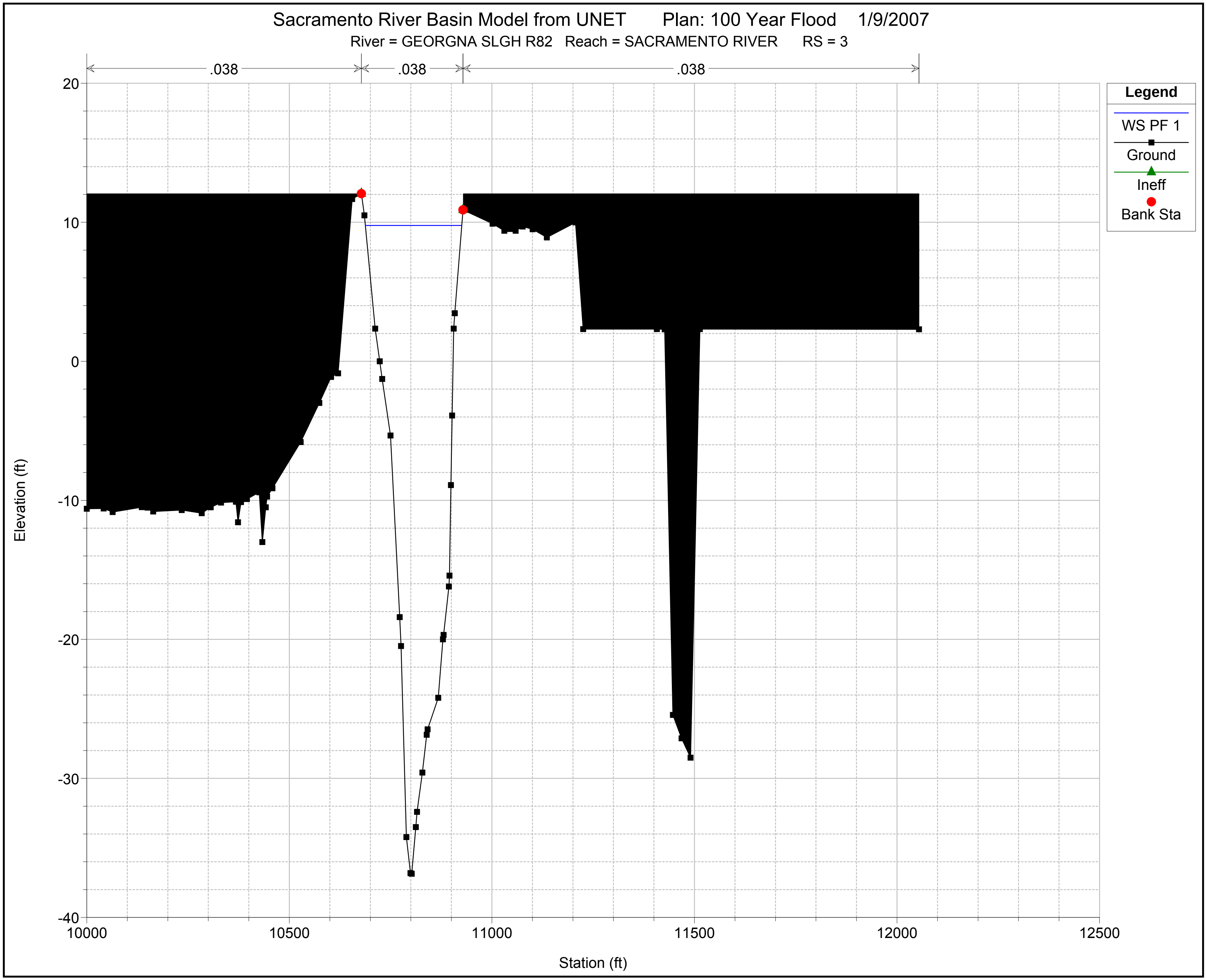


Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = GEORGNA SLGH R82 Reach = SACRAMENTO RIVER RS = 3.75
.038
.038
.038
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
20
10
0
-10
-20
-30
10000
10500
11000
11500
12000

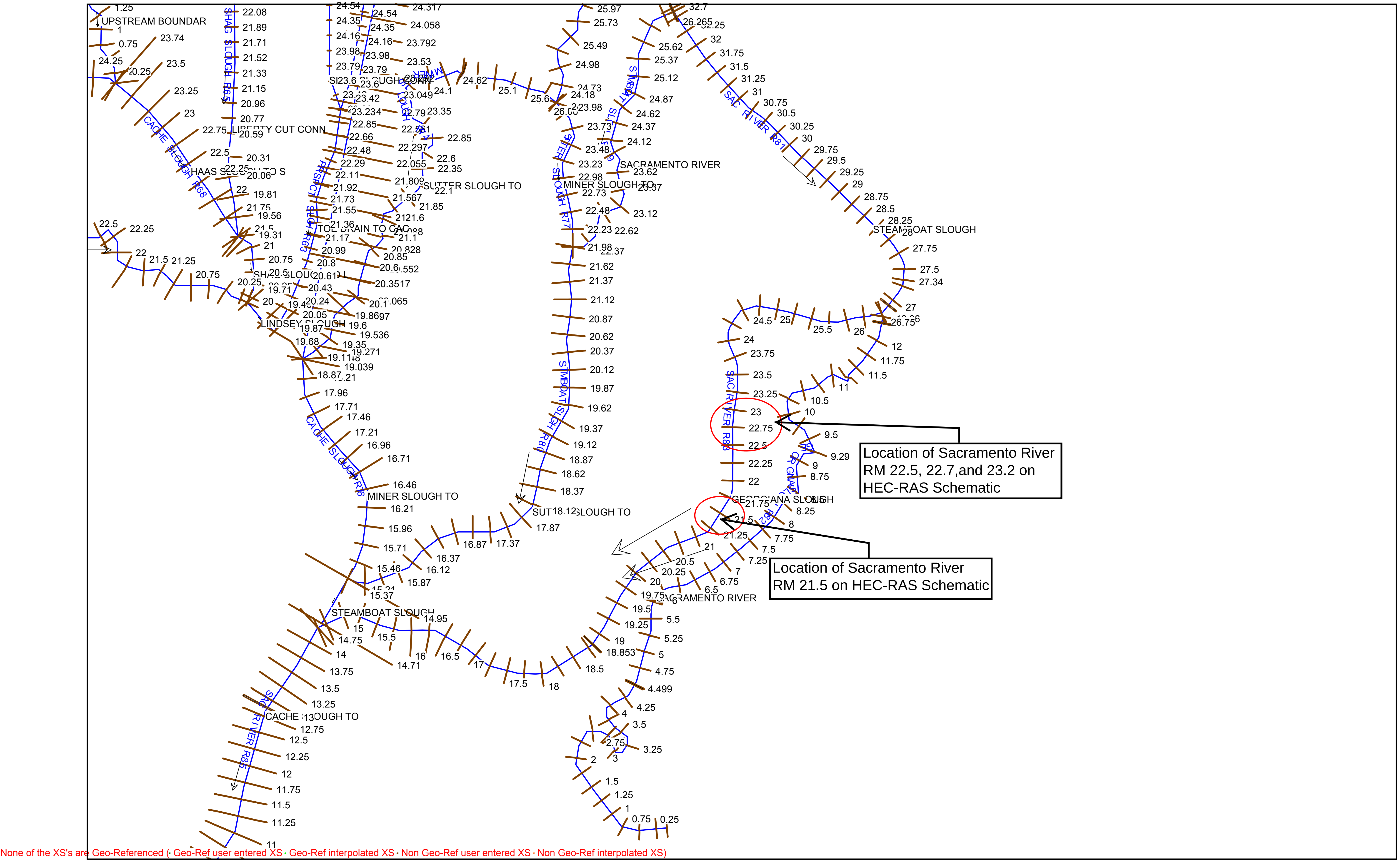

Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = GEORGNA SLGH R82 Reach = SACRAMENTO RIVER RS = 3.5
.038
.038
.038
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
20
10
0
-10
-20
-30
10000
10500
11000
11500
12000
12500
13000



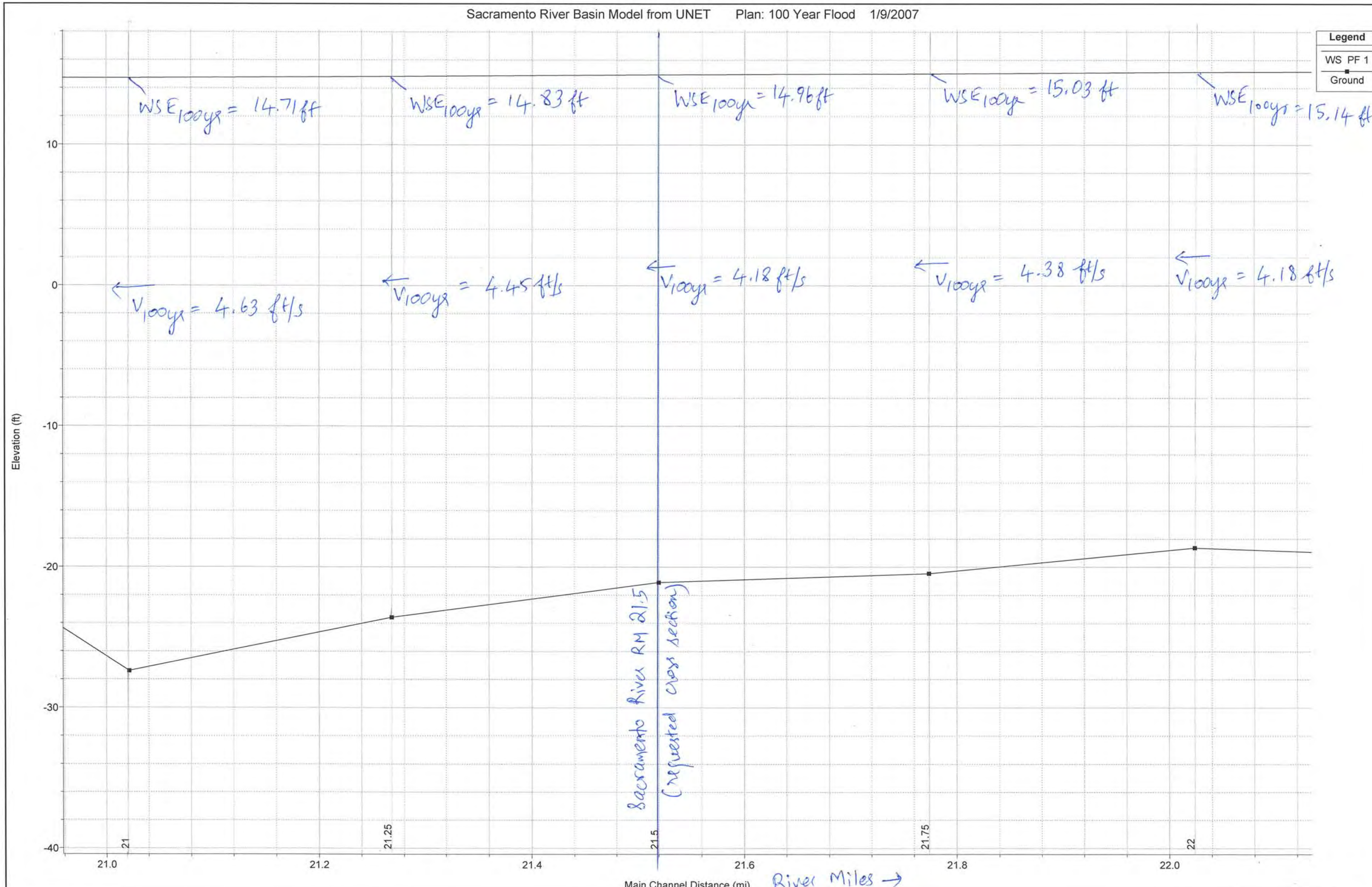
Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = GEORGNA SLGH R82 Reach = SACRAMENTO RIVER RS = 3.25
.038
.038
.038
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800
Station (ft)



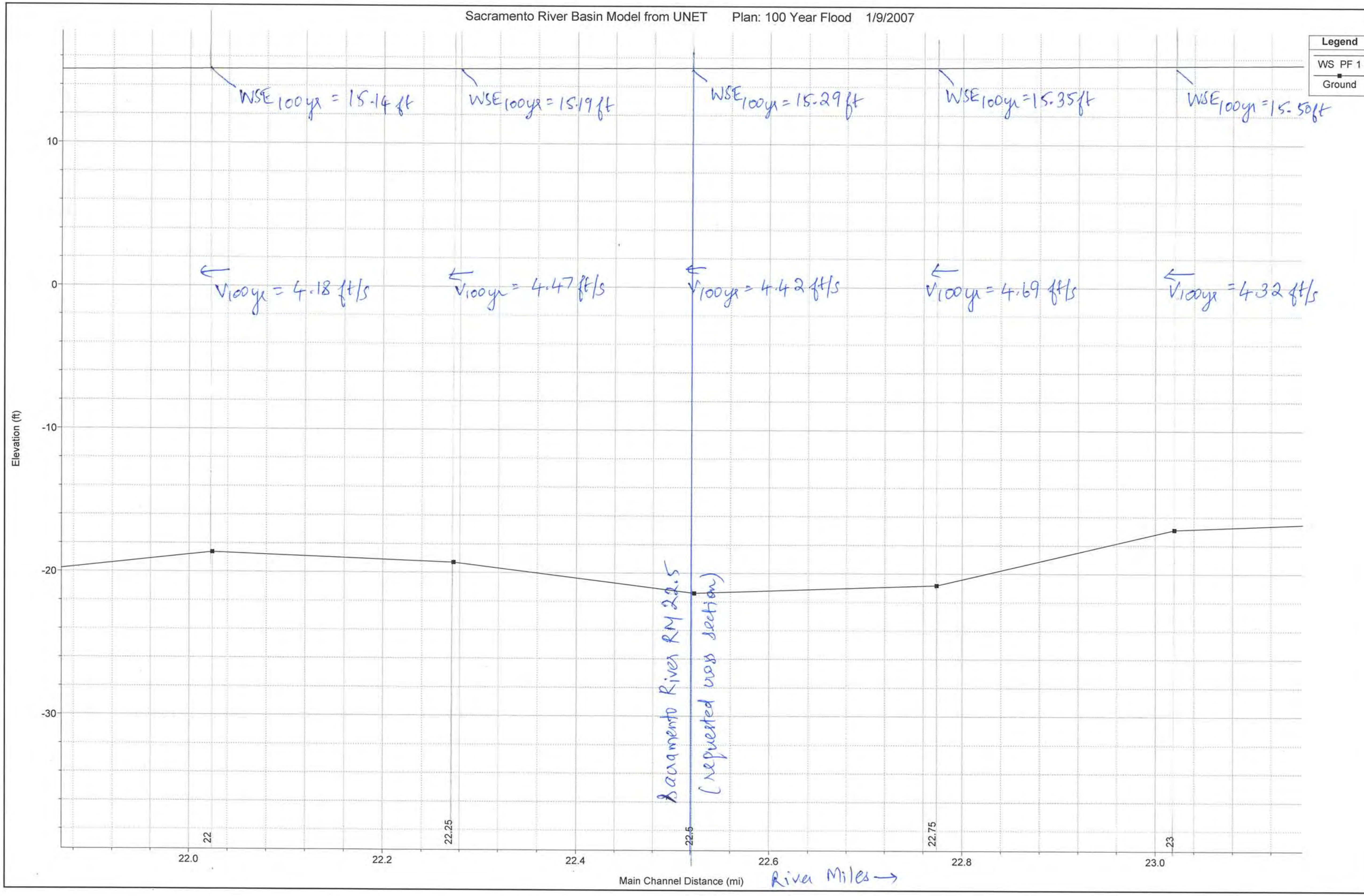
Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = GEORGNA SLGH R82 Reach = SACRAMENTO RIVER RS = 3
.038
.038
.038
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
20
10
0
-10
-20
-30
-40
10000
10500
11000
11500
12000
12500


UPSTREAM BOUNDAR
LIBERTY CUT CONN
HAAS SLOUGH TO S
SUTTER SLOUGH TO
SACRAMENTO RIVER
MINER SLOUGH TO
STEAMBOAT SLOUGH
TOE DRAIN TO CAC
LINDSEY SLOUGH
GEORGIANA SLOUGH
MINER SLOUGH TO
CACHE SLOUGH TO
Location of Sacramento River
RM 22.5, 22.7,and 23.2 on
HEC-RAS Schematic
Location of Sacramento River
RM 21.5 on HEC-RAS Schematic
None of the XS's are Geo-Referenced (Geo-Ref user entered XS Geo-Ref interpolated XS Non Geo-Ref user entered XS Non Geo-Ref interpolated XS)

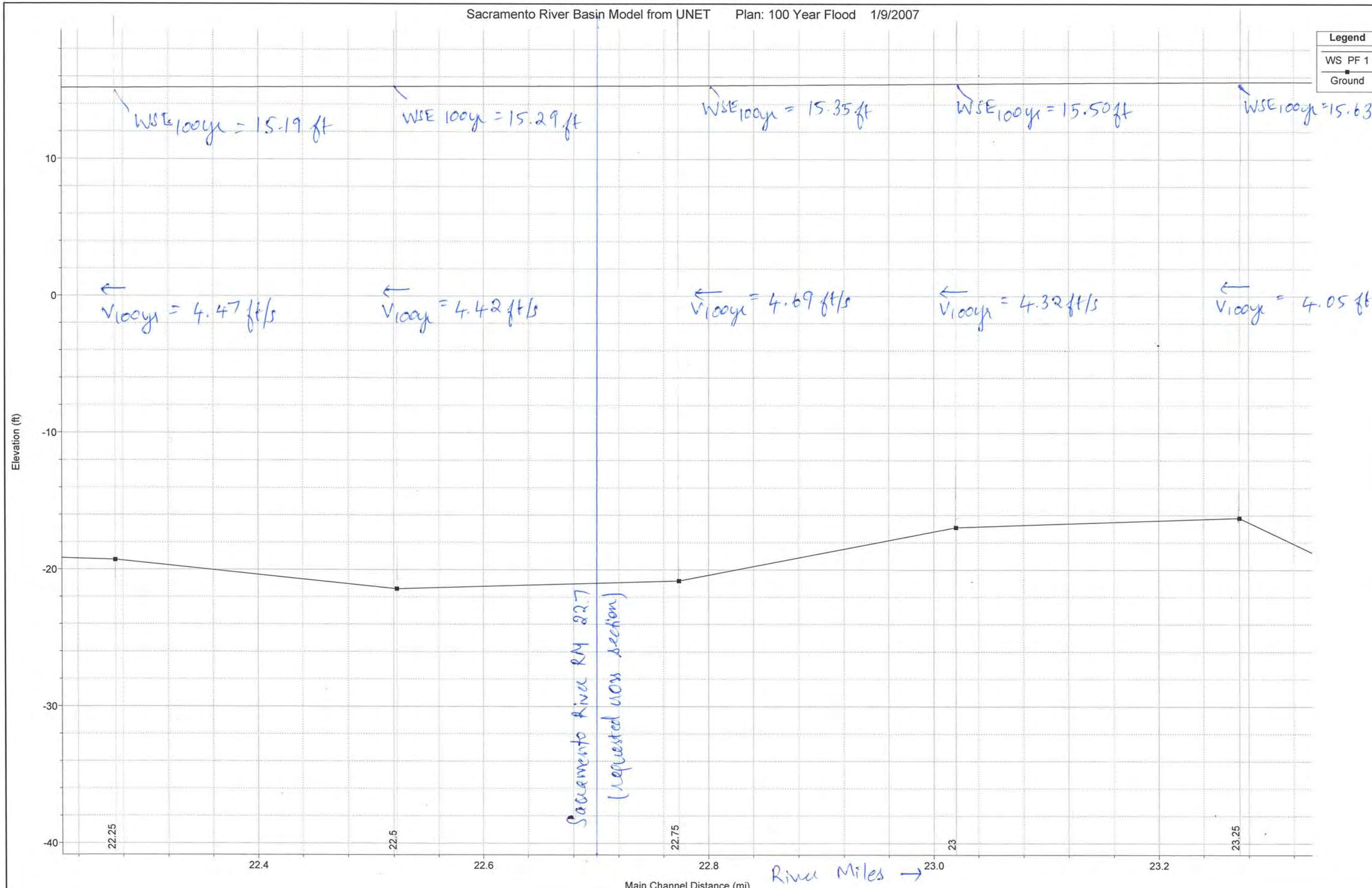
HEC-RAS Profile Drawing for Sacramento River RM 21.5



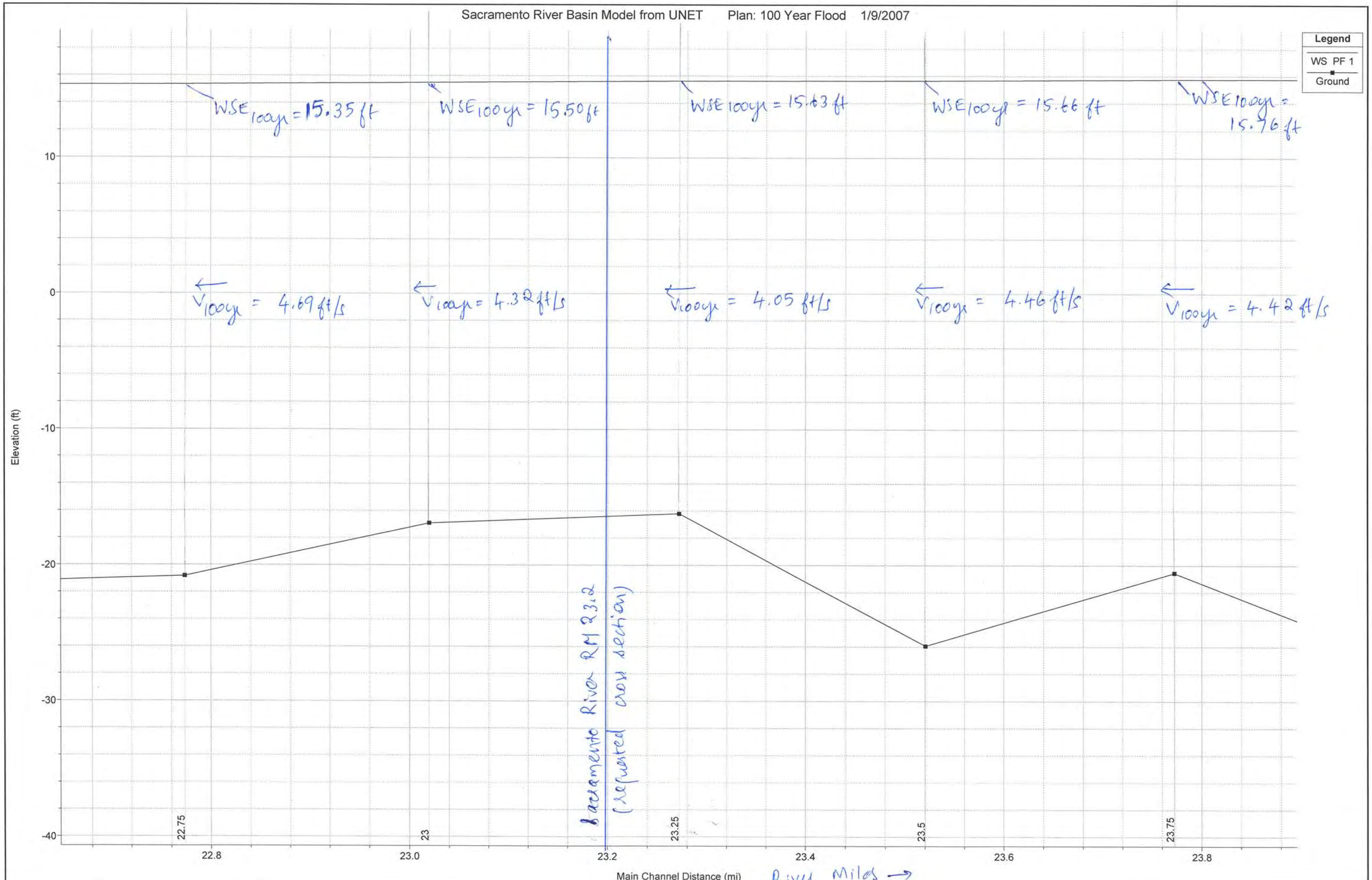
HEC-RAS Profile Drawing for Sacramento River RM 22.5

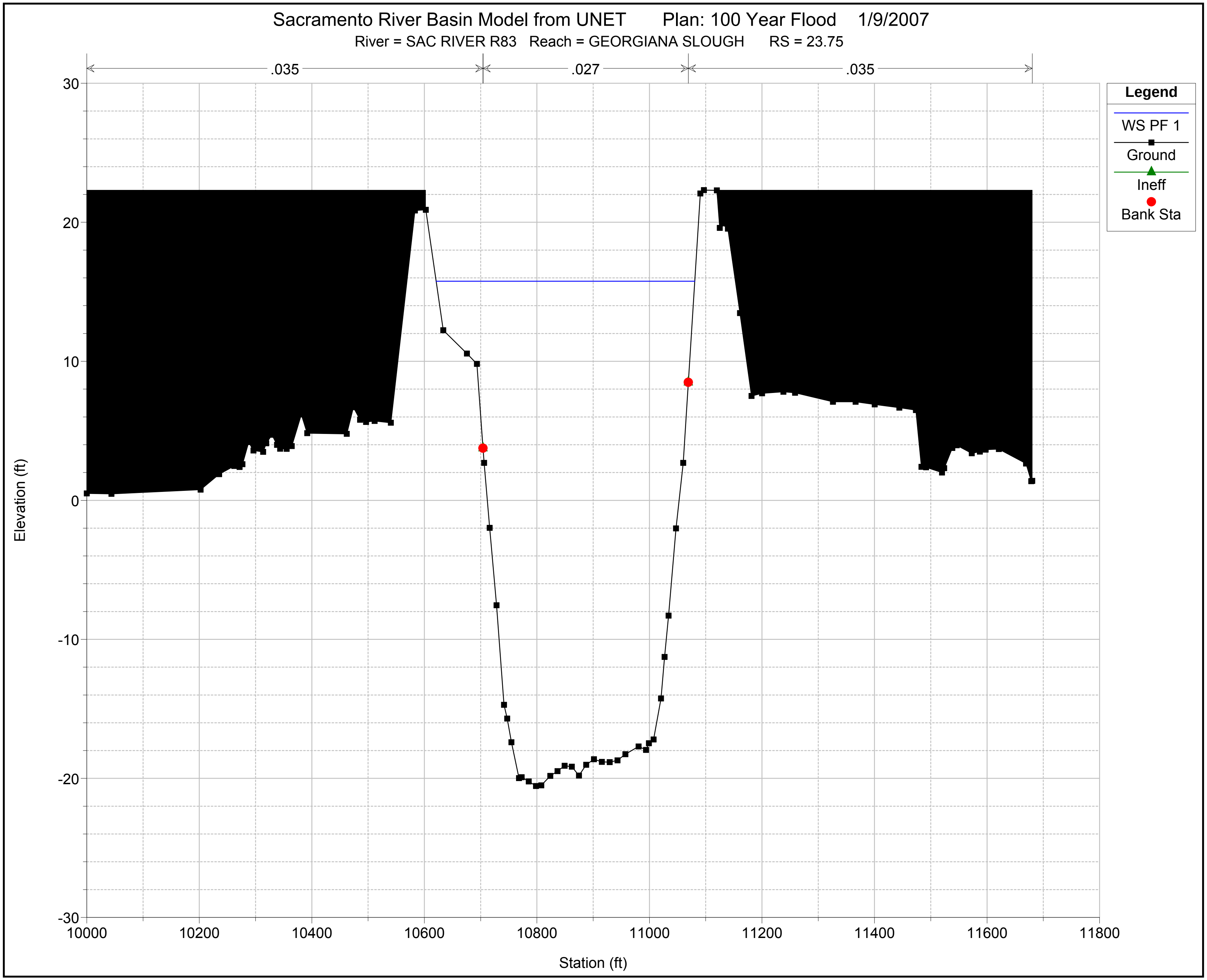


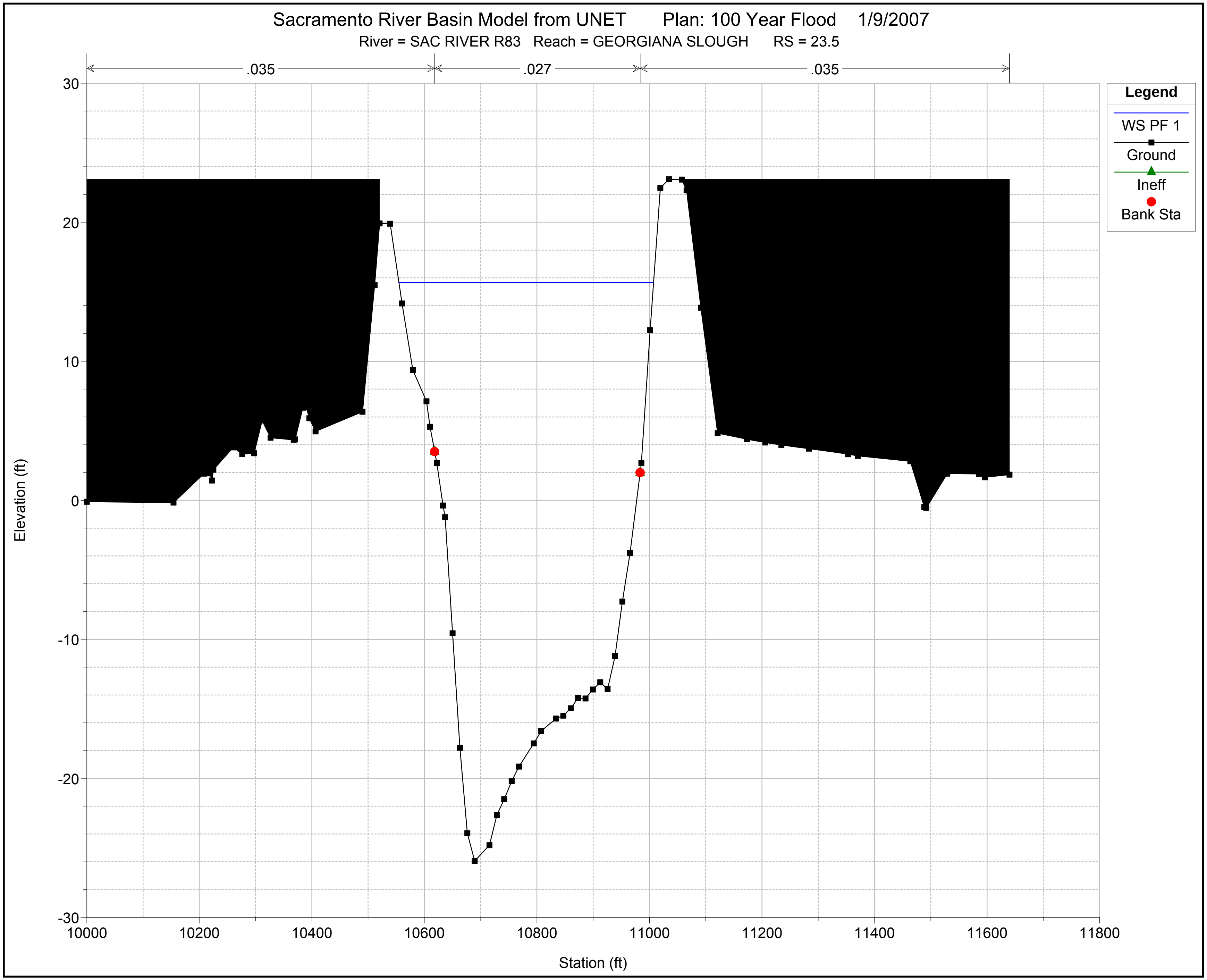
HEC-RAS Profile Drawing for Sacramento River RM 22.7



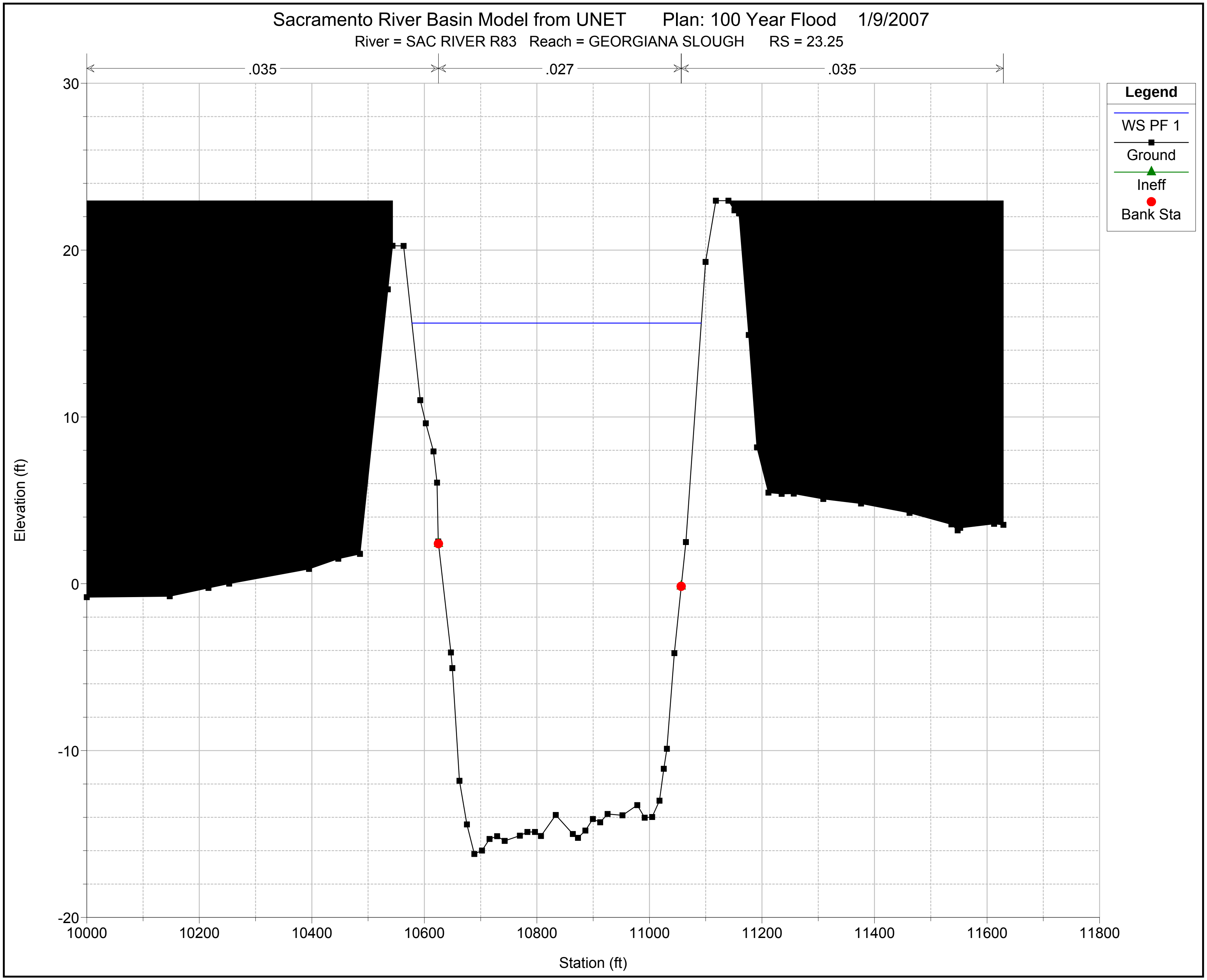
HEC-RAS Profile Drawing for Sacramento River RM 23.2



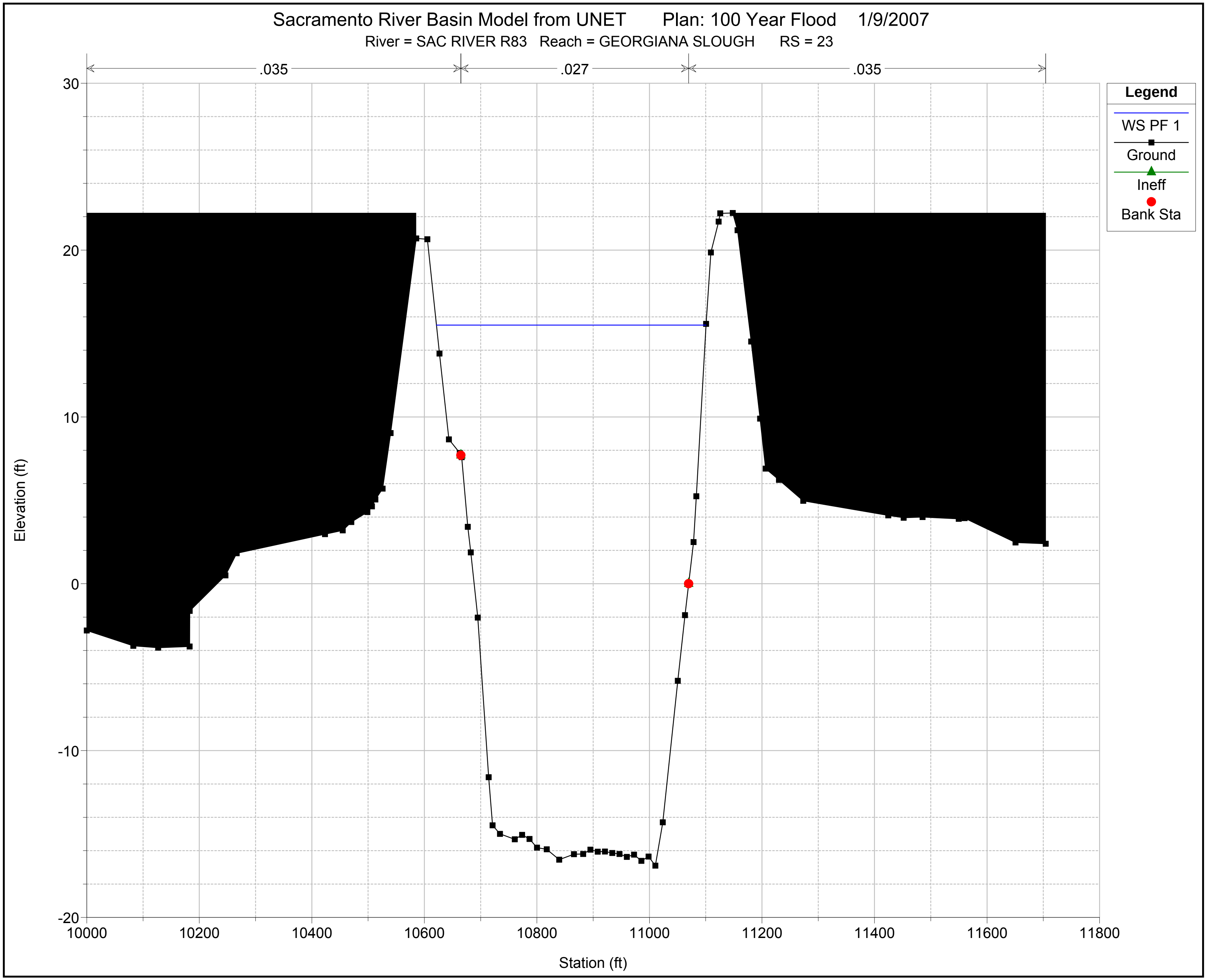

Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = SAC RIVER R83 Reach = GEORGIANA SLOUGH RS = 23.75
.035
.027
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
30
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800

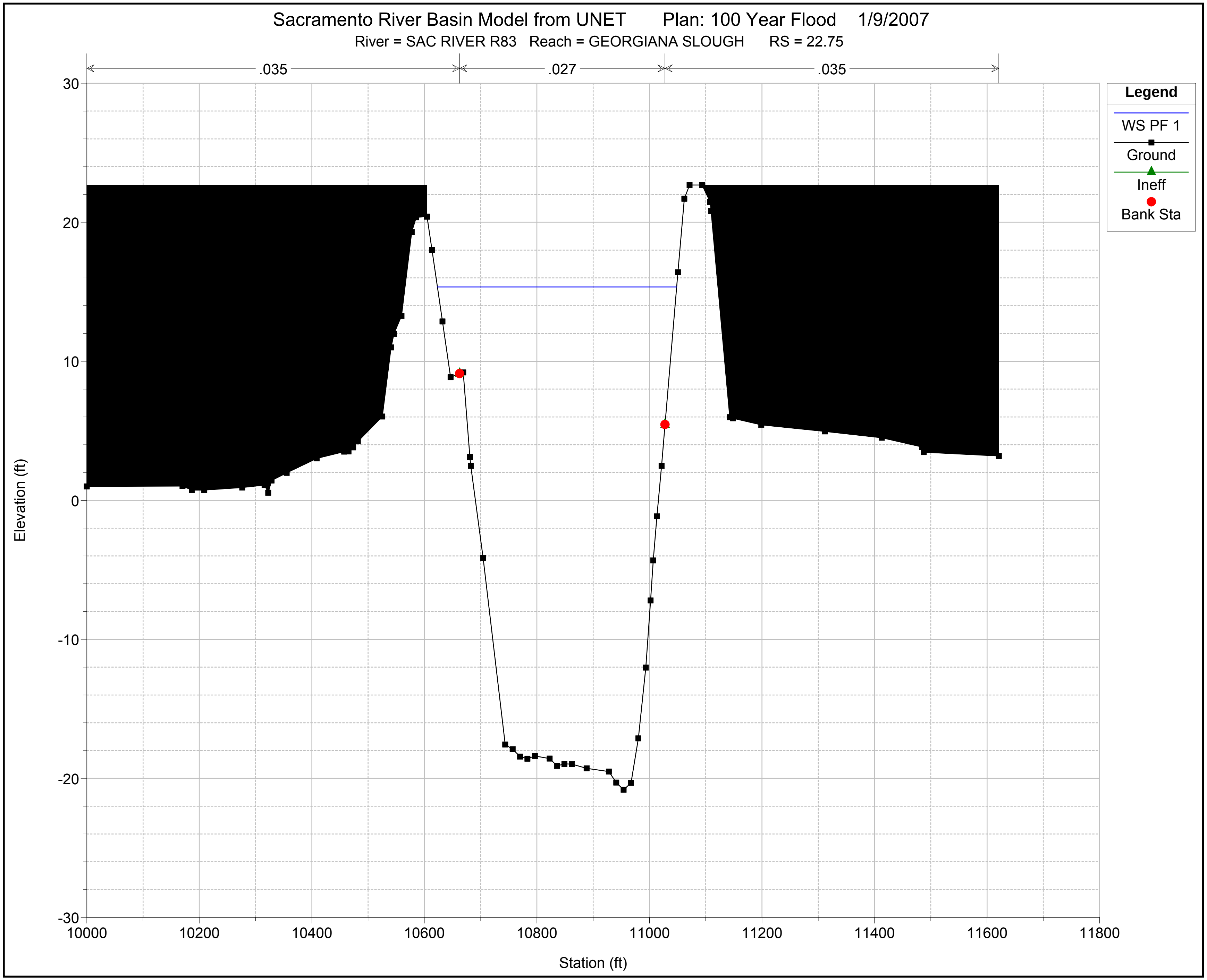


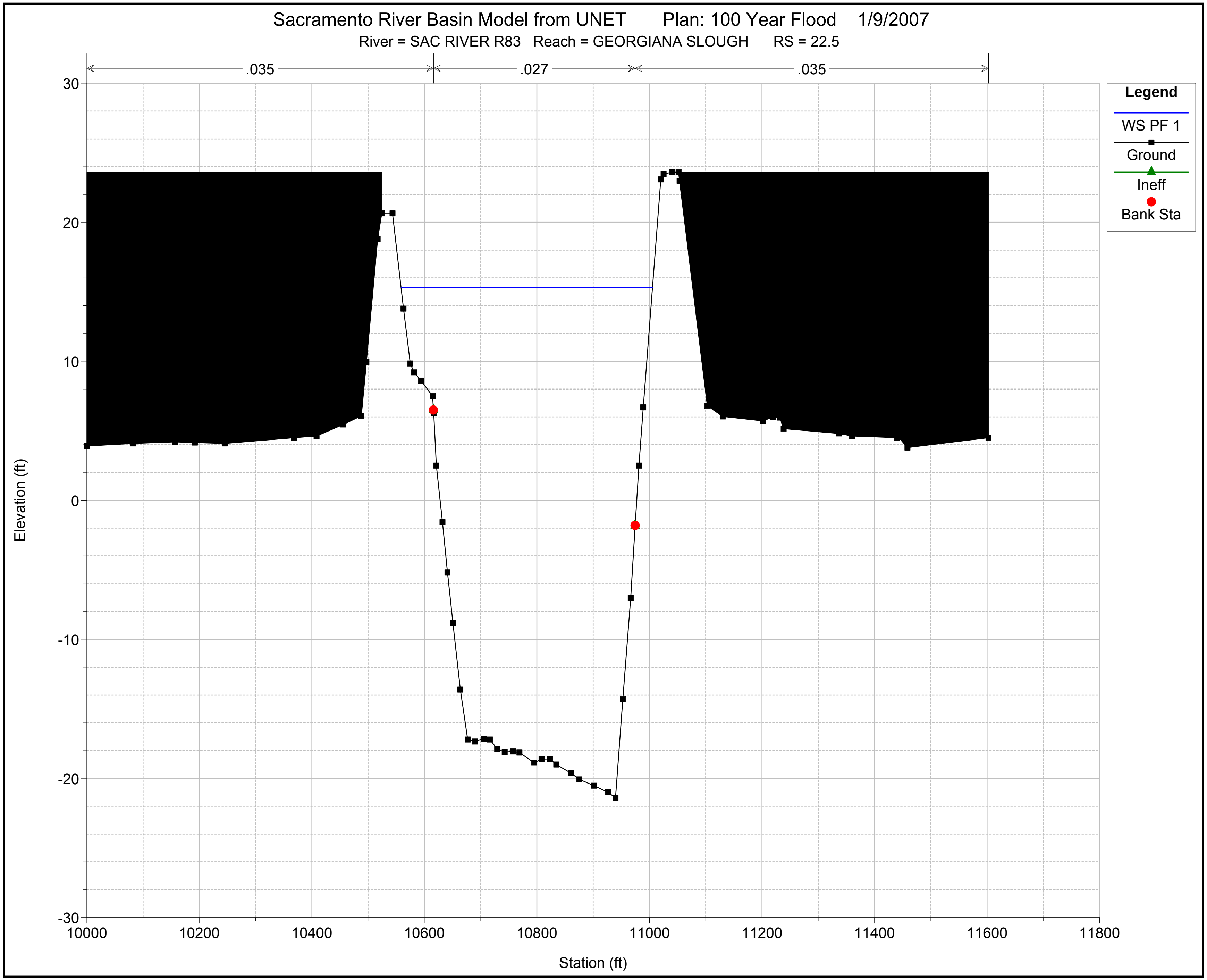
Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = SAC RIVER R83 Reach = GEORGIANA SLOUGH RS = 23.5
.035
.027
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
30
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800
Station (ft)

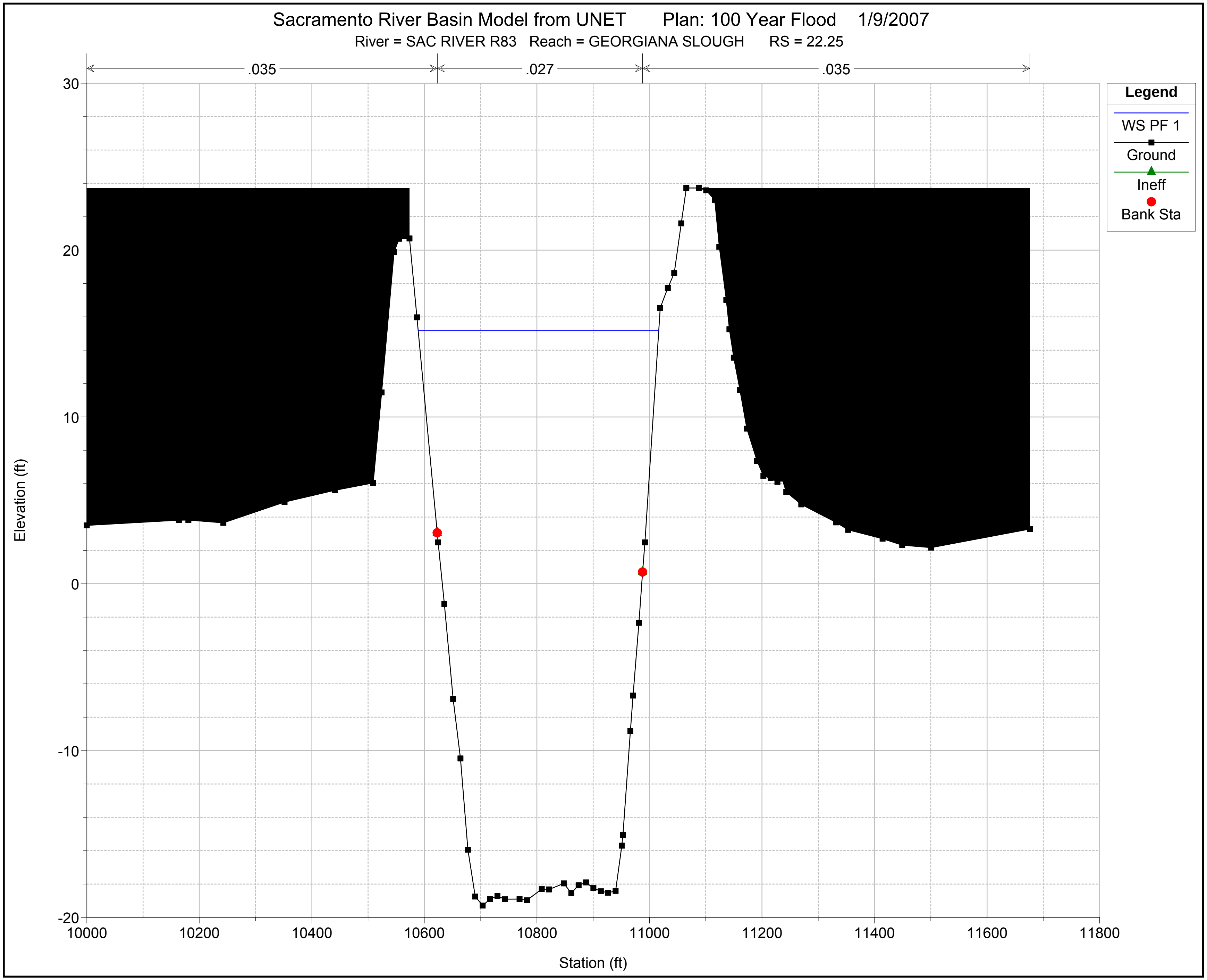


Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = SAC RIVER R83 Reach = GEORGIANA SLOUGH RS = 23.25
.035
.027
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
30
20
10
0
-10
-20
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800
Station (ft)

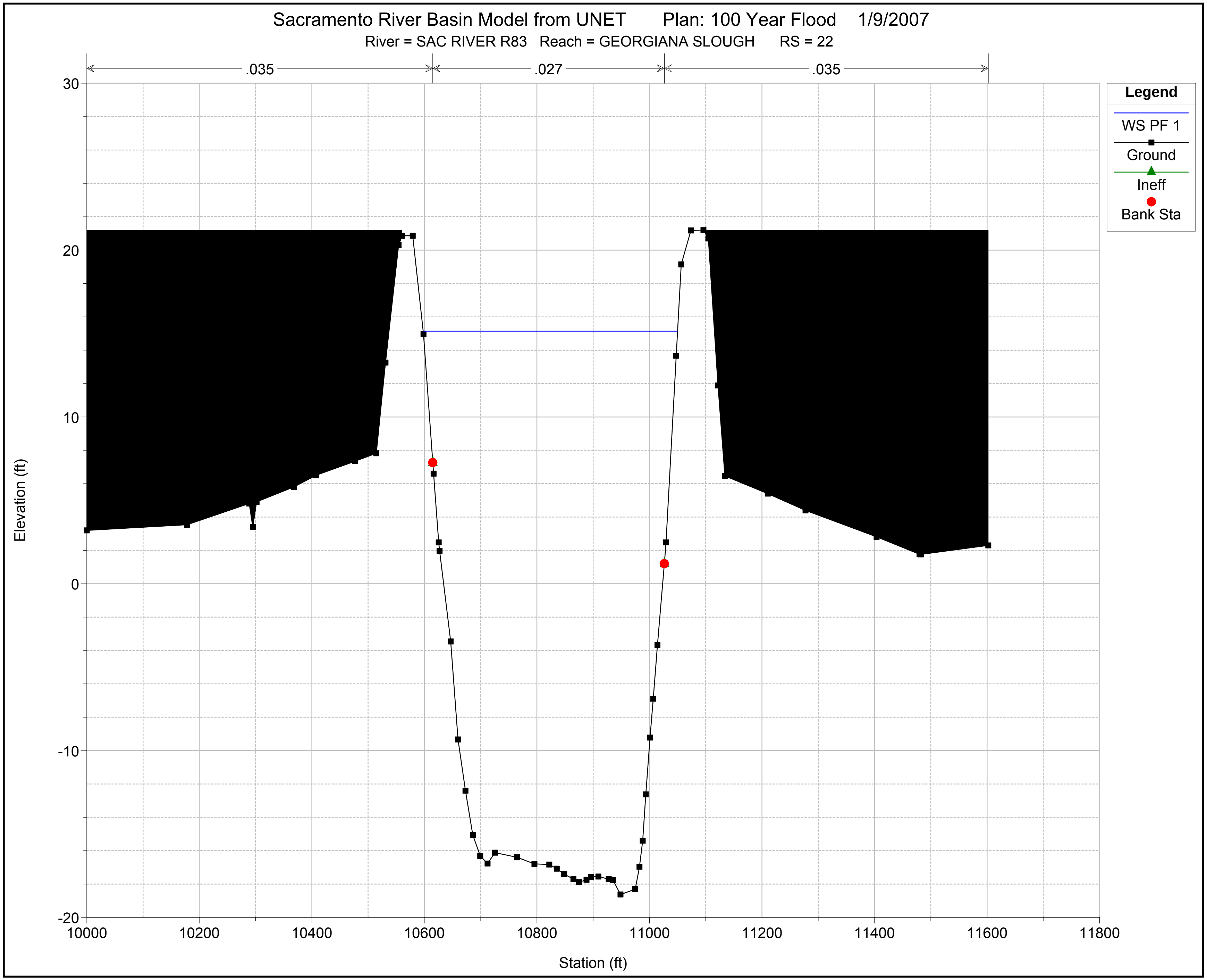

Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = SAC RIVER R83 Reach = GEORGIANA SLOUGH RS = 23
.035
.027
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
30
20
10
0
-10
-20
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800


Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = SAC RIVER R83 Reach = GEORGIANA SLOUGH RS = 22.75
.035
.027
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
30
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800
Station (ft)

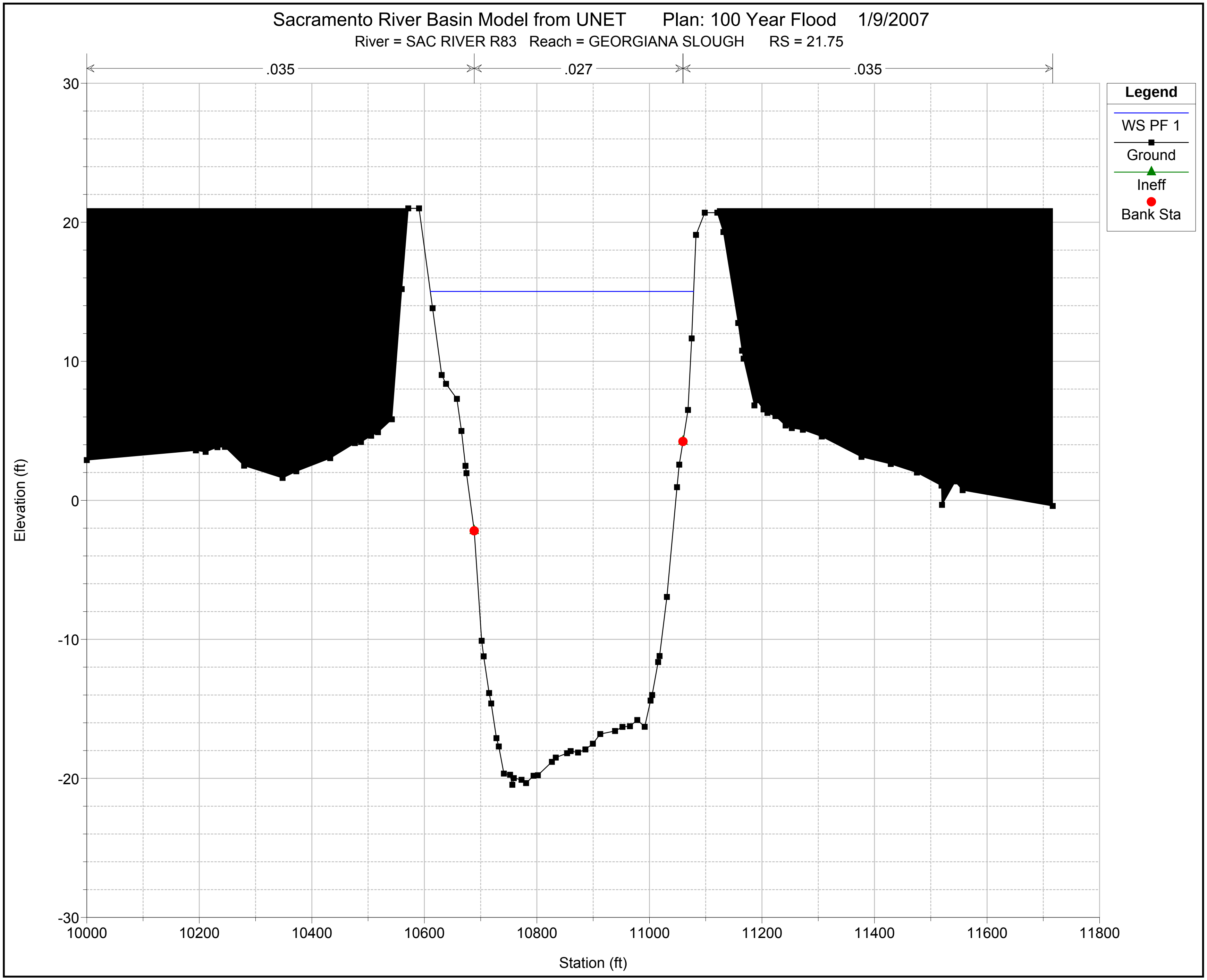

Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = SAC RIVER R83 Reach = GEORGIANA SLOUGH RS = 22.5
.035
.027
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
30
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800
Station (ft)

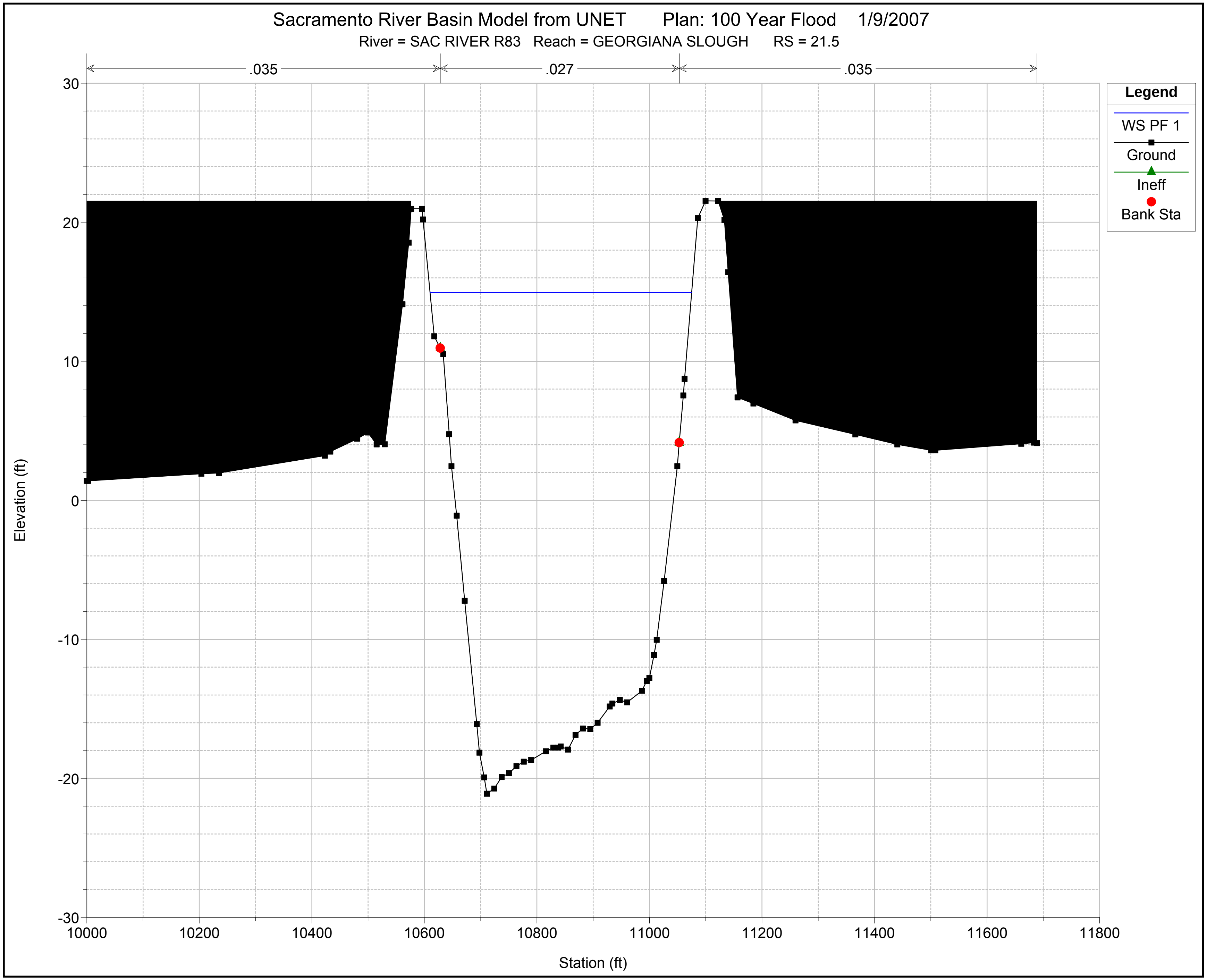


Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = SAC RIVER R83 Reach = GEORGIANA SLOUGH RS = 22.25
.035
.027
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
30
20
10
0
-10
-20
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800
Station (ft)

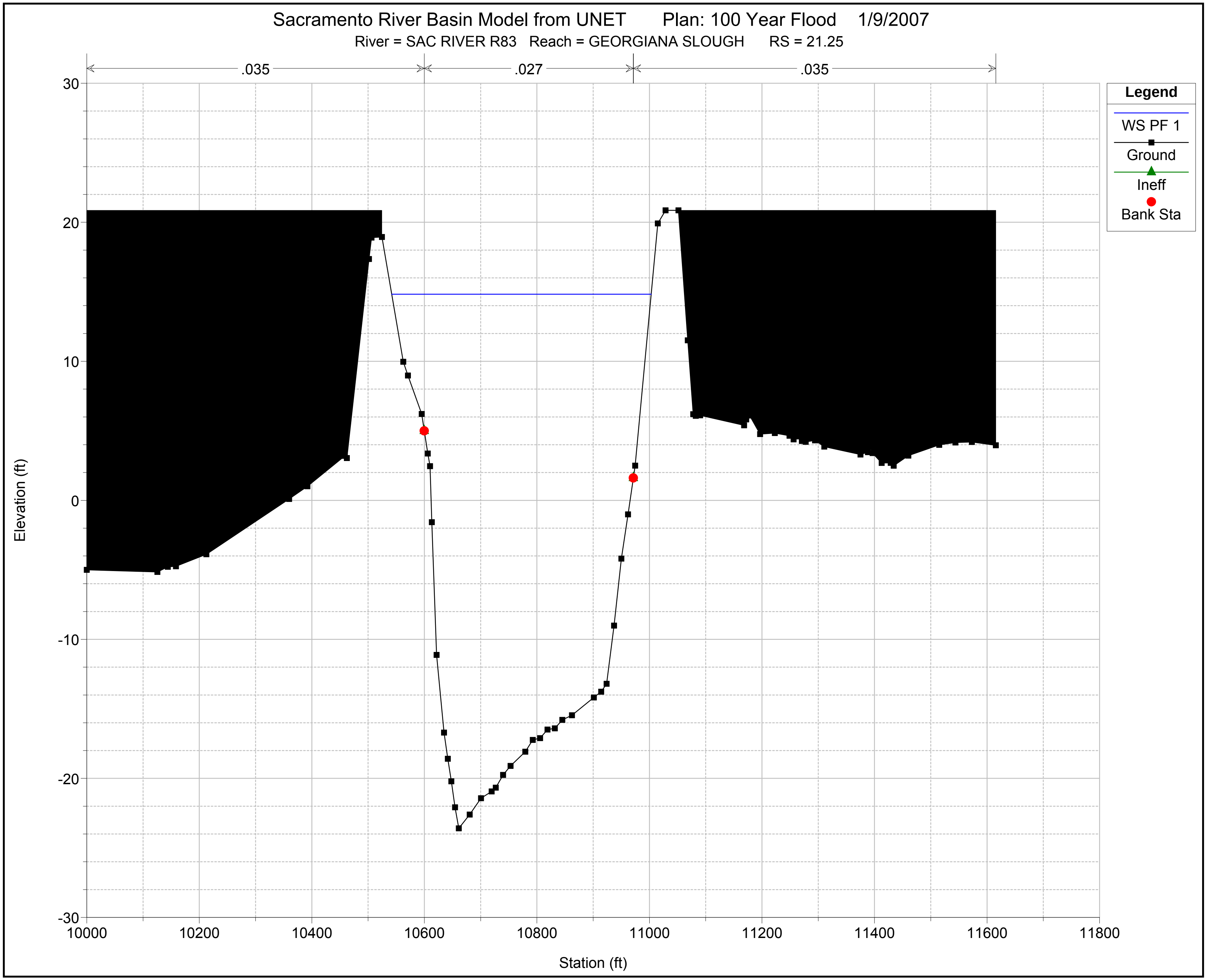


Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = SAC RIVER R83 Reach = GEORGIANA SLOUGH RS = 22
.035
.027
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
30
20
10
0
-10
-20
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800

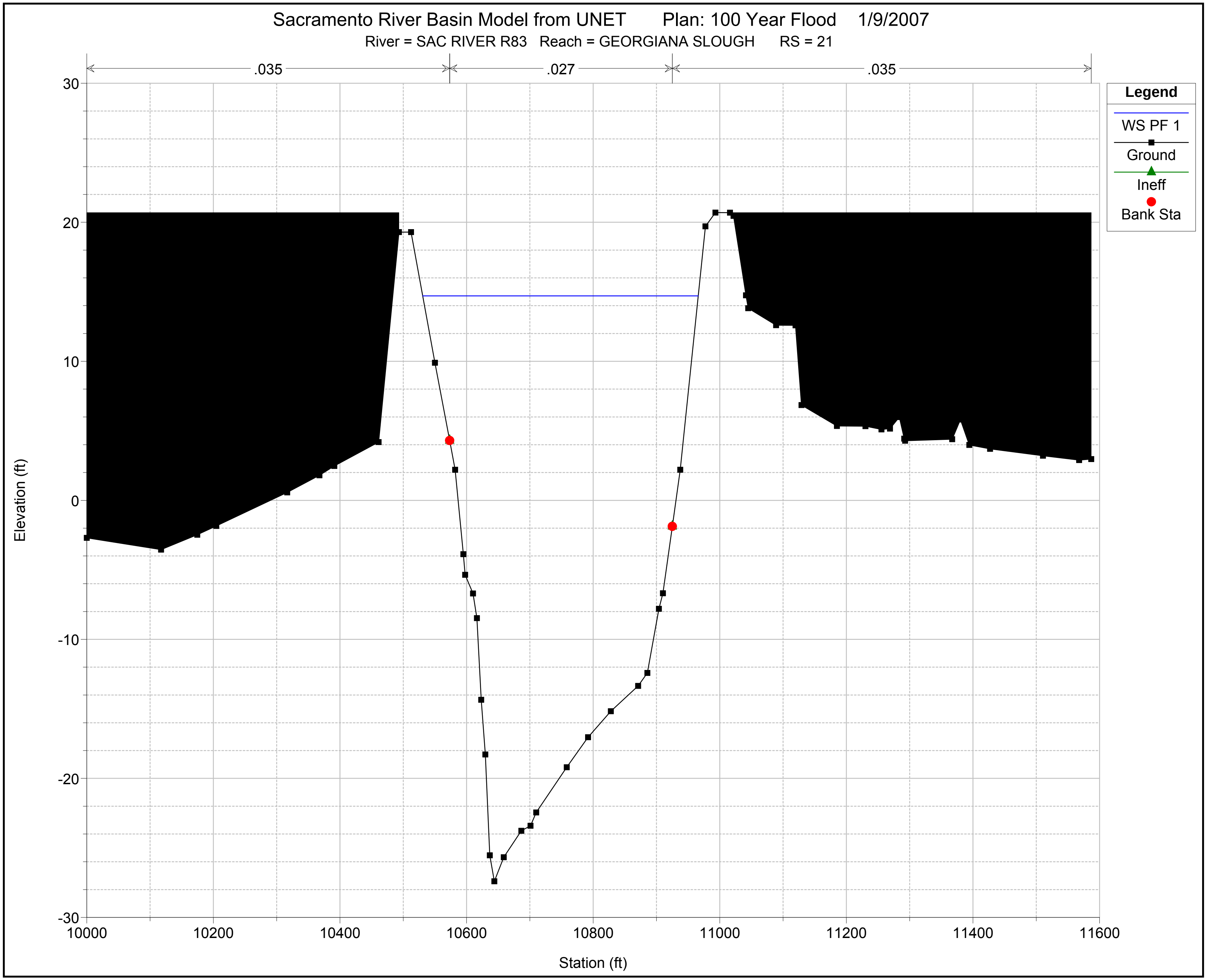

Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = SAC RIVER R83 Reach = GEORGIANA SLOUGH RS = 21.75
.035
.027
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
30
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800



Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = SAC RIVER R83 Reach = GEORGIANA SLOUGH RS = 21.5
.035
.027
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
30
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800



Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = SAC RIVER R83 Reach = GEORGIANA SLOUGH RS = 21.25
.035
.027
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
30
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
11800



Sacramento River Basin Model from UNET Plan: 100 Year Flood 1/9/2007
River = SAC RIVER R83 Reach = GEORGIANA SLOUGH RS = 21
.035
.027
.035
Legend
WS PF 1
Ground
Ineff
Bank Sta
Elevation (ft)
30
20
10
0
-10
-20
-30
10000
10200
10400
10600
10800
11000
11200
11400
11600
Station (ft)

Sub-Appendix A2. Cost Engineering

Total Project Cost Summary

Cost and Schedule Risk Analysis Report

WALLA WALLA COST ENGINEERING MANDATORY CENTER OF EXPERTISE

COST AGENCY TECHNICAL REVIEW

CERTIFICATION STATEMENT

For Project No. 105606

SPK – Sacramento River Bank Protection
Seven Economically Feasible Sub-Basins (~7,204LF)

The Sacramento River Bank Protection Project presented by Sacramento District represents an approximate 7,204 linear feet of protection deemed as the economically justified portion of the authorization. It has undergone a successful Cost Agency Technical Review (Cost ATR), performed by the Walla Walla District Cost Engineering Mandatory Center of Expertise (Cost MCX) team. The Cost ATR included study of the project scope, report, cost estimates, schedules, escalation, and risk-based contingencies. This certification signifies the cost products meet the quality standards as prescribed in ER 1110-2-1150 Engineering and Design for Civil Works Projects and ER 1110-2-1302 Civil Works Cost Engineering.

As of March 19, 2019, the Cost MCX certifies the estimated total project cost:

FY 2019 Price Level: $51,048,000
Fully Funded Amount: $60,255,000

It remains the responsibility of the District to correctly reflect these cost values within the Final Report and to implement effective project management controls and implementation procedures including risk management throughout the life of the project.

JACOBS.MICHAEL.PIERRE.1160569537

Digitally signed by JACOBS.MICHAEL.PIERRE.1160569537
DN: c=US, o=U.S. Government, ou=DoD, ou=PKI, ou=USA, cn=JACOBS.MICHAEL.PIERRE.1160569537
Date: 2019.03.19 15:53:21 -07'00'

Michael P. Jacobs, PE, CCE
Chief, Cost Engineering MCX
Walla Walla District

PROJECT NO: P2 105606
LOCATION: Seven Economically Feasible Sub-Basins

POC: CHIEF, COST ENGINEERING, Theresa A. Gneiting-James

This Estimate reflects the scope and schedule in report; Post Authorization Change Report

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)						TOTAL PROJECT COST (FULLY FUNDED)			
						Program Year (Budget EC): 2019									
						Effective Price Level Date: 1 OCT 18									
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	**TOTAL ($K)**	Spent Thru: **10/1/2018** ($K)	**TOTAL FIRST COST** ($K)	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*				*M*	*N*	*O*
02	RELOCATIONS	$309	$96	31%	$405	2.0%	$315	$98	$413	$0	$413	17.1%	$369	$114	$483
06	FISH & WILDLIFE FACILITIES	$4,011	$1,243	31%	$5,254	2.0%	$4,091	$1,268	$5,360	$0	$5,360	22.3%	$5,004	$1,551	$6,555
11	LEVEES & FLOODWALLS	$2,384	$739	31%	$3,123	2.0%	$2,431	$754	$3,185	$0	$3,185	17.1%	$2,848	$883	$3,730
16	BANK STABILIZATION	$17,019	$5,276	31%	$22,295	2.0%	$17,361	$5,382	$22,742	$0	$22,742	15.0%	$19,958	$6,187	$26,144
	CONSTRUCTION ESTIMATE TOTALS:	$23,722	$7,354		$31,076	2.0%	$24,199	$7,502	$31,700	$0	$31,700	16.4%	$28,178	$8,735	$36,914
01	LANDS AND DAMAGES	$4,970	$1,740	35%	$6,710	2.0%	$5,070	$1,774	$6,844	$0	$6,844	11.9%	$5,675	$1,986	$7,661
30	PLANNING, ENGINEERING & DESIGN	$5,459	$1,692	31%	$7,151	3.8%	$5,667	$1,757	$7,423	$0	$7,423	21.2%	$6,868	$2,129	$8,998
31	CONSTRUCTION MANAGEMENT	$3,440	$1,066	31%	$4,506	3.8%	$3,571	$1,107	$4,678	$0	$4,678	33.0%	$4,748	$1,472	$6,219
18	CULTURAL RESOURCE PRESERVATION	$301	$93	31%	$394	2.0%	$307	$95	$402	$0	$402	15.2%	$353	$110	$463
	PROJECT COST TOTALS:	$37,892	$11,945	32%	$49,837		$38,813	$12,235	**$51,048**	$0	**$51,048**	18.0%	$45,823	$14,432	$60,255

CHIEF, COST ENGINEERING, Theresa A. Gneiting-James

PROJECT MANAGER, Steve Osgood

CHIEF, REAL ESTATE, Diane Simpson

ESTIMATED TOTAL PROJECT COST: **$60,255**

CHIEF, ENGINEERING, Rick Poeppelman

**** CONTRACT COST SUMMARY ****

PROJECT: Sacramento River Bank Protection Project
LOCATION: Seven Economically Feasible Sub-Basins
This Estimate reflects the scope and schedule in report; Post Authorization Change Report

DISTRICT: SPK Sacramento District PREPARED: 6/14/2018
POC: CHIEF, COST ENGINEERING, Theresa A. Gneiting-James

Civil Works Work Breakdown Structure			ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
			Estimate Prepared: Effective Price Level:			5/23/2018 10/1/2017	Program Year (Budget EC): Effective Price Level Date:			2019 1 OCT 18					
WBS NUMBER	Civil Works Feature & Sub-Feature Description		COST ($K)	CNTG ($K)	RISK BASED 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*
	CONTRACT GROUP 1														
16	BANK STABILIZATION	Butte Basin, Sacramento River 152.8 L {198 lf}	$1,075	$333	31%	$1,409	2.0%	$1,097	$340	$1,437	2022Q3	10.4%	$1,211	$375	$1,586
16	BANK STABILIZATION	Butte Basin, Sacramento River 163 L {1,213 lf}	$1,562	$484	31%	$2,046	2.0%	$1,593	$494	$2,087	2022Q3	10.4%	$1,759	$545	$2,304
06	FISH & WILDLIFE FACILITIES	Butte Basin, Sacramento River 152.8 L	$24	$7	31%	$31	2.0%	$24	$8	$32	2022Q3	10.4%	$27	$8	$35
06	FISH & WILDLIFE FACILITIES	Butte Basin, Sacramento River 163.0 L	$144	$45	31%	$189	2.0%	$147	$46	$192	2022Q3	10.4%	$162	$50	$212
06	FISH & WILDLIFE FACILITIES	Butte Basin, Sacramento River 152.8 L - Revegeation & Plant Establishment	$67	$21	31%	$88	2.0%	$68	$21	$90	2025Q1	18.8%	$81	$25	$106
06	FISH & WILDLIFE FACILITIES	Butte Basin, Sacramento River 163 L - Revegeation & Plant Establishment	$217	$67	31%	$284	2.0%	$221	$69	$290	2025Q1	18.8%	$263	$82	$345
06	FISH & WILDLIFE FACILITIES	Butte Basin, Sacramento River 152.8 L - Monitoring	$74	$23	31%	$97	2.0%	$75	$23	$99	2025Q1	18.8%	$90	$28	$118
06	FISH & WILDLIFE FACILITIES	Butte Basin, Sacramento River 163.0 L - Monitoring	$455	$141	31%	$596	2.0%	$464	$144	$608	2025Q1	18.8%	$552	$171	$722
	CONSTRUCTION ESTIMATE TOTALS:		$3,618	$1,122	31%	**$4,739**		$3,690	$1,144	$4,835			$4,144	$1,285	$5,429
01	LANDS AND DAMAGES	Butte Basin, Sacramento River 152.8 L	$142	$50	35%	$192	2.0%	$145	$51	$196	2021Q3	7.2%	$155	$54	$210
01	LANDS AND DAMAGES	Butte Basin, Sacramento River 163 L	$568	$199	35%	$767	2.0%	$579	$203	$782	2021Q3	7.2%	$621	$217	$838
30	PLANNING, ENGINEERING & DESIGN														
2.5%	Project Management		$90	$28	31%	$118	3.8%	$93	$29	$122	2022Q1	11.8%	$104	$32	$137
2.0%	Planning & Environmental Compliance		$72	$22	31%	$94	3.8%	$75	$23	$98	2022Q1	11.8%	$84	$26	$109
8.5%	Engineering & Design		$308	$95	31%	$403	3.8%	$320	$99	$419	2022Q1	11.8%	$358	$111	$468
0.5%	Reviews, ATRs, IEPRs, VE		$18	$6	31%	$24	3.8%	$19	$6	$24	2022Q1	11.8%	$21	$6	$27
0.5%	Life Cycle Updates (cost, schedule, risks)		$18	$6	31%	$24	3.8%	$19	$6	$24	2022Q1	11.8%	$21	$6	$27
2.0%	Contracting & Reprographics		$72	$22	31%	$94	3.8%	$75	$23	$98	2022Q1	11.8%	$84	$26	$109
3.0%	Engineering During Construction		$109	$34	31%	$143	3.8%	$113	$35	$148	2025Q2	26.0%	$143	$44	$187
2.0%	Planning During Construction		$72	$22	31%	$94	3.8%	$75	$23	$98	2025Q2	26.0%	$94	$29	$123
2.0%	Project Operations		$72	$22	31%	$94	3.8%	$75	$23	$98	2022Q1	11.8%	$84	$26	$109
31	CONSTRUCTION MANAGEMENT														
10.0%	Construction Management		$362	$112	31%	$474	3.8%	$376	$116	$492	2025Q2	26.0%	$473	$147	$620
2.0%	Project Operation:		$72	$22	31%	$94	3.8%	$75	$23	$98	2025Q2	26.0%	$94	$29	$123
2.5%	Project Management		$90	$28	31%	$118	3.8%	$93	$29	$122	2025Q2	26.0%	$118	$36	$154
18	CULTURAL RESOURCE PRESERVATION		$45	$14	31%	$60	2.0%	$46	$14	$61	2022Q3	10.4%	$51	$16	$67
	CONTRACT COST TOTALS:		$5,728	$1,804		**$7,532**		$5,868	$1,848	$7,716			$6,648	$2,092	**$8,740**

**** CONTRACT COST SUMMARY ****

PROJECT: Sacramento River Bank Protection Project
LOCATION: Seven Economically Feasible Sub-Basins
This Estimate reflects the scope and schedule in report; Post Authorization Change Report

DISTRICT: SPK Sacramento District PREPARED: 6/14/2018
POC: CHIEF, COST ENGINEERING, Theresa A. Gneiting-James

Civil Works Work Breakdown Structure			ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)					TOTAL PROJECT COST (FULLY FUNDED)				
			Estimate Prepared: Effective Price Level:			5/23/2018 10/1/2017	Program Year (Budget EC): Effective Price Level Date:			2019 1 OCT 18						
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*
	CONTRACT GROUP 2														
16	BANK STABILIZATION	Butte Basin, Sacramento River 168.3 L {546 LF}	$1,399	$434	31%	$1,832	2.0%	$1,427	$442	$1,869	2023Q3	13.7%	$1,622	$503	$2,125
16	BANK STABILIZATION	Butte Basin, Sacramento River 172.0 L {525 LF}	$745	$231	31%	$975	2.0%	$760	$235	$995	2023Q3	13.7%	$864	$268	$1,131
16	BANK STABILIZATION	Natomas, Sacramento River 78.3 L {657 LF}	$1,253	$388	31%	$1,642	2.0%	$1,278	$396	$1,675	2023Q3	13.7%	$1,453	$451	$1,904
16	BANK STABILIZATION	Sacramento, Sacramento River 56.6 L {86 LF}	$456	$141	31%	$598	2.0%	$466	$144	$610	2023Q3	13.7%	$529	$164	$693
16	BANK STABILIZATION	Southport, Sacramento River 56.5 R {373 LF}	$1,607	$498	31%	$2,105	2.0%	$1,639	$508	$2,147	2023Q3	13.7%	$1,864	$578	$2,442
16	BANK STABILIZATION	Southport, Sacramento River 56.7 R {665 LF}	$4,111	$1,274	31%	$5,385	2.0%	$4,193	$1,300	$5,493	2023Q3	13.7%	$4,768	$1,478	$6,246
06	FISH & WILDLIFE FACILITIES	Butte Basin, Sacramento River 168.3 L	$65	$20	31%	$85	2.0%	$66	$20	$86	2023Q3	13.7%	$75	$23	$98
06	FISH & WILDLIFE FACILITIES	Butte Basin, Sacramento River 172.0 L	$62	$19	31%	$82	2.0%	$64	$20	$83	2023Q3	13.7%	$72	$22	$95
06	FISH & WILDLIFE FACILITIES	Natomas, Sacramento River 78.3 L	$78	$24	31%	$102	2.0%	$79	$25	$104	2023Q3	13.7%	$90	$28	$118
06	FISH & WILDLIFE FACILITIES	Sacramento, Sacramento River 56.6 L	$7	$2	31%	$9	2.0%	$7	$2	$10	2023Q3	13.7%	$8	$3	$11
06	FISH & WILDLIFE FACILITIES	Southport, Sacramento River 56.5 R	$35	$11	31%	$45	2.0%	$35	$11	$46	2023Q3	13.7%	$40	$12	$53
06	FISH & WILDLIFE FACILITIES	Southport, Sacramento River 56.7 R	$90	$28	31%	$118	2.0%	$92	$29	$121	2023Q3	13.7%	$105	$32	$137
06	FISH & WILDLIFE FACILITIES	Butte Basin, Sacramento River 168.3 L - Revegeation & Plant Establishment	$142	$44	31%	$186	2.0%	$145	$45	$190	2026Q1	22.4%	$177	$55	$232
06	FISH & WILDLIFE FACILITIES	Butte Basin, Sacramento River 172.0 L - Revegeation & Plant Establishment	$31	$10	31%	$40	2.0%	$31	$10	$41	2026Q1	22.4%	$39	$12	$50
06	FISH & WILDLIFE FACILITIES	Natomas, Sacramento River 78.3 L - Revegeation & Plant Establishment	$248	$77	31%	$325	2.0%	$253	$78	$331	2026Q1	22.4%	$310	$96	$406
06	FISH & WILDLIFE FACILITIES	Butte Basin, Sacramento River 168.3 L - Monitoring	$205	$64	31%	$269	2.0%	$209	$65	$274	2026Q1	22.4%	$256	$79	$335
06	FISH & WILDLIFE FACILITIES	Butte Basin, Sacramento River 172.0 L - Monitoring	$197	$61	31%	$258	2.0%	$201	$62	$263	2026Q1	22.4%	$246	$76	$322
06	FISH & WILDLIFE FACILITIES	Natomas, Sacramento River 78.3 L - Monitoring	$246	$76	31%	$322	2.0%	$251	$78	$329	2026Q1	22.4%	$307	$95	$402
06	FISH & WILDLIFE FACILITIES	Sacramento, Sacramento River 56.6 L - Monitoring	$32	$10	31%	$42	2.0%	$33	$10	$43	2026Q1	22.4%	$40	$12	$52
06	FISH & WILDLIFE FACILITIES	Southport, Sacramento River 56.5 R & 56.7 R - Monitoring (assumed combined)	$389	$121	31%	$510	2.0%	$397	$123	$520	2026Q1	22.4%	$486	$151	$636
CONSTRUCTION ESTIMATE TOTALS:			$11,398	$3,533	31%	**$14,931**		$11,627	$3,604	$15,231			$13,352	$4,139	$17,492
01	LANDS AND DAMAGES	Butte Basin, Sacramento River 168.3 L	$284	$99	35%	$383	2.0%	$290	$101	$391	2022Q3	10.4%	$320	$112	$432
01	LANDS AND DAMAGES	Butte Basin, Sacramento River 172.0 L	$568	$199	35%	$767	2.0%	$579	$203	$782	2022Q3	10.4%	$640	$224	$863
01	LANDS AND DAMAGES	Natomas, Sacramento River 78.3 L	$142	$50	35%	$192	2.0%	$145	$51	$196	2022Q3	10.4%	$160	$56	$216
01	LANDS AND DAMAGES	Sacramento, Sacramento River 56.6 L	$426	$149	35%	$575	2.0%	$435	$152	$587	2022Q3	10.4%	$480	$168	$648
01	LANDS AND DAMAGES	Southport, Sacramento River 56.5 R	$426	$149	35%	$575	2.0%	$435	$152	$587	2022Q3	10.4%	$480	$168	$648
01	LANDS AND DAMAGES	Southport, Sacramento River 56.7 R	$284	$99	35%	$383	2.0%	$290	$101	$391	2022Q3	10.4%	$320	$112	$432
30	PLANNING, ENGINEERING & DESIGN														
2.5%	Project Management		$285	$88	31%	$373	3.8%	$296	$92	$388	2023Q1	16.1%	$343	$106	$450
2.0%	Planning & Environmental Compliance		$228	$71	31%	$299	3.8%	$237	$73	$310	2023Q1	16.1%	$275	$85	$360
8.5%	Engineering & Design		$969	$300	31%	$1,269	3.8%	$1,006	$312	$1,318	2023Q1	16.1%	$1,167	$362	$1,529
0.5%	Reviews, ATRs, IEPRs, VE		$57	$18	31%	$75	3.8%	$59	$18	$78	2023Q1	16.1%	$69	$21	$90
0.5%	Life Cycle Updates (cost, schedule, risks)		$57	$18	31%	$75	3.8%	$59	$18	$78	2023Q1	16.1%	$69	$21	$90
2.0%	Contracting & Reprographics		$228	$71	31%	$299	3.8%	$237	$73	$310	2023Q1	16.1%	$275	$85	$360
3.0%	Engineering During Construction		$342	$106	31%	$448	3.8%	$355	$110	$465	2026Q2	30.7%	$464	$144	$608
2.0%	Planning During Construction		$228	$71	31%	$299	3.8%	$237	$73	$310	2026Q2	30.7%	$309	$96	$405
2.0%	Project Operations		$228	$71	31%	$299	3.8%	$237	$73	$310	2023Q1	16.1%	$275	$85	$360
31	CONSTRUCTION MANAGEMENT														
10.0%	Construction Management		$1,140	$353	31%	$1,493	3.8%	$1,183	$367	$1,550	2026Q2	30.7%	$1,547	$480	$2,027
2.0%	Project Operation:		$228	$71	31%	$299	3.8%	$237	$73	$310	2026Q2	30.7%	$309	$96	$405
2.5%	Project Management		$285	$88	31%	$373	3.8%	$296	$92	$388	2026Q2	30.7%	$387	$120	$507
18	CULTURAL RESOURCE PRESERVATION		$142	$44	31%	$187	2.0%	$145	$45	$190	2023Q3	13.7%	$165	$51	$216
CONTRACT COST TOTALS:			$17,945	$5,648		**$23,594**		$18,383	$5,786	$24,168			$21,405	$6,732	**$28,137**

**** CONTRACT COST SUMMARY ****

PROJECT: Sacramento River Bank Protection Project
LOCATION: Seven Economically Feasible Sub-Basins
This Estimate reflects the scope and schedule in report; Post Authorization Change Report

DISTRICT: SPK Sacramento District PREPARED: 6/14/2018
POC: CHIEF, COST ENGINEERING, Theresa A. Gneiting-James

Civil Works Work Breakdown Structure			ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
			Estimate Prepared:			5/23/2018	Program Year (Budget EC):			2019					
			Effective Price Level:			10/1/2017	Effective Price Level Date:			1 OCT 18					
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*
	CONTRACT GROUP 3														
02	RELOCATIONS	Yolo, Cache Creek 3.9 L {480 LF}	$309	$96	31%	$405	2.0%	$315	$98	$413	2024Q3	17.1%	$369	$114	$483
06	FISH & WILDLIFE FACILITIES	West Sacramento, Sacramento River 62.9 R {175 LF}	$12	$4	31%	$16	2.0%	$13	$4	$16	2024Q3	17.1%	$15	$5	$19
06	FISH & WILDLIFE FACILITIES	West Sacramento, Sacramento River 63.0 R {87 LF}	$6	$2	31%	$8	2.0%	$6	$2	$8	2024Q3	17.1%	$7	$2	$10
06	FISH & WILDLIFE FACILITIES	West Sacramento, Sacramento River 62.9 R & 63.0 R - Monitoring	$98	$30	31%	$128	2.0%	$100	$31	$131	2027Q1	26.1%	$126	$39	$165
11	LEVEES & FLOODWALLS	Yolo, Cache Creek 3.9 L	$2,384	$739	31%	$3,123	2.0%	$2,431	$754	$3,185	2024Q3	17.1%	$2,848	$883	$3,730
16	BANK STABILIZATION	West Sacramento, Sacramento River 62.9 R	$539	$167	31%	$706	2.0%	$550	$170	$720	2024Q3	17.1%	$644	$200	$843
16	BANK STABILIZATION	West Sacramento, Sacramento River 63.0 R	$384	$119	31%	$503	2.0%	$391	$121	$513	2024Q3	17.1%	$458	$142	$601
								$0							
	CONSTRUCTION ESTIMATE TOTALS:		$3,731	$1,157	31%	**$4,888**		$3,806	$1,180	$4,986			$4,467	$1,385	$5,851
01	LANDS AND DAMAGES	West Sacramento, Sacramento River 62.9 R	$284	$99	35%	$383	2.0%	$290	$101	$391	2023Q3	13.7%	$329	$115	$445
01	LANDS AND DAMAGES	West Sacramento, Sacramento River 63.0 R	$284	$99	35%	$383	2.0%	$290	$101	$391	2023Q3	13.7%	$329	$115	$445
01	LANDS AND DAMAGES	Yolo, Cache Creek 3.9 L	$426	$149	35%	$575	2.0%	$435	$152	$587	2023Q3	13.7%	$494	$173	$667
30	PLANNING, ENGINEERING & DESIGN														
2.5%	Project Management		$93	$29	31%	$122	3.8%	$97	$30	$126	2024Q1	20.4%	$116	$36	$152
2.0%	Planning & Environmental Compliance		$75	$23	31%	$98	3.8%	$78	$24	$102	2024Q1	20.4%	$94	$29	$123
8.5%	Engineering & Design		$317	$98	31%	$415	3.8%	$329	$102	$431	2024Q1	20.4%	$396	$123	$519
0.5%	Reviews, ATRs, IEPRs, VE		$19	$6	31%	$25	3.8%	$20	$6	$26	2024Q1	20.4%	$24	$7	$31
0.5%	Life Cycle Updates (cost, schedule, risks)		$19	$6	31%	$25	3.8%	$20	$6	$26	2024Q1	20.4%	$24	$7	$31
2.0%	Contracting & Reprographics		$75	$23	31%	$98	3.8%	$78	$24	$102	2024Q1	20.4%	$94	$29	$123
3.0%	Engineering During Construction		$112	$35	31%	$147	3.8%	$116	$36	$152	2027Q2	35.7%	$158	$49	$207
2.0%	Planning During Construction		$75	$23	31%	$98	3.8%	$78	$24	$102	2027Q2	35.7%	$106	$33	$138
2.0%	Project Operations		$75	$23	31%	$98	3.8%	$78	$24	$102	2024Q1	20.4%	$94	$29	$123
31	CONSTRUCTION MANAGEMENT														
10.0%	Construction Management		$373	$116	31%	$489	3.8%	$387	$120	$507	2027Q2	35.7%	$525	$163	$688
2.0%	Project Operation:		$75	$23	31%	$98	3.8%	$78	$24	$102	2027Q2	35.7%	$106	$33	$138
2.5%	Project Management		$93	$29	31%	$122	3.8%	$97	$30	$126	2027Q2	35.7%	$131	$41	$172
18	CULTURAL RESOURCE PRESERVATION		$49	$15	31%	$64	2.0%	$50	$15	$65	2024Q3	17.1%	$59	$18	$77
	CONTRACT COST TOTALS:		$6,175	$1,954		**$8,130**		$6,325	$2,001	$8,326			$7,545	$2,385	**$9,930**

**** CONTRACT COST SUMMARY ****

PROJECT: Sacramento River Bank Protection Project
LOCATION: Seven Economically Feasible Sub-Basins
This Estimate reflects the scope and schedule in report; Post Authorization Change Report

DISTRICT: SPK Sacramento District PREPARED: 6/14/2018
POC: CHIEF, COST ENGINEERING, Theresa A. Gneiting-James

Civil Works Work Breakdown Structure			ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
			Estimate Prepared:			5/23/2018	Program Year (Budget EC):			2019	FULLY FUNDED PROJECT ESTIMATE				
			Effective Price Level:			10/1/2017	Effective Price Level Date:			1 OCT 18					
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*
	CONTRACT GROUP 4														
16	BANK STABILIZATION	Yolo, Knights Landing Ridge Cut 0.2 R {768 LF}	$118	$37	31%	$154	2.0%	$120	$37	$157	2025Q3	20.6%	$145	$45	$190
16	BANK STABILIZATION	Rio Oso, Bear River 0.8 L {233 LF}	$590	$183	31%	$773	2.0%	$602	$187	$789	2025Q3	20.6%	$726	$225	$951
16	BANK STABILIZATION	Rio Oso, Feather River 0.6 L {288 LF}	$947	$294	31%	$1,241	2.0%	$966	$300	$1,266	2025Q3	20.6%	$1,166	$361	$1,527
16	BANK STABILIZATION	Rio Oso, Feather River 5.0 L {910 LF}	$2,234	$692	31%	$2,926	2.0%	$2,279	$706	$2,985	2025Q3	20.6%	$2,749	$852	$3,601
06	FISH & WILDLIFE FACILITIES	Yolo, Knights Landing Ridge Cut 0.2 R	$91	$28	31%	$120	2.0%	$93	$29	$122	2025Q3	20.6%	$112	$35	$147
06	FISH & WILDLIFE FACILITIES	Rio Oso, Bear River 0.8 L	$28	$9	31%	$37	2.0%	$29	$9	$38	2028Q1	29.8%	$37	$12	$49
06	FISH & WILDLIFE FACILITIES	Rio Oso, Feather River 0.6 L	$35	$11	31%	$45	2.0%	$35	$11	$46	2028Q1	29.8%	$46	$14	$60
06	FISH & WILDLIFE FACILITIES	Rio Oso, Feather River 0.6 L - Monitoring	$108	$33	31%	$141	2.0%	$110	$34	$144	2028Q1	29.8%	$143	$44	$187
06	FISH & WILDLIFE FACILITIES	Rio Oso, Feather River 5.0 L	$108	$33	31%	$142	2.0%	$110	$34	$144	2028Q1	29.8%	$143	$44	$187
06	FISH & WILDLIFE FACILITIES	Yolo, Knights Landing Ridge Cut 0.2 R	$288	$89	31%	$377	2.0%	$294	$91	$385	2028Q1	29.8%	$381	$118	$500
06	FISH & WILDLIFE FACILITIES	Rio Oso, Bear River 0.8 L - Monitoring	$87	$27	31%	$114	2.0%	$89	$28	$116	2028Q1	29.8%	$115	$36	$151
06	FISH & WILDLIFE FACILITIES	Rio Oso, Feather River 5.0 L - Monitoring	$341	$106	31%	$447	2.0%	$348	$108	$456	2028Q1	29.8%	$452	$140	$592
								$0							
	CONSTRUCTION ESTIMATE TOTALS:		$4,975	$1,542	31%	**$6,517**		$5,075	$1,573	$6,648			$6,215	$1,927	$8,142
01	LANDS AND DAMAGES	Yolo, Knights Landing Ridge Cut 0.2 R	$426	$149	35%	$575	2.0%	$435	$152	$587	2024Q2	16.2%	$505	$177	$682
01	LANDS AND DAMAGES	Rio Oso, Bear River 0.8 L	$142	$50	35%	$192	2.0%	$145	$51	$196	2024Q2	16.2%	$168	$59	$227
01	LANDS AND DAMAGES	Rio Oso, Feather River 0.6 L	$284	$99	35%	$383	2.0%	$290	$101	$391	2024Q2	16.2%	$337	$118	$455
01	LANDS AND DAMAGES	Rio Oso, Feather River 5.0 L	$284	$99	35%	$383	2.0%	$290	$101	$391	2024Q2	16.2%	$337	$118	$455
30	PLANNING, ENGINEERING & DESIGN														
2.5%	Project Management		$124	$38	31%	$162	3.8%	$129	$40	$169	2025Q1	24.8%	$161	$50	$210
2.0%	Planning & Environmental Compliance		$100	$31	31%	$131	3.8%	$104	$32	$136	2025Q1	24.8%	$130	$40	$170
8.5%	Engineering & Design		$423	$131	31%	$554	3.8%	$439	$136	$575	2025Q1	24.8%	$548	$170	$718
0.5%	Reviews, ATRs, IEPRs, VE		$25	$8	31%	$33	3.8%	$26	$8	$34	2025Q1	24.8%	$32	$10	$42
0.5%	Life Cycle Updates (cost, schedule, risks)		$25	$8	31%	$33	3.8%	$26	$8	$34	2025Q1	24.8%	$32	$10	$42
2.0%	Contracting & Reprographics		$100	$31	31%	$131	3.8%	$104	$32	$136	2025Q1	24.8%	$130	$40	$170
3.0%	Engineering During Construction		$149	$46	31%	$195	3.8%	$155	$48	$203	2028Q2	41.0%	$218	$68	$286
2.0%	Planning During Construction		$100	$31	31%	$131	3.8%	$104	$32	$136	2028Q2	41.0%	$146	$45	$192
2.0%	Project Operations		$100	$31	31%	$131	3.8%	$104	$32	$136	2025Q1	24.8%	$130	$40	$170
31	CONSTRUCTION MANAGEMENT														
10.0%	Construction Management		$498	$154	31%	$652	3.8%	$517	$160	$677	2028Q2	41.0%	$729	$226	$955
2.0%	Project Operation:		$100	$31	31%	$131	3.8%	$104	$32	$136	2028Q2	41.0%	$146	$45	$192
2.5%	Project Management		$124	$38	31%	$162	3.8%	$129	$40	$169	2028Q2	41.0%	$182	$56	$238
18	CULTURAL RESOURCE PRESERVATION		$64	$20	31%	$84	2.0%	$65	$20	$85	2025Q3	20.6%	$79	$24	$103
	CONTRACT COST TOTALS:		$8,043	$2,539		**$10,582**		$8,238	$2,600	$10,838			$10,225	$3,223	**$13,448**

US Army Corps
of Engineers®

Sacramento River Bank Protection Project (SRBPP)

Project Cost and Schedule Risk Analysis Report

Prepared for:

U.S. Army Corps of Engineers,
Sacramento District

Prepared by:

U.S. Army Corps of Engineers
Cost Engineering Mandatory Center of Expertise, Walla Walla

December 2016

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

EXECUTIVE SUMMARY

The US Army Corps of Engineers (USACE), Sacramento District, presents this cost and schedule risk analysis (CSRA) report regarding the risk findings and recommended contingencies for the Sacramento River Bank Protection Project (SRBPP) Limited Reevaluation Report (LRR). In compliance with Engineer Regulation (ER) 1110-2-1302 CIVIL WORKS COST ENGINEERING, dated September 15, 2008, a formal risk analysis, *Monte-Carlo* based-study was conducted by the Project Development Team (PDT) on remaining costs. The purpose of this risk analysis study is to present the cost and schedule risks considered, those determined and respective project contingencies at a recommended 80% confidence level of successful execution to project completion.

The SRBPP is a long-range construction project to identify significant erosion problems, prioritize sites, and design and construct bank protection. Corrective measures are applied only to affected banks and levees that are part of the Federal Sacramento River Flood Control Project (SRFCP). Per Section 3031 of the Water Resources Development Act of 2007 (WRDA 2007), an additional 80,000 LF of bank protection was added to the original SRBPP Phase II project authorization. The portion included in this analysis is for some 16 sites with approximately 7,865 LF.

Cost estimates fluctuate over time. During this period of study, minor cost fluctuations can and have occurred. For this reason, contingency reporting is based in cost and per cent values. Should cost vary to a slight degree with similar scope and risks, contingency per cent values will be reported, cost values rounded

Table ES-1. Project Contingency Results

Base Case Project Cost Estimate (Excluding Real Estate)	**$36,869,000**	
Confidence Level	**Project Value ($$)**	**Contingency (%)**
5%	$1,106,000	3%
50%	$5,530,000	15%
80%	**$8,111,000**	**22%**
95%	$10,323,000	28%

KEY FINDINGS/OBSERVATIONS RECOMMENDATIONS

The PDT worked through the risk register on 8 April 2014. It quickly became evident that the team understands the project, the experienced risks and those risks already incorporated into the current designs and estimated costs (i.e., the major risks have already been experienced and mitigated through various means). The key risk drivers identified through sensitivity analysis suggest an 80% confidence level total contingency of $8.1M. Findings indicate no schedule risks that would result in any substantial cost impacts or resulting contingencies.

Cost Risks: From the CSRA, the key or greater Cost Risk items of include:

- EST-1: Quantities - During design or awarded it could be determined additional erosion has occurred and quantities will increase.
- RE-4: Onsite Mitigation - Resource agencies requirements for onsite mitigation continue to evolve, resulting in additional onsite mitigation requirements. ESA consultations have yet to occur. Until consultations occur, restoration ratios have not been established.

Moderate risks, when combined, can also become a cost impact; though no major contributors were noted other than Water and Air Quality, Differing Site Conditions, Offsite Mitigation and Construction Oversight.

Schedule Risks: All schedule risk drivers where either outside the scope of this risk analysis and therefore not modeled or will be resolved prior to schedule impacts being realized. Specific schedule risks identified included:

- PPM-3: Internal Red Tape – Discussions on Economic Justification have delayed schedules. Economically disadvantaged sites are some 5 years or more from implementation, allowing for sufficient time for resolution prior to site implementation. This issue is not a Risk for Economically Justified Sites so will not be considered for this evaluation.
- PPM-4: Project Partnership Agreement Signature – PPA signature is due within the next year and must be signed for project to continue. USACE HQ and State sponsor are currently at an impasse on signature of the PPA due to current ETL policy (levee vegetation requirements). If PPA is not signed, project funding will cease and project schedule will slip. Given the potential huge project impact if PPA is not achieved, modeling this risk is outside scope of this risk analysis and will not be included.
- PR-2: Design Criteria Agreement – Sponsor and USACE agreements on Levee Vegetation. While the sponsor and USACE have managed to work around this issue in the past, it is possible this issue will come to a head; requiring either resolution or termination of this project. That discussion is outside the scope of this risk analysis and will not be modeled here.

MAIN REPORT

1.0 PURPOSE

Under the authority of the US Army Corps of Engineers (USACE), Sacramento District presents this cost and schedule risk analysis, identified major risks and recommendations for the total project cost and schedule contingencies for the Sacramento River Bank Protection Project (SRBPP).

2.0 BACKGROUND

The SRBPP is a long-range construction project to identify significant erosion problems, prioritize sites, and design and construct bank protection. Corrective measures are applied only to affected banks and levees that are part of the Federal Sacramento River Flood Control Project (SRFCP). Per Section 3031 of the Water Resources Development Act of 2007 (WRDA 2007), an additional 80,000 LF of bank protection was added to the original SRBPP Phase II project authorization. The portion included in this analysis is for some 16 sites with approximately 7,865 LF.

3.0 REPORT SCOPE

The scope of the risk analysis report is to identify cost and schedule risks with a resulting recommendation for contingencies at the 80 percent confidence level using the risk analysis processes, as mandated by U.S. Army Corps of Engineers (USACE) Engineer Regulation (ER) 1110-2-1150, Engineering and Design for Civil Works, ER 1110-2-1302, Civil Works Cost Engineering, and Engineer Technical Letter 1110-2-573, Construction Cost Estimating Guide for Civil Works. The report presents the contingency results for cost risks for construction features. The CSRA excludes Real Estate costs and does not include consideration for life cycle costs.

3.1 Project Scope

The formal process included extensive involvement of the PDT for risk identification and the development of the risk register. The analysis process evaluated the Micro Computer Aided Cost Estimating System (MCACES) cost estimate, project schedule, and funding profiles using Crystal Ball software to conduct a *Monte Carlo* simulation and statistical sensitivity analysis, per the guidance in Engineer Technical Letter (ETL) CONSTRUCTION COST ESTIMATING GUIDE FOR CIVIL WORKS, dated September 30, 2008.

The project technical scope, estimates and schedules were developed and presented by the Sacramento District. Consequently, these documents serve as the basis for the risk analysis.

The scope of this study addresses the identification of concerns, needs, opportunities and potential solutions that are viable from an economic, environmental, and engineering viewpoint.

3.2 USACE Risk Analysis Process

The risk analysis process for this study follows the USACE Headquarters requirements as well as the guidance provided by the Cost Engineering MCX. The risk analysis process reflected within this report uses probabilistic cost and schedule risk analysis methods within the framework of the Crystal Ball software. Furthermore, the scope of the report includes the identification and communication of important steps, logic, key assumptions, limitations, and decisions to help ensure that risk analysis results can be appropriately interpreted.

Risk analysis results are also intended to provide project leadership with contingency information for scheduling, budgeting, and project control purposes, as well as to provide tools to support decision making and risk management as the project progresses through planning and implementation. To fully recognize its benefits, cost and schedule risk analysis should 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, cost estimating, budgeting and scheduling.

In addition to broadly defined risk analysis standards and recommended practices, this risk analysis was performed to meet the requirements and recommendations of the following documents and sources:

- Cost and Schedule Risk Analysis Process guidance prepared by the USACE Cost Engineering MCX.

- Engineer Regulation (ER) 1110-2-1302 CIVIL WORKS COST ENGINEERING, dated September 15, 2008.

- Engineer Technical Letter (ETL) CONSTRUCTION COST ESTIMATING GUIDE FOR CIVIL WORKS, dated September 30, 2008.

4.0 METHODOLOGY / PROCESS

The Cost Engineering MCX performed the Cost and Schedule Risk Analysis, relying on local Sacramento District staff to provide expertise and information gathering. The initial risk identification meeting also included qualitative analysis to produce a risk register that served as the draft framework for the risk analysis. Follow on meetings updated project development and refined risk modeling. Participants in the risk identification meeting included:

Risk Register Development Meeting

Tuesday, April 8, 2014

Attendance	Name	Representing
Civil Design	Hans Carota	Sacramento District
Civil Design – Tech Lead	Pamlyn Hill	Sacramento District
Planning	Karin Lee	Sacramento District
Cost Engineer	Joe Reynolds	Sacramento District
Real Estate	Greg Garner	DWR
Environmental	Kip Young	DWR
Planner	Thomas Adams	HDR
Cost Engineer	Robert Vrchoticky	Sacramento District
Real Estate	Kelly Boyd	Sacramento District
Cost Engineer	Tri Duong	Sacramento District
Project Manager	Cynthia Brooks	Sacramento District
Risk Analyst	William Bolte	Cost Engineering MCX

The risk analysis process for this study is intended to determine the probability of various cost outcomes and quantify the required contingency needed in the cost estimate to achieve the desired level of cost confidence. Per regulation and guidance, the P80 confidence level (80% confidence level) is the normal and accepted cost confidence level. District Management has the prerogative to select different confidence levels, pending approval from Headquarters, USACE.

In simple terms, contingency is an amount added to an estimate to allow for items, conditions or events for which the occurrence or impact is uncertain and that experience suggests will likely result in additional costs being incurred or additional time being required. The amount of contingency included in project control plans depends, at least in part, on the project leadership's willingness to accept risk of project overruns. The less risk that project leadership is willing to accept the more contingency should be applied in the project control plans. The risk of overrun is expressed, in a probabilistic context, using confidence levels.

The Cost MCX guidance for cost and schedule risk analysis generally focuses on the 80-percent level of confidence (P80) for cost contingency calculation. It should be noted that use of P80 as a decision criteria is a risk averse approach (whereas the use of P50 would be a risk neutral approach, and use of levels less than 50 percent would be risk seeking). Thus, a P80 confidence level results in greater contingency as compared to a P50 confidence level. The selection of contingency at a particular confidence level is ultimately the decision and responsibility of the project's District and/or Division management.

The risk analysis process uses *Monte Carlo* techniques to determine probabilities and contingency. The *Monte Carlo* techniques are facilitated computationally by a commercially available risk analysis software package (Crystal Ball) that is an add-in to Microsoft Excel. Cost estimates are packaged into an Excel format and used directly for cost risk analysis purposes. The level of detail recreated in the Excel-format schedule is sufficient for risk analysis purposes that reflect the established risk register, but generally less than that of the native format.

The primary steps, in functional terms, of the risk analysis process are described in the following subsections. Risk analysis results are provided in Section 6.

4.1 Identify and Assess Risk Factors

Identifying the risk factors via the PDT is considered a qualitative process that results in establishing a risk register that serves as the document for the quantitative study using the Crystal Ball risk software. Risk factors are events and conditions that may influence or drive uncertainty in project performance. They may be inherent characteristics or conditions of the project or external influences, events, or conditions such as weather or economic conditions. Risk factors may have either favorable or unfavorable impacts on project cost and schedule.

A formal PDT meeting held 8 April 2014 included capable and qualified representatives from multiple project team disciplines and functions, including project management, cost engineering, design, environmental compliance, and real estate.

The initial formal meetings focused primarily on risk factor identification using brainstorming techniques, but also included some facilitated discussions based on risk factors common to projects of similar scope and geographic location. Additionally, conference calls and informal meetings were conducted throughout the risk analysis process on an as-needed basis to further facilitate risk factor identification, market analysis, and risk assessment.

4.2 Quantify Risk Factor Impacts

The quantitative impacts (putting it to numbers of cost and time) of risk factors on project plans were analyzed using a combination of professional judgment, empirical data and analytical techniques. Risk factor impacts were quantified using probability distributions (density functions) because risk factors are entered into the Crystal Ball software in the form of probability density functions.

Similar to the identification and assessment process, risk factor quantification involved multiple project team disciplines and functions. However, the quantification process relied more extensively on collaboration between cost engineering and risk analysis team members with lesser inputs from other functions and disciplines. This process used an iterative approach to estimate the following elements of each risk factor:

- Maximum possible value for the risk factor
- Minimum possible value for the risk factor
- Most likely value (the statistical mode), if applicable
- Nature of the probability density function used to approximate risk factor uncertainty
- Mathematical correlations between risk factors
- Affected cost estimate and schedule elements

The resulting product from the PDT discussions is captured within a risk register as presented in Appendix A for both cost and schedule risk concerns. Note that the risk register records the PDT's risk concerns, discussions related to those concerns, and potential impacts to the current cost and schedule estimates. The concerns and discussions support the team's decisions related to event likelihood, impact, and the resulting risk levels for each risk event.

4.3 Analyze Cost Estimate and Schedule Contingency

Contingency is analyzed using the Crystal Ball software, an add-in to the Microsoft Excel format of the cost estimate and schedule. *Monte Carlo* simulations are performed by applying the risk factors (quantified as probability density functions) to the appropriate estimated cost and schedule elements identified by the PDT. Contingencies are calculated by applying only the moderate and high level risks identified for each option (i.e., low-level risks are typically not considered, but remain within the risk register to serve historical purposes as well as support follow-on risk studies as the project and risks evolve).

For the cost estimate, the contingency is calculated as the difference between the P80 cost forecast and the baseline cost estimate. Each option-specific contingency is then allocated on a civil works feature level based on the dollar-weighted relative risk of each feature as quantified by *Monte Carlo* simulation. Standard deviation is used as the feature-specific measure of risk for contingency allocation purposes. This approach results in a relatively larger portion of all the project feature cost contingency being allocated to features with relatively higher estimated cost uncertainty.

5.0 PROJECT ASSUMPTIONS

The following data sources and assumptions were used in determining the cost and schedule risks.

a. The Sacramento District provided a 2 June 2014 Total Project Cost Summary Excel Spreadsheet file electronically. The CSRA was performed on the final TPCS Project Costs (excluding Real Estate).

b. The cost comparisons and risk analyses performed and reflected within this report are based on project experience related to previous Phase 1 projects. The project scoping is well understood, the bulk of risks have been incorporated into more recent design and estimated construction costs. The contingency outcome of 20-25% was expected to be lower than a standard Feasibility Report of 25-35%.

c. The Cost Engineering MCX guidance generally focuses on the eighty-percent level of confidence (P80) for cost contingency calculation. For this risk analysis, the eighty-percent level of confidence (P80) was used. It should be noted that the use of P80 as a decision criteria is a moderately risk averse approach, generally resulting in higher cost contingencies. However, the P80 level of confidence also assumes a small degree of risk that the recommended contingencies may be inadequate to capture actual project costs.

d. Only high and moderate risk level impacts, as identified in the risk register, were considered for the purposes of calculating cost contingency. Low level risk impacts should be maintained in project management documentation, and reviewed at each project milestone to determine if they should be placed on the risk “watch list”.

6.0 RESULTS

The cost and schedule risk analysis results are provided in the following sections. In addition to contingency calculation results, sensitivity analyses are presented to provide decision makers with an understanding of variability and the key contributors to the cause of this variability.

6.1 Risk Register

A risk register is a tool commonly used in project planning and risk analysis. The actual risk register is provided in Appendix A. The complete risk register includes low level risks, as well as additional information regarding the nature and impacts of each risk.

It is important to note that a risk register can be an effective tool for managing identified risks throughout the project life cycle. As such, it is generally recommended that risk registers be updated as the designs, cost estimates, and schedule are further refined, especially on large projects with extended schedules. Recommended uses of the risk register going forward include:

- Documenting risk mitigation strategies being pursued in response to the identified risks and their assessment in terms of probability and impact.
- Providing project sponsors, stakeholders, and leadership/management with a documented framework from which risk status can be reported in the context of project controls.
- Communicating risk management issues.
- Providing a mechanism for eliciting feedback and project control input.
- Identifying risk transfer, elimination, or mitigation actions required for implementation of risk management plans.

6.2 Cost Contingency and Sensitivity Analysis

The result of risk or uncertainty analysis is quantification of the cumulative impact of all analyzed risks or uncertainties as compared to probability of occurrence. These results, as applied to the analysis herein, depict the overall project cost at intervals of confidence (probability).

Table 1 provides the construction cost contingencies calculated for the P80 confidence level and rounded to the nearest thousand. The project cost contingencies for the P5, P50, P80 and P95 confidence levels are also provided for illustrative purposes only.

Contingency was quantified as approximately $8.1 Million at the P80 confidence level (22% of the baseline cost estimate). For comparison, the cost contingency at the P50 and P95 confidence levels was quantified as 15% and 28% of the baseline cost estimate, respectively.

Table 1. Project Cost Contingency Summary

Base Case Project Cost Estimate (Excluding Real Estate)	**$36,869,000**	
Confidence Level	**Project Value ($$)**	**Contingency (%)**
5%	$1,106,000	3%
50%	$5,530,000	15%
80%	**$8,111,000**	**22%**
95%	$10,323,000	28%

6.2.1 Sensitivity Analysis

Sensitivity analysis generally ranks the relative impact of each risk/opportunity as a percentage of total cost uncertainty. The Crystal Ball software uses a statistical measure (contribution to variance) that approximates the impact of each risk/opportunity contributing to variability of cost outcomes during *Monte Carlo* simulation.

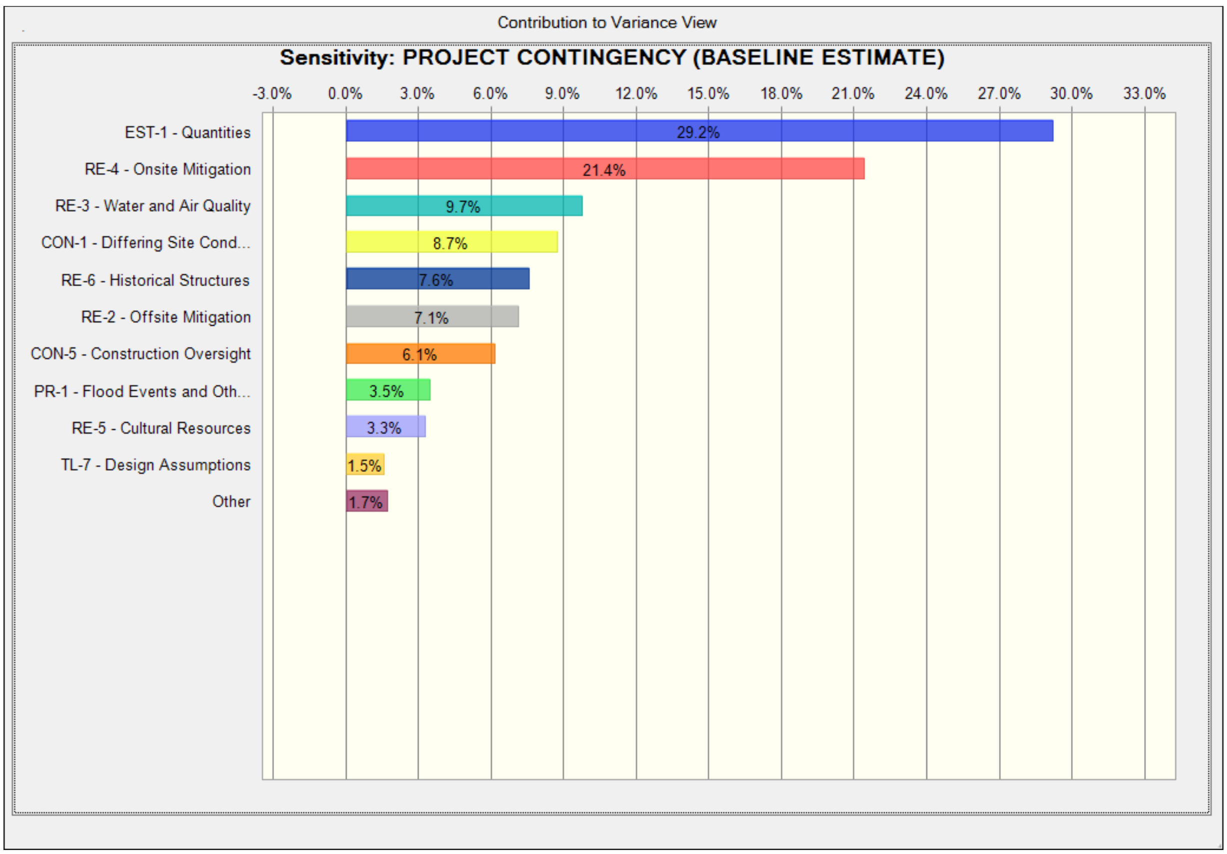
Key cost drivers identified in the sensitivity analysis can be used to support development of a risk management plan that will facilitate control of risk factors and their potential impacts throughout the project lifecycle. Together with the risk register, sensitivity analysis results can also be used to support development of strategies to eliminate, mitigate, accept or transfer key risks.

6.2.2 Sensitivity Analysis Results

The risks/opportunities considered as key or primary cost drivers and the respective value variance are ranked in order of importance in contribution to variance bar charts. Opportunities that have a potential to reduce project cost and are shown with a negative sign; risks are shown with a positive sign to reflect the potential to increase project cost. A longer bar in the sensitivity analysis chart represents a greater potential impact to project cost.

Figure 1 presents a sensitivity analysis for cost growth risk from the high level cost risks identified in the risk register. Likewise, Figure 2 presents a sensitivity analysis for schedule growth risk from the high level schedule risks identified in the risk register.

Figure 1. Cost Sensitivity Analysis



6.3 Schedule and Contingency Risk Analysis

The Sacramento River Bank Protection Project (SRBPP) consists of multiple separate sites with most if not all taking one construction season or less to complete. Individual sites will be addressed as issues arise and delays at any one site will not impact overall project completion schedule, therefore Schedule Risk Analysis becomes somewhat irrelevant for this project.

7.0 MAJOR FINDINGS/OBSERVATIONS/RECOMMENDATIONS

This section provides a summary of significant risk analysis results that are identified in the preceding sections of the report. Risk analysis results are intended to provide project leadership with contingency information for scheduling, budgeting, and project control purposes, as well as to provide tools to support decision making and risk management as projects progress through planning and implementation. Because of the potential for use of risk analysis results for such diverse purposes, this section also reiterates and highlights important steps, logic, key assumptions, limitations, and decisions to help ensure that the risk analysis results are appropriately interpreted.

7.1 Major Findings/Observations

Project cost summaries are provided in Table 2. Additional major findings and observations of the risk analysis are listed below.

The PDT worked through the risk register on 8 April 2014. It quickly became evident that the team understands the project, the experienced risks and those risks already incorporated into the current designs and estimated costs (i.e., the major risks have already been experienced and mitigated through various means). The key risk drivers identified through sensitivity analysis suggest an 80% confidence level total contingency of $8.1M. Findings indicate no schedule risks that would result in any substantial cost impacts or resulting contingencies.

Cost Risks: From the CSRA, the key or greater Cost Risk items of include:

- EST-1: Quantities - During design or awarded it could be determined additional erosion has occurred and quantities will increase.
- RE-4: Onsite Mitigation - Resource agencies requirements for onsite mitigation continue to evolve, resulting in additional onsite mitigation requirements. ESA consultations have yet to occur. Until consultations occur, restoration ratios have not been established.

Moderate risks, when combined, can also become a cost impact; though no major contributors were noted other than Water and Air Quality, Differing Site Conditions, Offsite Mitigation and Construction Oversight.

Schedule Risks: All schedule risk drivers where either outside the scope of this risk analysis and therefore not modeled or will be resolved prior to schedule impacts being realized. Specific schedule risks identified included:

- PPM-3: Internal Red Tape – Discussions on Economic Justification have delayed schedules. Economically disadvantaged sites are some 5 years or more from implementation, allowing for sufficient time for resolution prior to site implementation. This issue is not a Risk for Economically Justified Sites so will not be considered for this evaluation.

- PPM-4: Project Partnership Agreement Signature – PPA signature is due within the next year and must be signed for project to continue. USACE HQ and State sponsor are currently at an impasse on signature of the PPA due to current ETL policy (levee vegetation requirements). If PPA is not signed, project funding will cease and project schedule will slip. Given the potential huge project impact if PPA is not achieved, modeling this risk is outside scope of this risk analysis and will not be included.
- PR-2: Design Criteria Agreement – Sponsor and USACE agreements on Levee Vegetation. While the sponsor and USACE have managed to work around this issue in the past, it is possible this issue will come to a head; requiring either resolution or termination of this project. That discussion is outside the scope of this risk analysis and will not be modeled here.

Table 2. Project Cost Comparison Summary (Uncertainty Analysis)

Most Likely Cost Estimate	$36,869,000		
Confidence Level	**Project Cost**	**Contingency**	**Contingency %**
0%	**$33,919,480**	**($2,949,520)**	**-8.00%**
5%	**$37,975,070**	**$1,106,070**	**3.00%**
10%	**$38,712,450**	**$1,843,450**	**5.00%**
15%	**$39,081,140**	**$2,212,140**	**6.00%**
20%	**$39,818,520**	**$2,949,520**	**8.00%**
25%	**$40,187,210**	**$3,318,210**	**9.00%**
30%	**$40,924,590**	**$4,055,590**	**11.00%**
35%	**$41,293,280**	**$4,424,280**	**12.00%**
40%	**$41,661,970**	**$4,792,970**	**13.00%**
45%	**$42,030,660**	**$5,161,660**	**14.00%**
50%	**$42,399,350**	**$5,530,350**	**15.00%**
55%	**$42,768,040**	**$5,899,040**	**16.00%**
60%	**$43,136,730**	**$6,267,730**	**17.00%**
65%	**$43,505,420**	**$6,636,420**	**18.00%**
70%	**$43,874,110**	**$7,005,110**	**19.00%**
75%	**$44,242,800**	**$7,373,800**	**20.00%**
80%	**$44,980,180**	**$8,111,180**	**22.00%**
85%	**$45,717,560**	**$8,848,560**	**24.00%**
90%	**$46,454,940**	**$9,585,940**	**26.00%**
95%	**$47,192,320**	**$10,323,320**	**28.00%**
100%	**$51,985,290**	**$15,116,290**	**41.00%**

7.2 Recommendations

Risk Management is an all-encompassing, iterative, and life-cycle process of project management. The Project Management Institute's (PMI) *A Guide to the Project Management Body of Knowledge (PMBOK® Guide)*, *4th edition*, states that "project risk management includes the processes concerned with conducting risk management planning, identification, analysis, responses, and monitoring and control on a project." Risk identification and analysis are processes within the knowledge area of risk management. Its outputs pertinent to this effort include the risk register, risk quantification (risk analysis model), contingency report, and the sensitivity analysis.

The intended use of these outputs is implementation by the project leadership with respect to risk responses (such as mitigation) and risk monitoring and control. In short, the effectiveness of the project risk management effort requires that the proactive management of risks not conclude with the study completed in this report.

The Cost and Schedule Risk Analysis (CSRA) produced by the PDT identifies issues that require the development of subsequent risk response and mitigation plans. This section provides a list of recommendations for continued management of the risks identified and analyzed in this study. Note that this list is not all inclusive and should not substitute a formal risk management and response plan.

Risk Management: Project leadership should use of the outputs created during the risk analysis effort as tools in future risk management processes. The risk register should be updated at each major project milestone. The results of the sensitivity analysis may also be used for response planning strategy and development. These tools should be used in conjunction with regular risk review meetings.

Risk Analysis Updates: Project leadership should review risk items identified in the original risk register and add others, as required, throughout the project life-cycle. Risks should be reviewed for status and reevaluation (using qualitative measure, at a minimum) and placed on risk management watch lists if any risk's likelihood or impact significantly increases. Project leadership should also be mindful of the potential for secondary (new risks created specifically by the response to an original risk) and residual risks (risks that remain and have unintended impact following response).

Project Specific: Funding and bidding competition must be periodically re-evaluated to ensure sufficient budget is available to perform the work objectives as authorized. .

APPENDIX A – RISK REGISTER

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions	Project Cost Risk Level*	Schedule Risk Level*
Contract Risks (Internal Risk Items are those that are generated, caused, or controlled within the PDT's sphere of influence.)					
	PROJECT & PROGRAM MGMT				
PPM-1	Scope Definition	Questions remain unsettled about controlling criteria. Project is authorized for additional 80,000 LF yet recent HQ guidance now requires additional bank protection to comply with Corps planning policy (i.e. B/C ratios etc).	District has agreed to perform B/C economic analysis for all sites deemed critical. Estimate is based on 106 representative sites, of which some 12,000LF have economic justification. In the future, sites may change but project costs and risks will be based on 80,000 LF. Given the potential huge project changes if economic justification is required, modeling this risk is outside scope of this risk analysis and will not be modeled.	HIGH	LOW
PPM-2	Project Priorities	Given the long project duration with undefined critical path and conflicts with District priorities; project has received intermittent support. Only after emergency events does this project receive priority status.	Limited resources and project staffing turnover affect continuity, lost efficiencies and schedule. Districts historical averages have been used for the estimate, it is possible design costs could increase but only marginally at most.	LOW	LOW
PPM-3	Internal Red Tape	Internal decision making process has delayed project.	Discussions on Economic Justification have delayed schedules. Economically disadvantaged sites are some 5 years or more from implementation, allowing for sufficient time for resolution prior to site implementation. Not a Risk for Economically Justified Sites so will not be considered for this evaluation.	LOW	MODERATE
PPM-4	Project Partnership Agreement Signature	PPA signature is due within the next year and must be signed for project to continue.	USACE HQ and State sponsor are currently at an impasse on signature of the PPA due to current ETL policy (levee vegetation requirements). If PPA is not signed, project funding will cease and project schedule will slip. Given the potential huge project impact if PPA is not achieved, modeling this risk is outside scope of this risk analysis and will not be included.	LOW	HIGH

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions	Project Cost Risk Level*	Schedule Risk Level*
	CONTRACT ACQUISITION RISKS				
CA-1	Small Business vs. Full and Open	Potential for Small Business Contracts	Much of this work is conducive for small business contracts. The estimate currently assumes full and open contracts. If individual sites are advertised via Small Business, 8(a) contractors, anticipate additional contract acquisition costs, construction costs and district resources for oversight and administration.	HIGH	LOW
CA-2	Numerous Contracts	Contracts will attempt to group sites by Fiscal Year wherever practical to minimize the number of individual contracts.	Multiple sites could be awarded fifty miles or more apart limiting the number of small contractors able to perform the work and potentially lends to more full and open large business contracts.	LOW	LOW

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions	Project Cost Risk Level*	Schedule Risk Level*
	TECHNICAL RISKS				
TL-1	HTRW	HTRW could be encountered during site excavation and construction.	Borings will be done in a proactive attempt to locate any HTRW. Estimate currently assumes no HTRW is located. It is likely HTRW will be encountered, with marginal cost impacts anticipated. When HTRW is encountered it is possible individual sites schedule may slip but overall project schedule will not slip.	MODERATE	LOW
TL-2	Exploratory Borings	Limited exploratory borings have been taken. Additional geotechnical investigation will be required especially in areas of levee realignment.	Depending on exploratory results, site specific design could change. Design changes are anticipated to be marginal.	MODERATE	LOW
TL-3	Borrow/Fill Sources	Borrow sources have not been located. It is typically the contracts responsibility to procure borrow material.	Estimate assumes purchased material. For large fill volumes this could be impossible. Haul distances or commercial prices could increase significantly.	HIGH	LOW
TL-4	Rip Rap Supply	Rock quarry availability over time.	Rock placement has been ongoing since 1960's and will be required for another 40years. Availability of suitable rip rap at current haul distances may not be possible.	MODERATE	LOW
TL-5	Survey Data	Delayed survey data.	For previous project locations obtaining temporary site access has been delayed postponing survey data consequently postponing design and resulting in compressed schedules or construction schedules slipping to next FY. Risk does not necessarily cause overall program schedule impacts but does result in increased PED costs.	MODERATE	LOW
TL-6	Design Criteria	Delays in procurement have resulted in need to update designs for revised criteria.	Design criteria changes have lead to changes for projects put "on the shelf". When projects are awarded additional design updates are required with marginal construction cost increases.	MODERATE	LOW
TL-7	Design Assumptions	Current construction and design are all based on certain core design assumptions and principals. Changes to those assumptions would result in significant design re-work.	Many sites have been constructed. If inspections of constructed sites show current design methodology is not performing as expected designs could change resulting in significant design re-work.	MODERATE	LOW

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions	Project Cost Risk Level*	Schedule Risk Level*
	LANDS AND DAMAGES RISKS				
LD-1	Real Estate Acquisition	Large portions of the existing levee (majority) are still privately owned. Design may require acquisition of new real estate to enable repair requirements.	Almost all areas will require real estate actions; ranging from letters to State asking for easements on State land to acquisition of private property. Real Estate costs have been developed for the representative 106 sites, a majority of which required real estate actions. Any variation in sites will probably experience similar real estate costs. Current design features sections of riverside erosion control that could instead be replaced with landside setback levees requiring additional real estate acquisition with significant cost impacts. Real Estate acquisition is critical driver for all project sites. For Risk Mitigation purposes, site selection is flexible. If Real Estate acquisition is difficult, different sites can be selected. Project is scheduled for 40 years, allowing time for flexible real estate acquisitions. REAL ESTATE CONTINGENCY HAS BEEN DEVELOPED INDEPENDENTLY AND WILL NOT BE INCLUDED IN THIS EVALUATION.	HIGH	LOW
LD-2	Railroad Involvement	Interactions with railroad have been problematic.	Every effort will be made to work outside railroad properties, but there are areas where the railroad is located on the levee. Given the 40 year project duration, PDT is being proactive and pursuing difficult acquisitions with sufficient lead time to address issues prior to fixes at sites. REAL ESTATE CONTINGENCY HAS BEEN DEVELOPED INDEPENDENTLY AND WILL NOT BE INCLUDED IN THIS EVALUATION.	MODERATE	LOW
LD-3	Environmental Mitigation - Real Estate	Real Estate acquisitions for environmental acquisitions can be both on and off site.	Estimate captures cost/scope for environmental mitigation acquisition requirements. It is possible additional real estate will be required. REAL ESTATE CONTINGENCY HAS BEEN DEVELOPED INDEPENDENTLY AND WILL NOT BE INCLUDED IN THIS EVALUATION.	HIGH	LOW
LD-4	Utility Relocations	Large number and variety of requirements for utility relocations.	Variable nature of relocation requirements is difficult to quantify. Real Estate estimates do well in capturing most known utility requirements, but potential unknown utilities remain. REAL ESTATE CONTINGENCY HAS BEEN DEVELOPED INDEPENDENTLY AND WILL NOT BE INCLUDED IN THIS EVALUATION.	MODERATE	LOW

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions	Project Cost Risk Level*	Schedule Risk Level*
	REGULATORY AND ENVIRONMENTAL RISKS				
RE-1	Endangered Species Act	Additional species could be added to ESA.	Additional species could result in additional mitigation costs or design adaptations and changes. It is unlikely to impact cost and no impacts to schedule would be anticipated.	LOW	LOW
RE-2	Offsite Mitigation	Additional offsite mitigation could be required.	As sites information is further refined, it could be discovered additional offsite mitigation efforts will be required to offset impacts. Additional offsite mitigation shouldn't impact schedule.	HIGH	LOW
RE-3	Water and Air Quality	Construction could require air quality credits. Air quality is legislated by local California Resource Board by county and program will overlap multiple regions. Construction could be halted or limited due to water quality impacts.	Baseline Estimate includes costs for monitoring. Marginal additional construction cost impacts should be encountered.	MODERATE	LOW
RE-4	Onsite Mitigation	Depending on Agencies, additional onsite mitigation could be required.	Resource agencies requirements for onsite mitigation continue to evolve, resulting in additional onsite mitigation requirements. ESA consultations have yet to occur. Until consultations occur, restoration ratios have not been established. Additional setback levees in place of riverside repairs may be required.	HIGH	LOW
RE-5	Cultural Resources	It is possible cultural resources could be encountered.	Estimate includes costs for cultural investigations but no costs for mitigations. Cost need to be added for some mitigation for discovery of cultural sites; typically coordinating with local tribes and not removing but protecting resource on site.	MODERATE	LOW
RE-6	Historical Structures	Consultation with State SHIPO has yet to occur.	Additional costs may be necessary for historic documentation of existing levee.	MODERATE	LOW

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions	Project Cost Risk Level*	Schedule Risk Level*
	CONSTRUCTION RISKS				
CON-1	Differing Site Conditions	Heavily dependent on geotechnical design solutions.	Inherent with any geotechnical design comes the possibility of differing site conditions. Given the nature of design solutions (either build new setback levee or overlay existing levees) institute conditions will not be exposed as much as on other typical levee projects. Anticipate lower risks with this item.	MODERATE	LOW
CON-2	Unknown Utilities	Based on previous experience in the project, unknown utilities have rarely been discovered.	For setback levees, it is likely unknown utilities will be encountered, for all other fixes unknown utility impacts are not anticipated.	LOW	LOW
CON-3	Site Access	While access may be remote or round-about for some sites, site and maintenance access is well established.	Minimal Risk is anticipated.	LOW	LOW
CON-4	Construction Windows	All in water work must be completed between April 15 to Nov 30. Depending on contract award dates, durations, and inefficient contractors some contracts could be limited or delayed to the following construction season.	In general this has been a minimal risk, with worst case a one season schedule slip may occur, impacting local contract schedule but not does not impact overall project schedule.	LOW	LOW
CON-5	Construction Oversight	Given the large number of potential sites/contracts per year, submittal turn around times and construction oversight could be an issue.	Based on previous expense, mods and claims have been experienced leading to cost increases.	MODERATE	LOW

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions	Project Cost Risk Level*	Schedule Risk Level*
	ESTIMATE AND SCHEDULE RISKS				
EST-1	Quantities	Differences in quantities.	During design or awarded it could be determined additional erosion has occurred and quantities will increase.	HIGH	LOW
EST-2	Utility Relocations	Large number and variety of requirements for utility relocations.	Variable nature of relocation requirements is difficult to quantify. Potential unknown utilities remain.	MODERATE	LOW
EST-3	Estimate Assumptions and Quantities	Estimate is based on "typical" fixes per reach. A survey has been performed for the project, but has only established a single cross section per length of fix. Specific designs, quantity takeoffs and estimates have not been developed.	Feasibility level estimates have been developed. Quantities could vary marginally.	MODERATE	LOW

Risk No.	Risk/Opportunity Event	Concerns	PDT Discussions	Project Cost Risk Level*	Schedule Risk Level*
ECONOMIC RISKS					
FL-1	Funding Stream	Federal and Sponsor Funding has been sufficient.	Historically project has been funded $5 to $15 M per year which would be sufficient to maintain projected construction schedule assumptions.	LOW	LOW
Programmatic Risks			(External Risk Items are those that are generated, caused, or controlled exclusively outside the PDT's sphere of influence.)		
PR-1	Flood Events and Other Acts of God	Weather events could impact in water construction.	It is possible construction seasons could be delayed or postponed with storm or other weather events resulting in additional construction costs but minimal overall project schedule impacts.	MODERATE	LOW
PR-2	Design Criteria Agreement	Sponsor and USACE agreements on Levee Vegetation	While the sponsor and USACE have managed to work around this issue in the past, it is possible this issue will come to a head; requiring either resolution or termination of this project. That discussion is outside the scope of this risk analysis and will not be modeled here.	HIGH	HIGH

Sub-Appendix A3. Geotechnical

Sacramento River Bank Protection Project
Geotechnical Appendix
23 August 2011
Revised on: September 2018

1. Introduction. The Sacramento River Bank Protection Project (SRBPP) is authorized to protect the riverbanks of the Sacramento River Flood Control Project (SRFCP) from erosion. The majority of the riverbanks along the SRFCP consist of unconsolidated materials that are erosive in nature.

2. Background. In the late 1800's, the flood capacity of the Sacramento River and its tributaries was greatly reduced due to debris from hydraulic mining. This also impaired navigation on the Sacramento River and its tributaries. Therefore, one design feature of the SRFCP was to encourage removal of debris by increasing flow velocity to induce scour. This was accomplished by reducing river meander at key locations by setting the levees near the banks of the rivers. Currently, the majority of debris, including natural sediments, have been removed by scour. The river continues to actively erode the banks as it continues to adjust to natural and human-caused events.

The SRBPP was originally authorized in 1960 to repair eroded riverbanks within the SRFCP and has included subsequent authorizations and phases. The original Phase II authorization was in 1974 and is nearing completion. Congress has authorized an additional 80,000 linear feet of erosion control work for Phase II per the Water Resources Development Act (WRDA) 2007.

3. Erosion Protection. The SRBPP Phase II is authorized to protect the banks (riverbanks and levees if no riverbank exists) within the SRFCP system from erosion. This will be accomplished by either: 1) repair of existing bank by placement of erosion resistant materials or 2) widen the waterside berm by setting the levee back and allow the river to erode the bank as part of the river's natural meandering process.

This Engineering Document Report (EDR) is programmatic. Therefore, geotechnical analysis and design will be conceptual and will be based on available geotechnical information using geotechnical engineering judgment.

Subsequent to this programmatic EDR, site-specific geotechnical analysis will be performed during site-specific EDRs and site-specific Design Document Reports (DDR). The complexity of the geotechnical analysis will be dependent on the site-specific conditions.

Sub-Appendix A4. Hydrology

Sacramento River Bank Protection Project
Phase II

Post-Authorization Change

Hydrology
Appendix

March 2011

STATEMENT OF TECHNICAL REVIEW
COMPLETION OF DISTRICT QUALITY CONTROL REVIEW

The District has completed the Phase II Post-Authorization Change Hydrology Appendix for the Sacramento River Bank Protection Project. Notice is hereby given that (1) a Quality Assurance review has been conducted as defined in the Quality Assurance Plan and (2) a district level technical review that is appropriate to the level of risk and complexity inherent in the project, has been conducted as defined in the project's Quality Management Plan. During the district level technical review, compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This included review of: assumptions, methods, procedures, and material used in analyses, alternatives evaluated, the appropriateness of data used and level obtained, and reasonableness of the result, including whether the product meets the customer's needs consistent with law and existing Corps policy. The district level technical review was accomplished by a senior hydraulic engineer. All comments resulting from QA have been resolved.

The project hydrology covered under this quality control certification includes a description, with documentation, of the Comprehensive Study methodology used to develop frequency estimates and balanced 30-day hydrographs used to develop hydraulics information for the Sacramento River Bank Protection Project.

Steven F Holmstrom — 6/8/2011

Steven F. Holmstrom, P.E. — Date
DQC Reviewer, Sacramento District

Laurine L. White — 8 June 2011

Laurine L. White — Date
Hydrologist, Sacramento District

CERTIFICATION OF DISTRICT QUALITY ASSURANCE REVIEW

I certify that an independent technical review at the District Quality Control level of the project indicated above has been completed and that all technical issues have been identified and resolved. I certify that the quality control process at the District level has been completed.

John M. High — 6/8/2011

John M. High — Date
Chief, Hydrology Section

Sacramento River Bank Protection Project Phase II
Post-Authorization Change
Hydrology Technical Documentation

EXECUTIVE SUMMARY

Scope. This Attachment (hydrology documentation) describes the development of the existing conditions synthetic hydrology for the Sacramento River Bank Protection Project. The project uses the existing hydrology for the Sacramento and San Joaquin River Basins Comprehensive Study (Comp Study). Hydrology documentation includes (1) Yuba River Basin Project General Reevaluation Report (Yuba GRR), Appendix A, "Synthetic Hydrology and Reservoir Operations Technical Documentation," dated April 2004, revised 2008, and (2) Hydrology Technical Documentation, Appendix B1 and B2, for the "Post-Authorization Change Report and Interim General Reevaluation Report, American River Watershed Common Features Project (ARCF GRR), Natomas Basin, Sacramento and Sutter Counties, California," dated August 2010. Documentation referenced here, but not included, is the Comp Study Technical Studies Documentation, Appendices B and C, dated December 2002.

Background. The Sacramento River Flood Control Project is a system of levees, weirs, pumping plants, and bypasses designed to safely convey Sacramento River and tributary flood flows. There are approximately 1,300 miles of project levees in this system, as shown on Figures 1 and 2. The Sacramento River Bank Protection Project (SRBPP) is a Federal program for inspecting the levees and associated natural banks and berms, identifying and ranking erosion problems, and providing remedial fixes. Phase I of the SRBPP was constructed from 1963 to 1975, and consisted of 430,000 linear feet of bank protection. Due to continued erosion problems, SRBPP Phase II was authorized in 1974 to repair an additional 405,000 linear feet of bank protection. The Water Resources Development Act of 2007 authorized an additional 80,000 linear feet of bank protection as part of the Phase II effort.

Comprehensive Study Methodology. The SRBPP is using existing conditions Comp Study hydrology, which is anticipated to be adequate for determining water surface profiles for the levee reaches included in the SRBPP. The existing hydrology for the SRBPP is based upon the storm centering method described in the Comp Study Technical Studies Documentation, Appendices B and C. Appendix B describes the development of unregulated synthetic hydrographs for specific flood frequencies at particular watershed locations, while Appendix C presents the transformation of the unregulated conditions synthetic hydrology to regulated conditions. The Yuba GRR Hydrology Appendix, included in the attached documentation, presents a shorter description of the Comp Study methodology in Chapter 2. The Comp Study synthetic hydrology represents the best available information for the sources of flooding against the levees in the SRBPP. The Common Features hydraulic model (HEC-RAS) was used to route the upstream synthetic flood hydrographs through the open channels, weirs, bypasses and storage areas to develop the water surface profiles down the Sacramento River and its tributaries. Comp Study hydrology has also been used for regional studies, such as the American River Common Features, Yuba River Basin, Sutter Basin, Marysville, and West Sacramento studies.

Synthetic Flood Centerings. Two Comp Study mainstem flood centerings (Ord Ferry and Latitude of Sacramento) and ten tributary flood centerings, including Shanghai-Yuba, were investigated in the development of existing conditions hydrology for the Sacramento watershed covered by the SRBPP levees. ARCF GRR Hydrology Appendix B1, Synthetic Hydrology Technical Documentation discusses the three flood centerings: the Latitude of Sacramento mainstem, the Shanghai-Yuba, and the tributary American River, used to develop hydrographs for the ARCF GRR hydrology. The Comp Study Technical Studies Documentation, Appendix B, discusses the other mainstem flood centering (Ord Ferry) and the rest of the tributary flood centerings.

Synthetic Flood Reservoir Operations. The Comp Study Technical Studies Documentation, Appendix C, discusses the reservoir operations involved in the transformation process of converting the unregulated flood hydrographs to regulated hydrographs. Operation of the reservoirs is as described in Appendix C, with the exception of Folsom Dam and Lake. ARCF GRR Appendix B2, American River Hydrology and Folsom Dam Reservoir Operations, discusses the changes to the American River flood hydrographs and in the operation of Folsom Dam. The concurrent American River flows in the Comp Study centerings include existing conditions operations for Folsom Dam (SAFCA diagram) with a 145,000 cfs maximum objective release and a future condition Joint federal Project (JFP) with a maximum objective release of 160,000 cfs. Development of a new Water Control Diagram is in progress that may change the future condition flows, although the maximum objective release is not expected to change.

Upstream Conditions Assumption. The assumption for upstream conditions is that levees upstream will not fail but will be overtopped as the water surface exceeds the top of levee. This condition was used in earlier studies.

Basis for SRBPP Flood Stages. Hydrology from the 2002 Comp Study, the 2004 Yuba Basin Project, and Folsom Dam modifications from the 2010 ARCF GRR was used by Hydraulic Design Section to develop stages for this analysis. The Comp Study uses the “Composite Floodplain” concept, which recognizes that the stages generated through modeling are not created by a single flood event, but by a combination of several events, each of which shapes the stage at different locations. The stages for the levee stretches shown on Figures 1 and 2 are based on the combination of the two mainstem and ten tributary centerings that resulted in the maximum stage possible at all locations.

In future, the Comp Study Hydrology may be used with UNET modeling to determine boundary conditions for site specific 2D models for hydraulic analysis and design.

Hydrology for Sea Level Change Analysis Report. The 50% and 1% chance flood hydrographs down the Yolo Bypass and Sacramento River from the Comp Study hydrology were used for the Common Features hydraulic model used for the SRBPP Sea Level Change Analysis Report prepared by the Corps Hydraulic Design Section. Inflow assumptions for the Yolo Bypass model included a constant 1,000 cfs flow contribution down the Willow Slough Bypass and 100 cfs flow apiece for Haas, Cache, and Lindsey sloughs.

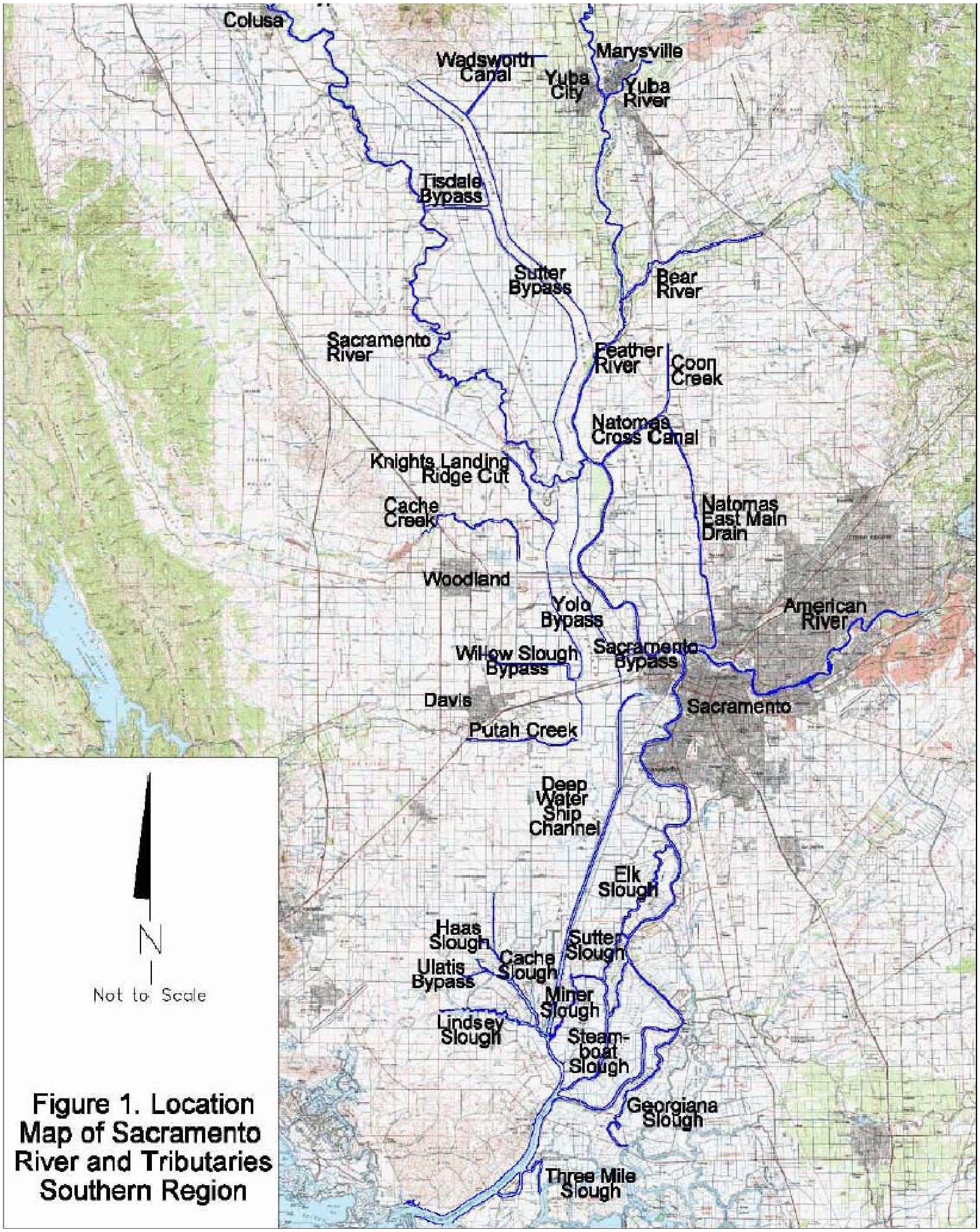


Figure 1. Location Map of Sacramento River and Tributaries Southern Region

Source: Ayers Associates, Inc. *2007 – Field Reconnaissance Report, Erosion Site Inventory and Priority Ranking*, December 18, 2007

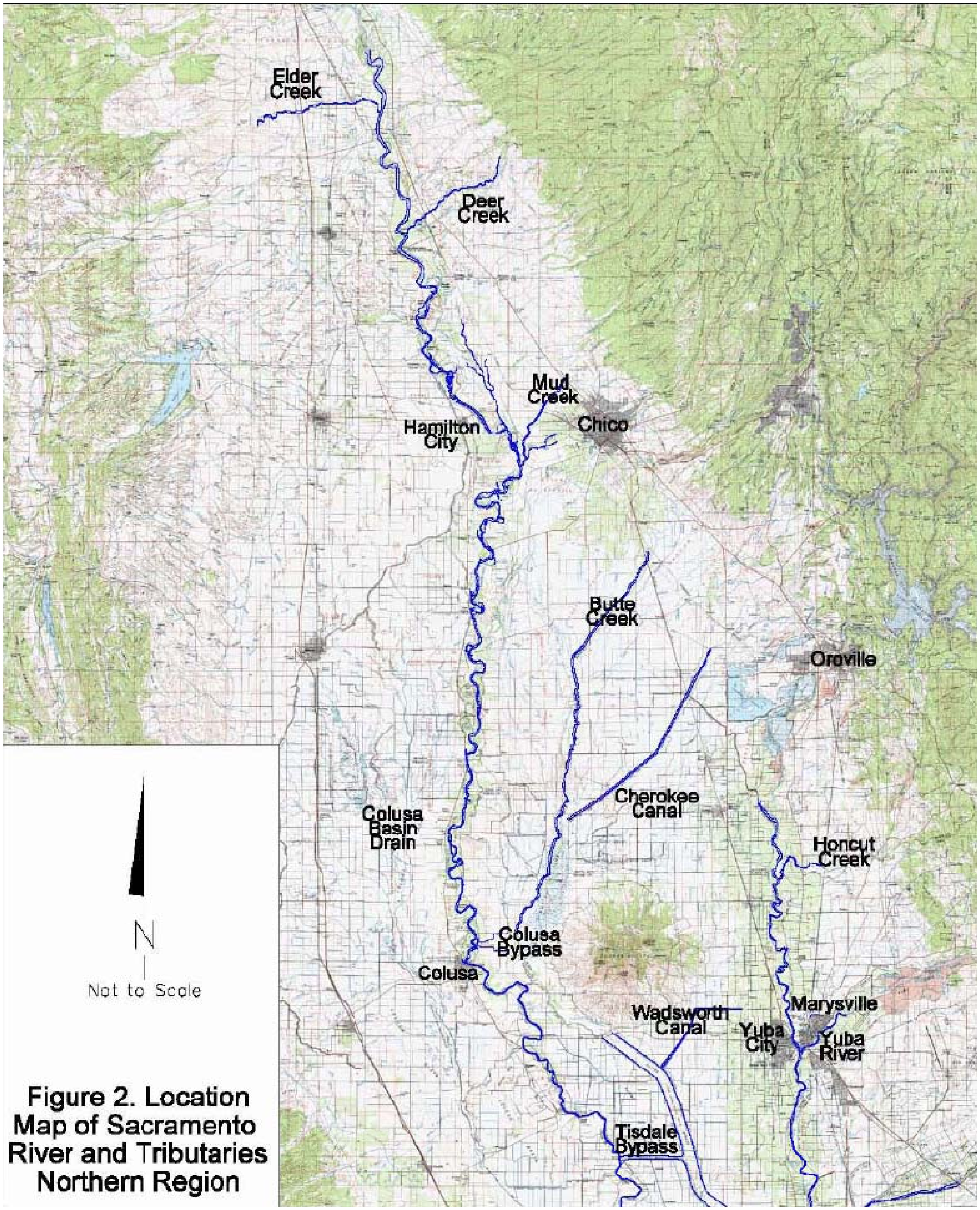


Figure 2. Location Map of Sacramento River and Tributaries Northern Region

Source: Ayers Associates, Inc. *2007 – Field Reconnaissance Report, Erosion Site Inventory and Priority Ranking*, December 18, 2007

The attached hydrology documentation is included below as follows:

Yuba River Basin Project General Reevaluation Report Appendix A, revised June 2008

Post-Authorization change Report and Interim General Reevaluation Report, American River Watershed Common Features Project, Natomas Basin, Sacramento and Sutter Counties, California, Appendix B (Appendices B1 and B2) – Hydrology Technical Documentation, dated August 2010

The Comprehensive Study Technical Documentation is available on the internet at URL: http://130.165.3.37/reports.html.

Yuba River Basin Project
General Reevaluation Report

Appendix A

Synthetic Hydrology and Reservoir Operations
Technical Documentation

April 2004
(Corrected June 2008)

U.S. Army Corps of Engineers
Sacramento District

Corrections

The April 2004 version of this report contained an error in the labeling of Tables 2 and 3 (here corrected to 1 and 2). In this June 2008 version, Table 1 is now correctly labeled Feather River Above Shanghai Bend Storm Centering A *with a Specific Centering on the Yuba River*, and Table 2 is labeled Feather River Above Shanghai Bend Storm Centering B *with a Specific Centering Above Oroville*. The italicized portions of each label were previously reversed.

The New Bullards Bar release schedule has also been added, as Table 6.

WATER MANAGEMENT SECTION
CERTIFICATION FOR INDEPENDENT TECHNICAL REVIEW

Yuba River Basin Project General Reevaluation Report
Appendix A, Synthetic Hydrology and Reservoir Operations
Technical Documentation, Sacramento District
April 2004, Revised June 2008

GENERAL FINDINGS

Compliance with clearly established policy, principles, and procedures, utilizing clearly justified and valid assumptions, has been verified for the subject project. This includes assumptions; methods, procedures and materials used in the analyses; the appropriateness of data used and level of data obtained; and the reasonableness of the results, including whether the product meets the customers' needs consistent with law and existing Corps criteria and policy.

I certify that an independent technical review of the project indicated above has been completed and all technical issues have been identified and resolved. I recommend certification that the quality control process has been completed.

In accordance with CESPD R 11 10-1-8, South Pacific Division Quality Management Plan, May 2000, this letter certifies that the without-project hydrology is appropriate as the basis for use in the hydraulic analysis for the Yuba River Basin Project General Reevaluation.

Laurine L. White
Laurine L. White
Hydrologist, SPK

17 July 2008
Date

James Chieh
James Chieh
Independent Technical Reviewer

17 July 2008
Date

[signature]
Edwin Townsley
Chief, Water Management Section, SPK

7/17/08
Date

SYNTHETIC HYDROLOGY & RESERVOIR OPERATIONS TECHNICAL DOCUMENTATION

TABLE OF CONTENTS

CHAPTER 1 - INTRODUCTION

CHAPTER 2 – FEATHER RIVER HYDROLOGY & RESERVOIR OPERATION MODELING

LIST OF TABLES

LIST OF PLATES

CHAPTER 1

INTRODUCTION

AUTHORITY

The Yuba River Basin, California Final Feasibility Report and Appendices dated April 1998 and approved by Chief of Engineers on November 25, 1998, was authorized by the Water Resources Development Act of 1999. Since the final Yuba River Basin Project was authorized, geotechnical investigations and new hydrology have identified previously unknown levee foundation problems in portions of the specifically authorized project. The preliminary design to effectively maintain the level of protection described in the Feasibility Report will cause the cost of the project to exceed the Section 902 cost limit of the Water Resources Development Act of 1986 for the specifically authorized project. Since flooding is still a significant problem for the affected communities along the Yuba and Feather Rivers, the Reclamation Board has requested that the Corps initiate a re-evaluation of the project. The reevaluation will not be limited to the elements of the authorized project, and new alternatives will be examined.

STUDY AREA

The study area is located in Yuba County about 50 miles north of Sacramento in northern California. The area encompasses the lower Yuba River basin and part of the Feather River basin and includes parts of the eastern Sacramento Valley and Sierra foothills. Elevations in the Yuba River basin range from 30 feet above sea level near the Feather River to over 9,100 feet in the Sierra Nevada. Located in the upper basin are the three forks of the Yuba River. New Bullards Bar Reservoir is located on the north fork, and the other two forks contain a number of much smaller reservoirs. Urban areas include Marysville, Linda, and Olivehurst. The areas of interest for the LRR are the levees surrounding the City of Marysville, 6.1 miles of levee on the left bank of the Yuba River upstream with the confluence with the Feather River, and approximately 10 miles of the left bank of the Feather River downstream of the confluence with the Yuba River.

PURPOSE OF DOCUMENTATION

This appendix documents the hydrology and reservoir operation modeling efforts conducted in support of the Yuba River Basin GRR. This work included both the use of existing technical information obtained from other studies, such as the Comprehensive Study, and new hydrologic analysis. The hydrology developed from the models was used in HEC-RAS by the Hydraulic Design section to 1) define water surface profiles---profiles that will be used to evaluate possible improvements to existing levees and to design new setback levees, and 2) to provide frequency-discharge-stage information required for evaluation of project performance at index locations.

REFERENCES

1. U.S. Army Corps of Engineers. 1999. Rain Flood Flow Frequency Analysis, Feather and Yuba Rivers, California. Sacramento District. Sacramento, California.
2. U.S. Army Corps of Engineers. 2003. Sacramento and San Joaquin River Basins Comprehensive Study, Reservoir Simulation Model User's Guide. Sacramento District. Sacramento, California.
3. U.S. Army Corps of Engineers. 2002. Sacramento and San Joaquin River Basins Comprehensive Study; Technical Studies Documentation, Appendix B, Synthetic Hydrology Technical Documentation. Sacramento District. Sacramento, California.
4. U.S. Army Corps of Engineers. 2002. Sacramento and San Joaquin River Basins Comprehensive Study; Technical Studies Documentation, Appendix C, Reservoir Operations Modeling. Sacramento District. Sacramento, California.

CHAPTER 2

FEATHER RIVER HYDROLOGY & RESERVOIR OPERATION MODELING

GENERAL

The hydrologic analysis for this region focused on the development of a storm that is centered on the Feather River. The Comprehensive Study developed tributary storm centerings on the Feather River at Oroville Dam and the Yuba River at Marysville. However, in order to determine the maximum inundation areas along the lower reaches of the Feather and Yuba rivers, another storm centered at both Shanghai Bend (near the confluence of the Yuba River with the Feather River) and at Verona (near the confluence of the Feather River with the Sacramento River) was needed. Comprehensive Study methods were adopted to derivate this new storm centering.

Reservoir modeling for the Feather and Yuba rivers was done with ResSim, the new software package developed by HEC in support of the Corps Water Management System (CWMS). Resulting regulated hydrographs from the ResSim model were used as input into the hydraulic model (HEC-RAS) to determine river stages and floodplain delineation. The hydrograph "handoff" locations included the Feather River at Oroville, Yuba River at Englebright, Bear River near Wheatland, and locations on other smaller tributaries (Honcut Creek, Deer Creek on the Yuba River, Dry Creek on the Yuba River, and Dry Creek on the Bear River). The analysis discussed in this chapter was conducted in support of three major studies in the area including the Yuba River Basin Project, the Sutter County Feasibility Study, and the Lower Feather Floodplain Mapping Study.

HYDROLOGIC ANALYSIS

Hypothetical Storm Pattern Generation

The intent of this hydrologic analysis is to prepare a hypothetical storm pattern and flood hydrographs that can be fed into reservoir system and hydraulic models for each frequency event (50-, 10-, 4-, 2-, 1-, 0.8-, 0.67-, 0.57-, 0.5-, and 0.2-percent chance exceedences). In order to define floodplains for the entire reach of the Feather River, synthetic storms centered over this area were developed. The Comprehensive Study includes a number of synthetic storms that produce large floods along the Feather and Yuba rivers, including storms centered at Oroville Dam on the Feather River, Marysville on the Yuba River, and at the Latitude of Sacramento (Reference 3). However, in order to determine the maximum inundation areas along the lower reaches of the Feather and Yuba rivers, another storm centered both at Shanghai Bend (near the confluence of the Yuba River with the Feather River) and at Verona (near the confluence of the Feather River with the Sacramento River) was needed.

Large floods at Shanghai Bend result from the combination of high flows from both the Yuba River and Upper Feather River. Historically, large events occurring at Shanghai Bend have resulted from rare events occurring on the Upper Feather River (above Oroville) and also on the Yuba River, with one of these rivers having a slightly rarer event than the other. For example, in 1997 a slightly less frequent event occurred at Oroville than on the Yuba River at Marysville and in 1965, Marysville experienced a less frequent event than at Oroville. However, in both of these years, large floods occurred at Shanghai Bend. Because of the possibility that either scenario could happen, two different hypothetical storm patterns were produced. These storm patterns are shown in Tables 1 and 2. Table 1 shows the storm patterns (actually, flood patterns expressed as percent chance exceedence floods) for the Yuba River centering. The synthetic exceedence frequencies are assigned to each tributary in column 1 in such a way that the regulated and routed hydrographs for the Feather River, Yuba River, and Deer Creek have the volumes for a flood series centered at Shanghai Bend downstream of the Feather-Yuba confluence. The specific storm centerings (Storm Centering A) are on the two Yuba River index points; the concurrent storms are on the Feather River at Oroville.

TABLE 1
Feather River Above Shanghai Bend Storm Centering A
With a Specific Centering on the Yuba River

Index Point	Percent Chance Exceedence						
	50	**10**	**4**	**2**	**1**	**0.5**	**0.2**
Sacramento R at Shasta	101.01	20.20	8.08	5.77	2.89	1.44	0.58
Clear Cr at Whiskeytown	344.83	68.97	27.59	19.70	9.85	4.93	1.97
Cow Cr nr Millville	196.08	39.22	15.69	11.20	5.60	2.80	1.12
Cottonwood Cr nr Cottonwood	344.83	68.97	27.59	19.70	9.85	4.93	1.97
Battle Cr blw Coleman FH	196.08	39.22	15.69	11.20	5.60	2.80	1.12
Mill Cr nr Los Molinos	76.34	15.27	6.11	4.36	2.18	1.09	0.44
Elder Cr nr Paskenta	140.85	28.17	11.27	8.05	4.02	2.01	0.80
Thomes Cr at Paskenta	140.85	28.17	11.27	8.05	4.02	2.01	0.80
Deer Cr nr Vina	76.34	15.27	6.11	4.36	2.18	1.09	0.44
Big Chico Cr nr Chico	76.34	15.27	6.11	4.36	2.18	1.09	0.44
Stony Cr at Black Butte	140.85	28.17	11.27	8.05	4.02	2.01	0.80
Butte Cr nr Chico	76.34	15.27	6.11	4.36	2.18	1.09	0.44
Feather R. at Oroville	54.95	10.87	4.35	2.17	1.06	0.53	0.21
Yuba R. at New Bullards Bar	50.00	10.00	4.00	2.00	1.00	0.50	0.20
Yuba R nr Marysville	50.00	10.00	4.00	2.00	1.00	0.50	0.20
Deer Cr nr Smartsville	125.00	25.00	10.00	5.00	2.50	1.25	0.50
Bear R nr Wheatland	125.00	25.00	10.00	5.00	2.50	1.25	0.50
Cache Cr at Clear Lake	153.85	30.77	12.31	6.15	3.08	1.54	0.62
Cache Cr at Indian Valley	153.85	30.77	12.31	6.15	3.08	1.54	0.62
American R at Folsom	76.34	15.27	6.11	3.05	1.53	0.76	0.31
Putah Cr at Berryessa	153.85	30.77	12.31	6.15	3.08	1.54	0.62

Note – The seven frequency storms centered at Shanghai Bend and Verona are the bold values located in the column headers. The concurrent frequency values for each index location are given below each column header. For example, a 2.89% chance exceedence event occurs on the Sacramento River above Shasta Dam during the 1% chance exceedence event centered at Shanghai Bend and Verona.

TABLE 2
Feather River Above Shanghai Bend Storm Centering B
With a Specific Centering Above Oroville

Index Point	Percent Chance Exceedence									
	50	**10**	**4**	**2**	**1**	**0.80**	**0.67**	**0.57**	**0.50**	**0.20**
Sacramento R at Shasta	101.01	20.20	8.08	5.77	2.89	2.31	1.92	1.65	1.44	0.58
Clear Cr at Whiskeytown	344.83	68.97	27.59	19.70	9.85	7.88	6.57	5.63	4.93	1.97
Cow Cr nr Millville	196.08	39.22	15.69	11.20	5.60	4.48	3.73	3.20	2.80	1.12
Cottonwood Cr nr Cottonwood	344.83	68.97	27.59	19.70	9.85	7.88	6.57	5.63	4.93	1.97
Battle Cr blw Coleman FH	196.08	39.22	15.69	11.20	5.60	4.48	3.73	3.20	2.80	1.12
Mill Cr nr Los Molinos	76.34	15.27	6.11	4.36	2.18	1.74	1.45	1.25	1.09	0.44
Elder Cr nr Paskenta	140.85	28.17	11.27	8.05	4.02	3.22	2.68	2.30	2.01	0.8
Thomes Cr at Paskenta	140.85	28.17	11.27	8.05	4.02	3.22	2.68	2.30	2.01	0.8
Deer Cr nr Vina	76.34	15.27	6.11	4.36	2.18	1.74	1.45	1.25	1.09	0.44
Big Chico Cr nr Chico	76.34	15.27	6.11	4.36	2.18	1.74	1.45	1.25	1.09	0.44
Stony Cr at Black Butte	140.85	28.17	11.27	8.05	4.02	3.22	2.68	2.30	2.01	0.8
Butte Cr nr Chico	76.34	15.27	6.11	4.36	2.18	1.74	1.45	1.25	1.09	0.44
Feather R. at Oroville	50.00	10.00	4.00	2.00	1.00	0.80	0.67	0.57	0.5	0.2
Yuba R. at New Bullards Bar	58.82	10.42	4.76	2.04	1.04	0.84	0.71	0.61	0.54	0.22
Yuba R nr Marysville	58.82	10.42	4.76	2.04	1.04	0.84	0.71	0.61	0.54	0.22
Deer Cr nr Smartsville	125.00	25.00	10.00	5.00	2.50	2.00	1.67	1.43	1.25	0.5
Bear R nr Wheatland	125.00	25.00	10.00	5.00	2.50	2.00	1.67	1.43	1.25	0.5
Cache Cr at Clear Lake	153.85	30.77	12.31	6.15	3.08	2.46	2.05	1.76	1.54	0.62
Cache Cr at Indian Valley	153.85	30.77	12.31	6.15	3.08	2.46	2.05	1.76	1.54	0.62
American R at Folsom	76.34	15.27	6.11	3.05	1.53	1.22	1.02	0.87	0.76	0.31
Putah Cr at Berryessa	153.85	30.77	12.31	6.15	3.08	2.46	2.05	1.76	1.54	0.62

Note – The seven frequency storms centered at Shanghai Bend and Verona are the bold values located in the column headers. The concurrent frequency values for each index location are given below each column header. For example, a 2.89% chance exceedence event occurs on the Sacramento River above Shasta Dam during the 1% chance exceedence event centered at Shanghai Bend and Verona.

There are only subtle differences between these two storm patterns. These differences lie within the index locations on the Feather and Yuba rivers. For storm centering A, exceedence frequency values generated at Shanghai Bend and the Latitude of Verona are the same as the frequency assigned to the Yuba River. However, for storm centering B, the Yuba River experiences a more frequent event, and the Feather River at Oroville is assigned the same exceedence frequency value that is produced at Shanghai Bend and the Latitude of Verona. In other words, storm centering A has more emphasis on the Yuba River, and storm centering B has more emphasis on the Feather River.

In developing these storm centerings, the guidelines for preparation of mainstem centerings developed for the Comprehensive Study were followed (Reference 3). Shanghai Bend and the Latitude of Verona are the bull's eyes of the storm. That is, no other location within the Sacramento River Basin experiences a larger flood than at Shanghai Bend and the Latitude of Verona for the 10 hypothetical storms (50-, 10-, 4-, 2-, 1-, 0.8-, 0.67-, 0.57-, 0.5-, and 0.2- percent chance exceedences). First, the distribution of storm intensity for the Upper Feather and Yuba River basins was developed. Initial exceedence frequency values were assigned to the Yuba River and Feather River index locations. Hydrographs were then constructed at these tributary locations and routed through the system to Shanghai Bend. Duration maxima (peak, 1-, 3-, 7-, 15-, and 30-day) were computed for the hydrographs at Shanghai Bend and compared with the average flows from the frequency curves. The initial pattern was then increased or decreased and the comparison process was repeated until results agreed reasonably with the unregulated rain flood frequency curves.

Once this portion of the pattern was set, the same process was followed for the Latitude of Verona index location. The storm pattern for the rest of the tributary index locations were based upon the average of the Feather and Yuba River storm centerings generated for the Comprehensive Study (Reference 3). This pattern was iteratively adjusted by a fixed percentage until the duration maxima (1-, 3-, 7-, 15-, and 30-day) computed at the Latitude of Verona agreed reasonably with the unregulated rain flood frequency curve at this index location.

Hydrograph Construct

The hydrographs generated at each tributary index location are hypothetical hourly hydrographs made up of six 5-day waves. The translation from a frequency to a hypothetical 30-day flood series is described in Plate 2. This process includes: 1) obtaining the average flood flow rates from the unregulated frequency curves 2) separating these average flows into wave volumes, and 3) distributing volumes into the 6-wave series. This process is performed only at the tributary locations. Mainstem flood hydrographs are the result from the routed contributions of upstream tributaries. Please refer to Reference 3 for further explanation of this process.

The frequency curves used in this process were obtained from the Comprehensive Study (Reference 3), except for the Shanghai Bend unregulated flow frequency curve. This curve was adopted from the 1999 FEMA report entitled, "Rain Flood Flow Frequency Analysis, Feather and Yuba Rivers" (Reference 1). No adjustments were made to any of the frequency curves except for the peak curve for Shanghai Bend. According to Reference 1, the peak mean for the unregulated flow frequency curve at Shanghai Bend was proportioned based on the relationship of the peak and 1-day means at Oroville, since no peak unregulated data at Shanghai Bend was available. However, the peak mean value on the Shanghai Bend flow frequency curve does not represent this relationship. Therefore, the peak mean value of 4.977 was replaced with the correct value of 4.951. This frequency curve with the modified statistics is presented in Plate 3.

The 1997 flood was chosen as the pattern for the five-day wave patterns. These wave patterns were constructed by adjusting regulated gage records for the 1997 flood event in accordance with changes in upstream storage. Natural series were computed for all tributaries locations except the Sacramento River at Shasta Dam, Feather River at Oroville, and Deer Creek near Smartsville. At these sites, insufficient data at headwater reservoirs precluded the accurate computation of natural flows; regulated flows were used as pattern hydrographs. All patterns remained unchanged except for the Yuba River. The shape that was used to form the pattern hydrograph for the North, Middle,

and South forks of the Yuba River was the 1997 inflow hydrograph to New Bullards Bar Reservoir. The top of this hydrograph is fairly flat, resulting in a peak of only about 7% higher than the maximum 24-hour average flow. Other historical events reveal a percentage that is much higher. For example, the 1986 and 1995 storms resulted in peaks 27% and 30% higher than the maximum 24-hour average flows.
The use of this 1997 shape posed a problem when trying to match the peak flow frequency curve at Marysville. In order to produce results that agreed reasonably with the unregulated rain flood peak frequency curve at Marysville, the pattern had to be manipulated, resulting in a peak increase of 25%. The timing of the peak was not changed and the volumes of the other durations were not affected significantly.

RESERVOIR OPERATIONS MODELING

Methodology

The reservoir modeling for the Feather River was accomplished using the new ResSim modeling package. The Sacramento District contracted with HEC to convert the Comprehensive Study HEC-5 models to ResSim for the Sacramento and San Joaquin watersheds in support of the District's CWMS modeling effort. The spatial extent of this model is shown in Plate 4.

The intent of this conversion was to replicate the results of the Comprehensive Study HEC-5 models using ResSim; therefore, all hydrologic routing parameters and methods, starting storage assumptions, and operational rules found in the Comprehensive Study HEC-5 models were incorporated into the ResSim model.

HEC is still in the process of developing ResSim models for some of the river basins; however, the ResSim model covering the Feather and Yuba River basins has been completed. All of the reservoirs included in both the headwater and lower basin Comprehensive Study HEC-5 models for the Feather and Yuba River basins are included in this ResSim model. See Table 3 for a complete listing of these reservoirs.

TABLE 3
Modeled Reservoirs in the Feather and Yuba River Basins

Reservoir	Tributary	Owner	Storage Capacity (ac-ft)	Drainage Area (sq mi)
Feather River				
Mountain Meadows	Hamilton Creek	PGE	24,800	158
Almanor	Nfk Feather Creek	PGE	1,308,000	503
Butt Valley	Butte Creek	PGE	49,800	86.2
Antelope	Indian Creek	DWR	22,566	71
Bucks Lake	Bucks Creek	PGE	103,000	29.5
Frenchman	Last Chance Creek	DWR	55,477	82
Lake Davis	Big Grizzly Creek	DWR	83,000	44
Little Grass Valley	Sfk Feather River	OWID	93,010	27.3
Sly Creek	Lost Creek	OWID	65,050	23.9
Oroville	Feather River	DWR	3,538,000	3,611
Yuba above Marysville				
New Bullards Bar	Nfk Yuba River	YCWA	960,000	489
Jackson Meadows	Mfk Yuba River	NID	52,500	37.11
Bowman	Canyon Creek	NID	64,000	28.91
Fordyce	Fordyce Creek	PGE	48,900	30
Spaulding	Sfk Jackson Creek	PGE	74,773	118
Scotts Flat	Deer Creek	NID	49,000	20
Merle Collins	Dry Creek	BVID	57,000	72.3

Model Changes

A number of modifications were made to the ResSim model delivered to the Sacramento District by HEC prior to use in the Sutter County Feasibility Study and the Lower Feather Floodplain Mapping Study. For both studies, starting storages for all but two headwater reservoirs were set at gross pool because storage capability below the normal pool elevation of dams operated primarily for purposes other than flood control should not be considered because the availability of such storage is uncertain. The storage for both Bucks Lake and Lake Almanor has never exceeded gross pool; therefore, the maximum storage that has occurred at the lakes for the months of December-March was used as the starting storage. Even though the model simulations began with the majority of the reservoirs at gross pool, effects of peak attenuation for many locations along the Feather and Yuba Rivers was still evident due to surcharge effects (Table 4).

TABLE 4

Effects of Headwater Regulation

Location	Annual Percent Chance Exceedence	Unregulated Peak Flow (cfs)	Regulated Peak Flow (cfs)	% Peak Reduction Due to Regulation
MF + SF of Yuba	50%	8,800	6,300	28.1
	10%	38,800	34,100	12.2
	4%	58,600	52,400	10.5
	2%	76,400	68,500	10.3
	1%	96,200	87,300	9.3
	0.5%	117,800	107,200	9.0
	0.2%	149,200	137,000	8.2
Deer Creek	50%	2,400	2,200	5.9
	10%	4,900	4,600	5.9
	4%	7,300	6,800	5.9
	2%	8,700	8,200	5.9
	1%	10,100	9,500	5.9
	0.5%	11,400	10,700	5.7
	0.2%	13,000	12,400	4.9
Dry Creek	50%	2,400	2,200	5.9
	10%	4,900	4,600	5.9
	4%	7,300	6,800	5.9
	2%	8,700	8,200	5.9
	1%	10,100	9,500	5.9
	0.5%	11,400	10,700	5.7
	0.2%	13,000	11,600	10.9
Oroville Inflow	50%	51,700	47,300	8.5
	10%	153,700	135,900	11.6
	4%	225,100	200,700	10.8
	2%	284,100	253,100	10.9
	1%	349,600	311,500	10.9
	0.5%	419200	373800	10.9
	0.2%	520,300	464,600	10.7

Notes:
% Peak Reduced = ((Maximum Unregulated Inflow)-(Maximum Regulated Inflow))/(Maximum Unregulated Inflow) X 100%
Values are from model simulations of the Feather River Storm Centering A

No changes were made to the Oroville or New Bullards Bar release schedule; those schedules are included in this report as Tables 5 and 6, respectively.

TABLE 5
Oroville Release Schedule

Actual or Forecasted Inflow (Whichever is Greater) (cfs)	Flood Control Space Used (acre-ft)	Required Releases (cfs)
0 – 15,000	0 – 5,000	Power demand
0 – 15,000	Greater than 5,000	Inflow
15,000 – 30,000	0 – 30,000	Lesser of 15,000 or maximum inflow
0 – 30,000	Greater than 30,000	Maximum inflow for flood
30,000 – 120,000	N/A	Lesser of maximum inflow or 60,000
120,000 – 175,000	N/A	Lesser of maximum inflow or 100,000
Greater than 175,000	N/A	Lesser of maximum inflow or 150,000

TABLE 6
New Bullards Bar Release Schedule

Actual Inflow (cfs)	Flood Control Space Used (ac-ft)	Required Releases (cfs)
0 – 50,000	0 – 170,000	Inflow
50,000 – 120,000	0 – 170,000	Inflow
Greater than 120,000	0 – 170,000	Inflow up to 180,000

Note – Emergency spillway release diagram used when the combination of the rate of rise and pool elevation dictate.

Both the Comprehensive Study HEC-5 model and the original ResSim model developed by HEC did not incorporate the forecasted inflow component of this release schedule. For example, releases would be restricted to 60,000 cfs until an actual inflow exceeded 120,000 cfs. At this time releases would begin to ramp up to the next specified flow value in the schedule (100,000 cfs for this example). In reality, releases would begin to ramp up to 100,000 cfs much earlier than this if a forecasted inflow greater than 120,000 cfs was known. All events greater than the 10% flood have peak flows greater than the largest value in the release schedule (175,000 cfs); so, for these events, Oroville releases were modeled to allow releases to ramp up freely to the maximum objective flow of 150,000 cfs at a rate of 5,000 cfs an hour.

Another change to the ResSim model involved travel times. Total travel time from Oroville Dam down to Yuba City was increased from 8 hours to 16 hours, which is consistent with the published travel times used by the Department of Water Resources and is in better agreement with what has been observed.

Lastly, changes were made to the model to incorporate a forecast uncertainty component to the local flow. The original models assumed complete certainty in local flow contributions downstream of a reservoir. This assumption yields high operational efficiency when operating for downstream flow criteria. In reality, however, local flow contributions could be greater or less than what was forecasted. Because of the possibility that local flows could be more than what is forecasted, reservoir releases are typically less than what the calculated releases would be based on the forecasted information. The magnitude of forecast uncertainty can vary from basin to basin and also from storm to storm. The Corps standard is to incorporate a 20% uncertainty in local flow contributions when operating for downstream flow targets. This uncertainty percentage was modeled in ResSim by reducing all downstream flow targets by 20% of the local flow contributing to that specific location. These modifications are listed in Table 7.

TABLE 7
Downstream Flow Target Reductions

Reservoir	Downstream Location	Target Flow (cfs)	Reduced Target Flow (cfs)
Oroville	Yuba City	180,000	174,000
	Below Yuba R. Confluence	300,000	280,000
	Below Bear R. Confluence	320,000	312,000
New Bullards Bar	Marysville	120,000/180,000	106,000/154,000

Model runs were also simulated assuming complete certainty in local flow contributions for all frequency events. Results from both scenarios were compared for each flood event. The scenario producing the larger flows was selected for defining baseline conditions. Generally, the complete certainty scenario was selected for events in which the reservoirs were able to satisfy downstream flow criteria, and the 20% uncertainty scenario was selected for those events in which the downstream flow criteria were exceeded.

Operational Risk

Computation of expected annual damages and annual exceedence probabilities for comparison of plan performance requires definition of the with- and without-project conditions. For every proposed alternative, the flood damage reduction potential depends on the performance as designed. No matter how well a project is designed, the performance is never a certainty. The Corps Engineering Manual entitled Risk-Based Analysis for Flood Damage Reduction Studies (EM 1110-2-1619) provides guidance and procedures for how to account for risk and uncertainty in flood damage reduction studies. Chapter 7 of the EM specifically addresses procedures for describing uncertainty of reservoir performance. Reservoir operational performance is dependent on a multitude of factors that are variable from storm to storm. Such factors include starting reservoir storages, operational response time, and forecasting accuracy. In Chapter 7, recommended procedures to account for such uncertainty are outlined in 4 main steps: 1) identify critical, uncertain factors that would affect peak outflow; 2) identify combinations of the factors to define a best-case, most-likely case, and a worst-case operation scenario; 3) select a probability distribution to represent the likelihood of the resulting scenarios based on expert subjective judgment; 4) compute outflows for a range of inflow peaks of known exceedence probabilities for all three cases. The resulting probabilistic description of uncertainty should then be included in sampling procedures described in Chapter 2 of EM 1110-2-1619. A significant amount of time and money would be needed in order to perform such an analysis for a system as complex as the Yuba-Feather. Therefore, a more simplistic approach was taken for this study: the starting storage changes and target flow reductions described above were included in the ResSim model to account for operational uncertainty.

Results

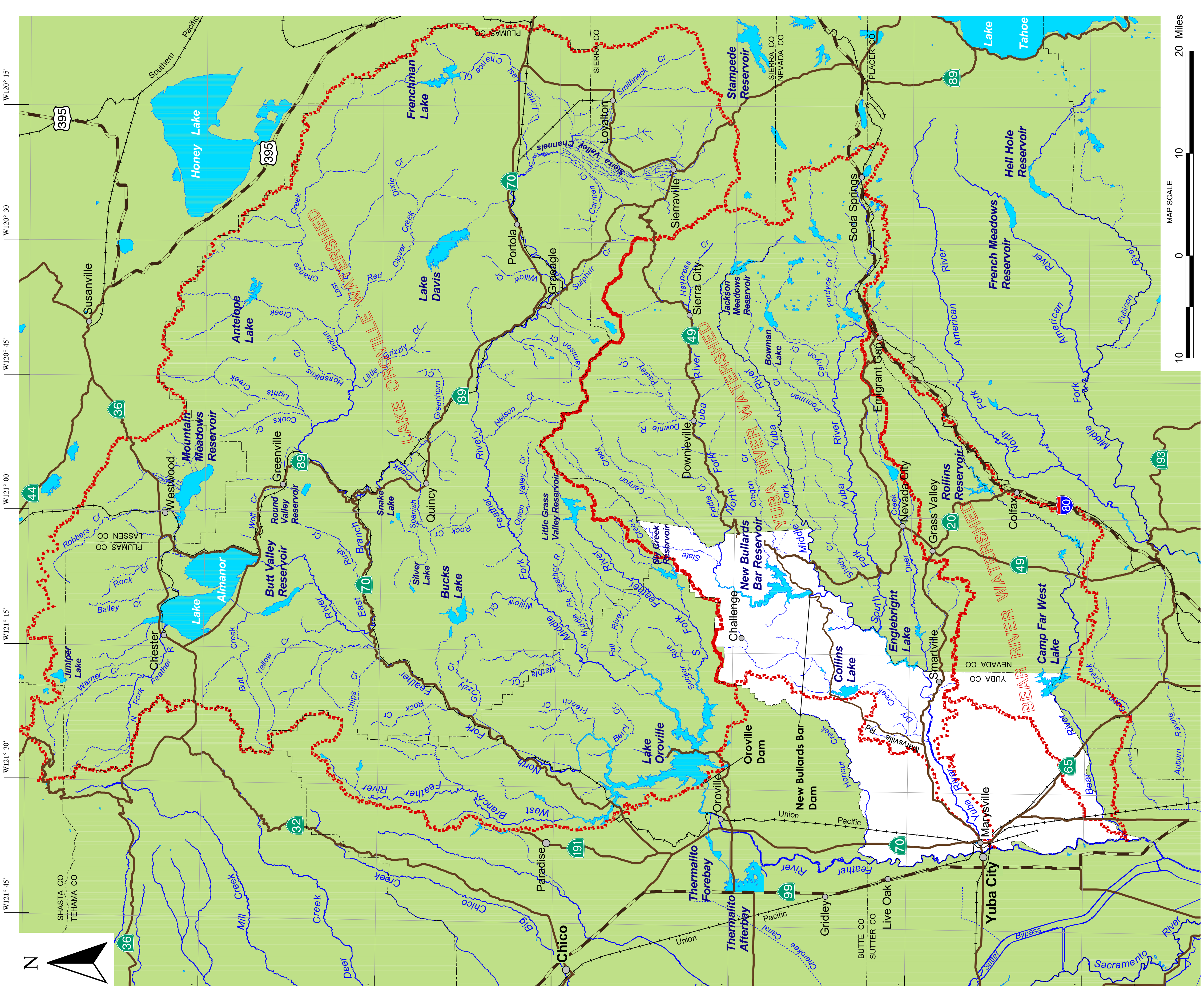
Discussion of results will focus on the area in which the synthetic storms are centered, the Feather-Yuba system, even though the spatial extent of the storms covered the entire Sacramento River Basin.

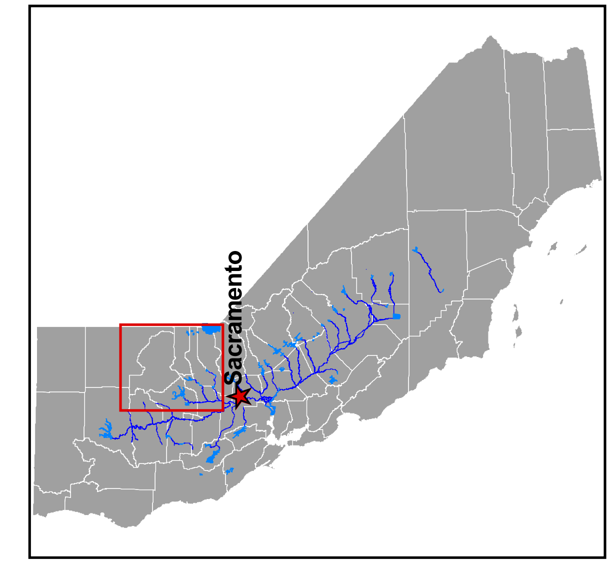
Seven reservoirs were modeled within the Yuba River Basin. New Bullards Bar, located on the North Fork of the Yuba River, is the only reservoir that has dedicated flood space. New Bullards Bar, which contains 170,000 acre-feet of flood space, operates to flow targets at Marysville. The flow criteria at Marysville is 180,000 cfs except when the Feather River is experiencing high flows. When the flows in the Feather River upstream of the Yuba River confluence are high, the flow target at Marysville is reduced to 120,000 cfs. This adjustment is made to assure that 300,000 cfs is not exceeded at the confluence of the Yuba River with the Feather River. New Bullards Bar is able to maintain its objective flow of 50,000 cfs for all events through the 2-percent chance exceedence event. For events larger than the 2-percent chance exceedence event, New Bullards Bar outflow exceeds 50,000 cfs. However, the 300,000 cfs flow target at the confluence is still met for the 0.8-percent chance exceedence event. Operation plots of New Bullards Bar are presented in Plates 15-24.

The other six reservoirs modeled in the Yuba Basin, known as headwater reservoirs, are much smaller and do not have any dedicated flood space. However, they still contribute to attenuating peak flows. Average peak flows along the Middle and South forks of the Yuba River were attenuated by 8.8% for the 1-, 0.5-, and 0.2-percent chance exceedence events.

A total of 9 headwater reservoirs were modeled in the watershed above Oroville. Only 20% of the natural flow hydrograph at Oroville was routed through these headwater reservoirs. However, these reservoirs still had a significant impact on attenuating flows into Oroville. Average peak inflows to Oroville were reduced by 10.8% for the 1-, 0.5-, and 0.2-percent chance exceedence events.

Oroville Reservoir has a maximum flood space reservation of 750,000 acre-feet, and is required to maintain flow targets at multiple downstream locations. It is also required to maintain flows at or below 180,000 cfs above the Yuba River confluence, 300,000 cfs below the Yuba River confluence, and 320,000 cfs below the Bear River confluence. These criteria were met for all events up to and including the 1-percent chance exceedence event. During the less frequent events (0.8-percent chance exceedence event and rarer) releases are triggered by the Emergency Spillway Release Diagram (ESRD). However, the ESRD does not require releases to go above the objective flow of 150,000 cfs until the 0.5-percent chance exceedence event. For the events between the 1- and 0.5-percent exceedence events the objective flow is not exceeded, but downstream flow targets are. The flow target of 320,000 cfs downstream of the Bear River confluence is exceeded during the 0.8-percent chance exceedence event because Oroville ESRD operational criteria cause releases to be increased during a time in the event in which releases should continue to be reduced to meet the flow target. Flow targets are exceeded below the Yuba River confluence and also below the Bear River confluence for all events rarer than the 0.8-percent chance exceedence event. Operation plots of Oroville are presented in Plates 5-14.

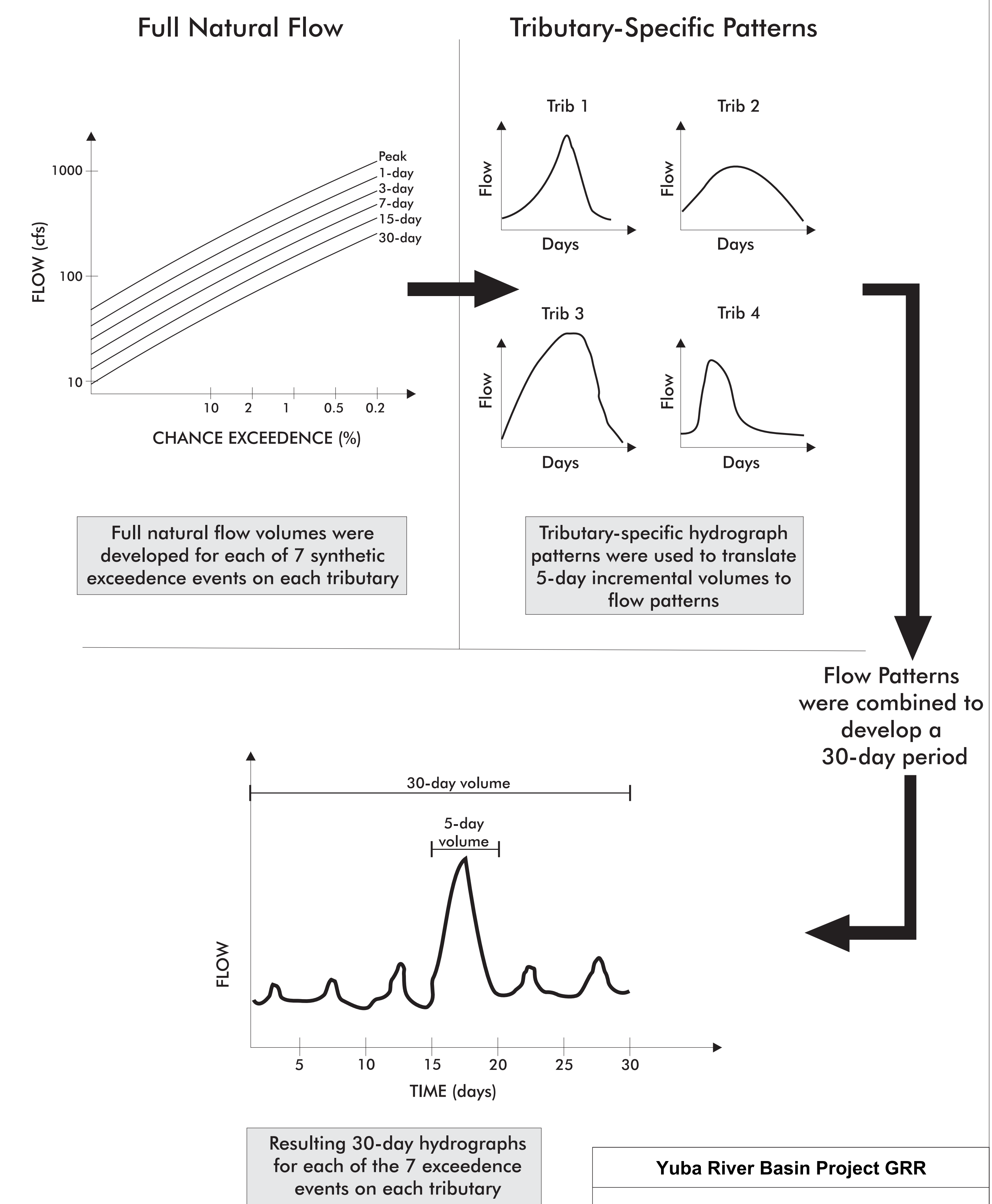




Yuba River Basin Project GRR

Plate 1
General Map

US Army Corps of Engineers **August 2004**

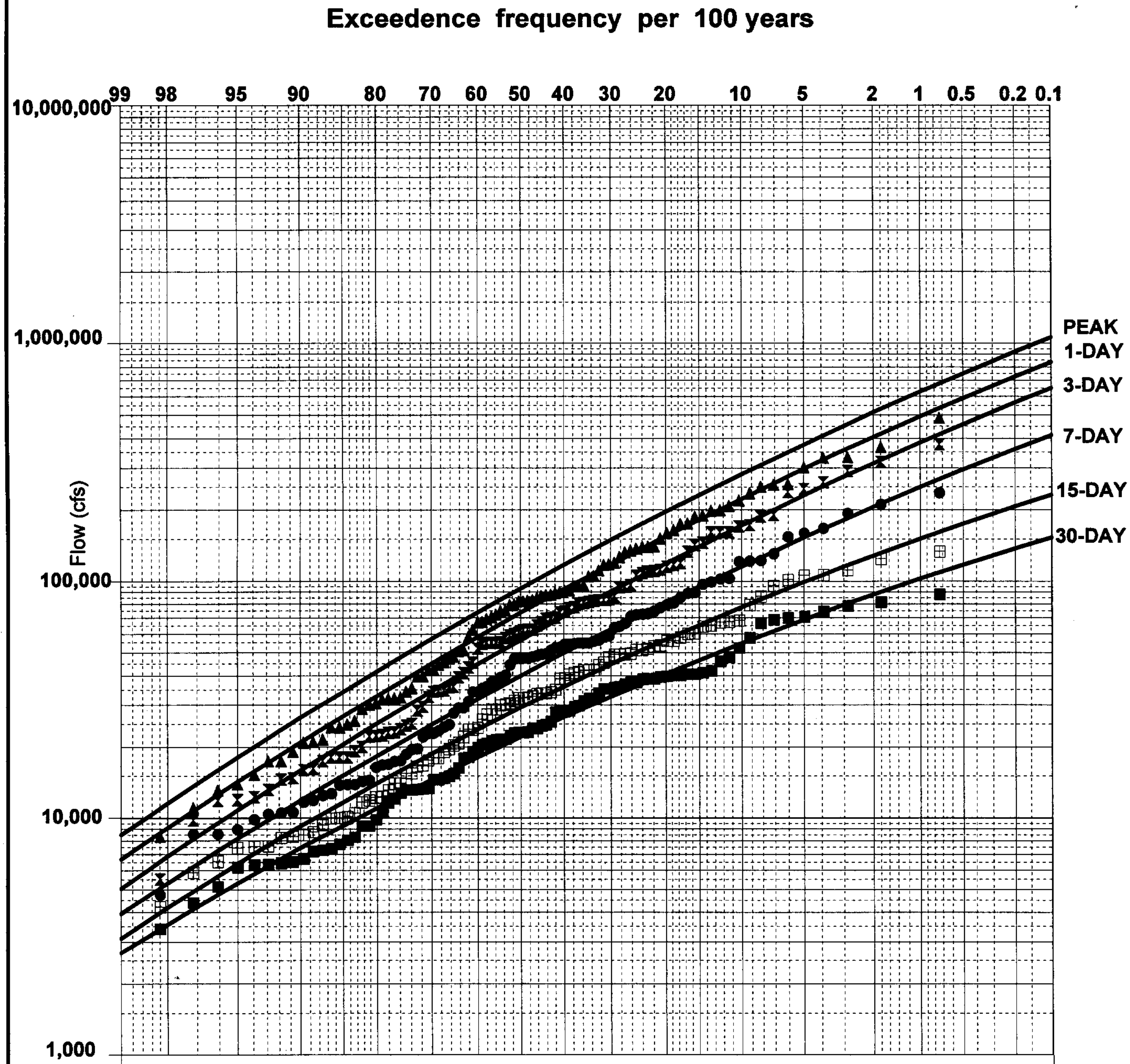


Yuba River Basin Project GRR

Plate 2
Hydrograph Construction

US Army Corps of Engineers **August 2004**

prepared by B.J.W.



ADOPTED CURVE STATISTICS

Curve	Mean	Std. Dev.	Skew
PEAK	4.951	0.402	-0.3
1-DAY	4.847	0.402	-0.3
3-DAY	4.733	0.404	-0.3
7-DAY	4.582	0.387	-0.3
15-DAY	4.443	0.363	-0.4
30-DAY	4.321	0.340	-0.4

NOTES:

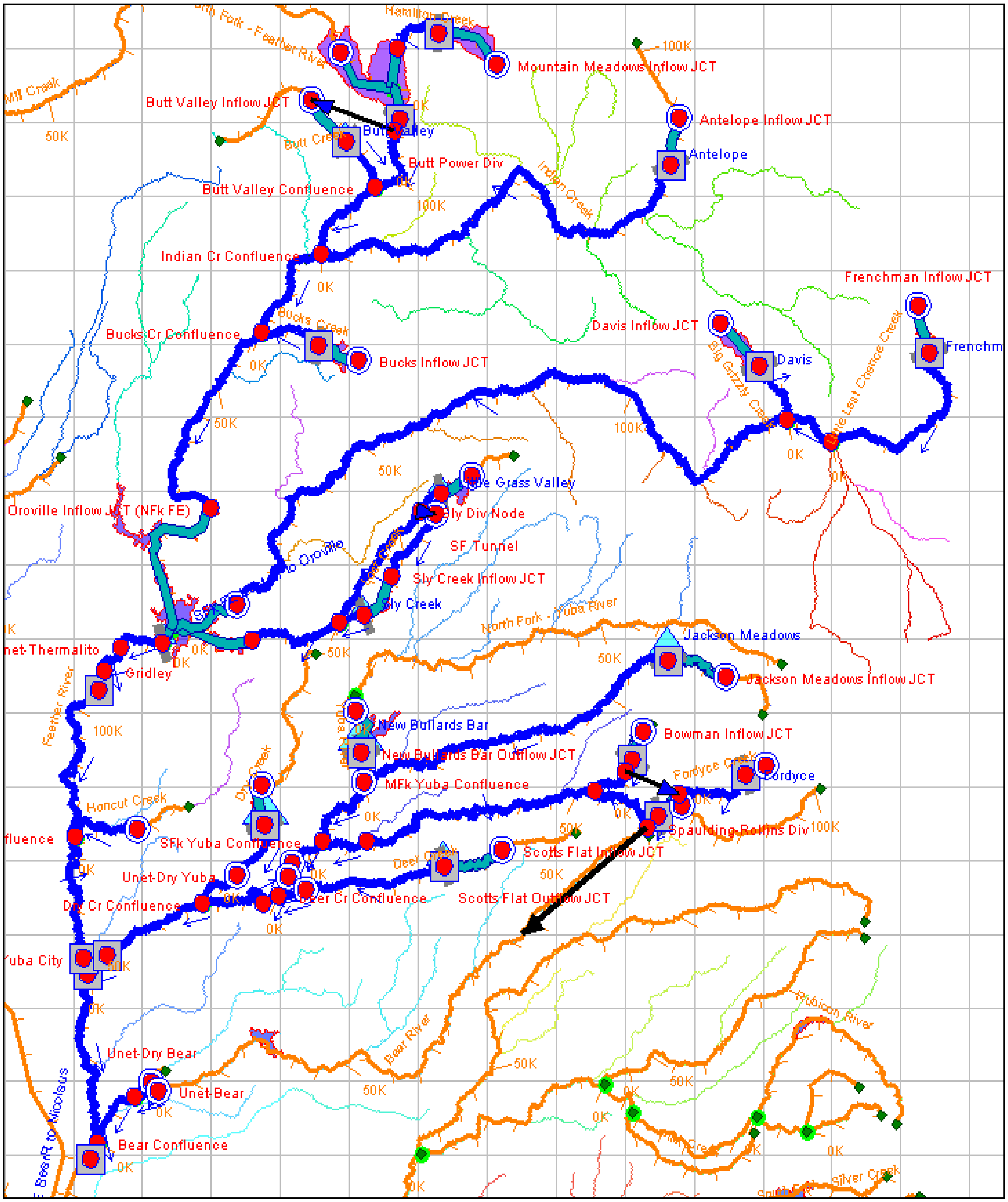
1. Median Plotting Positions
2. Drainage Area: 5313 sq. mi.
3. 94 years of record (1904 to 1997)

MODIFIED BY BJW MAY 02

Yuba River Basin Project GRR

Plate 3
Rain Flood Flow Frequency Curves
Feather River at Shanghai Bend
Unregulated Conditions

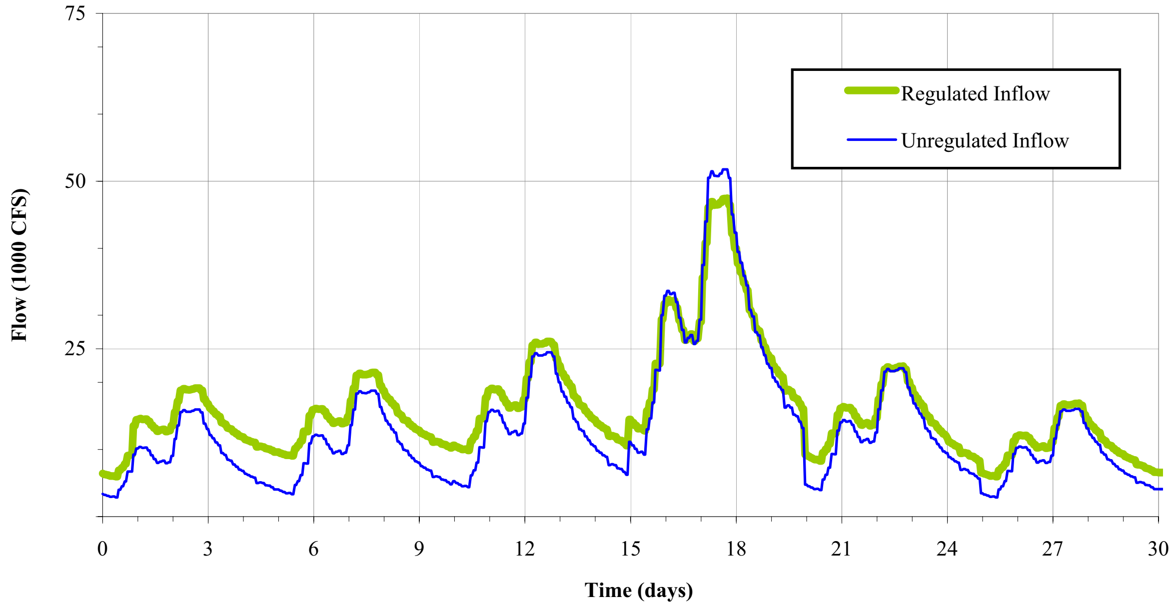
US Army Corps of Engineers August 2004



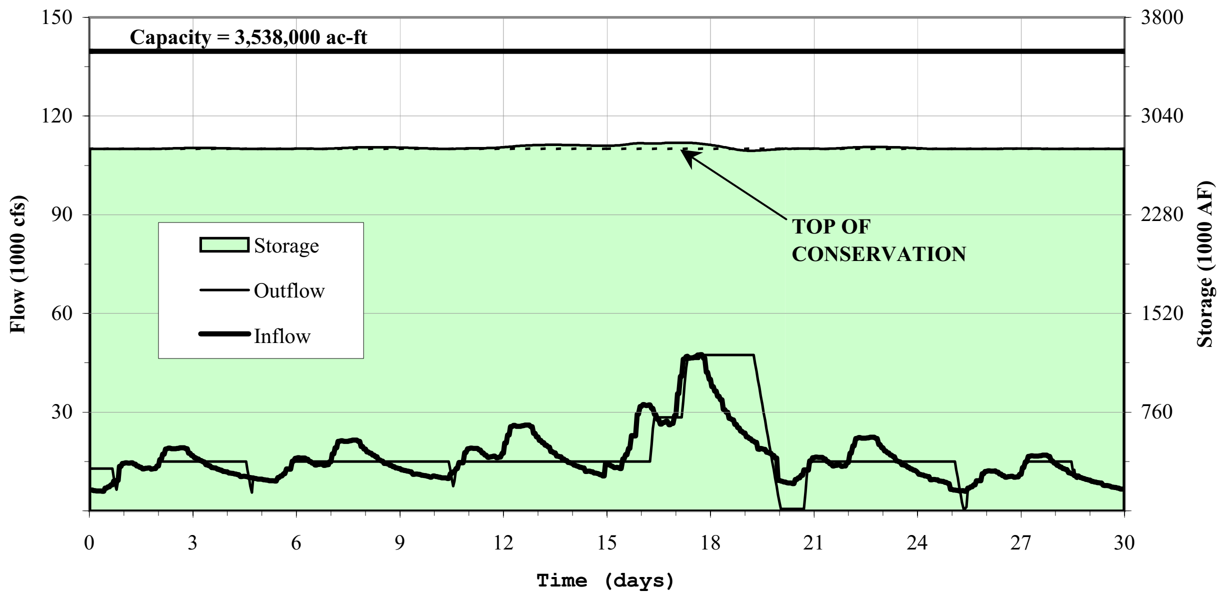
Yuba River Basin Project GRR	
Plate 4 **ResSim Model Schematic**	
US Army Corps of Engineers	August 2004

FEATHER RIVER STORM CENTERING A
Oroville Inflow (50% Chance Exceedence Event)

Regulated and Unregulated Inflow



Oroville Operations (50% Chance Exceedence Event)



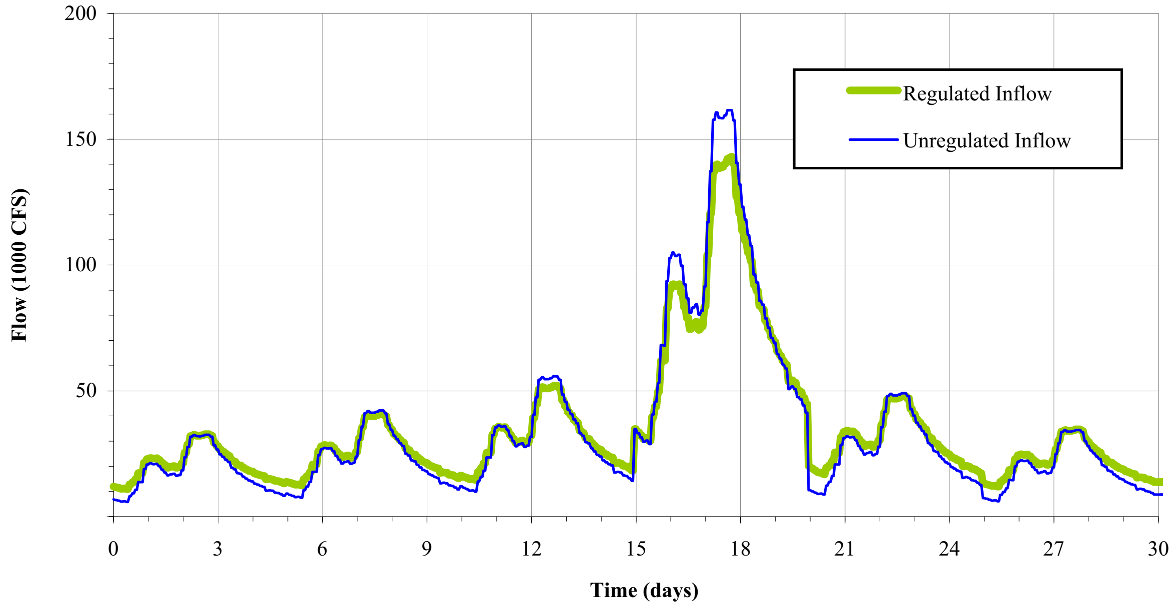
Notes:

1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

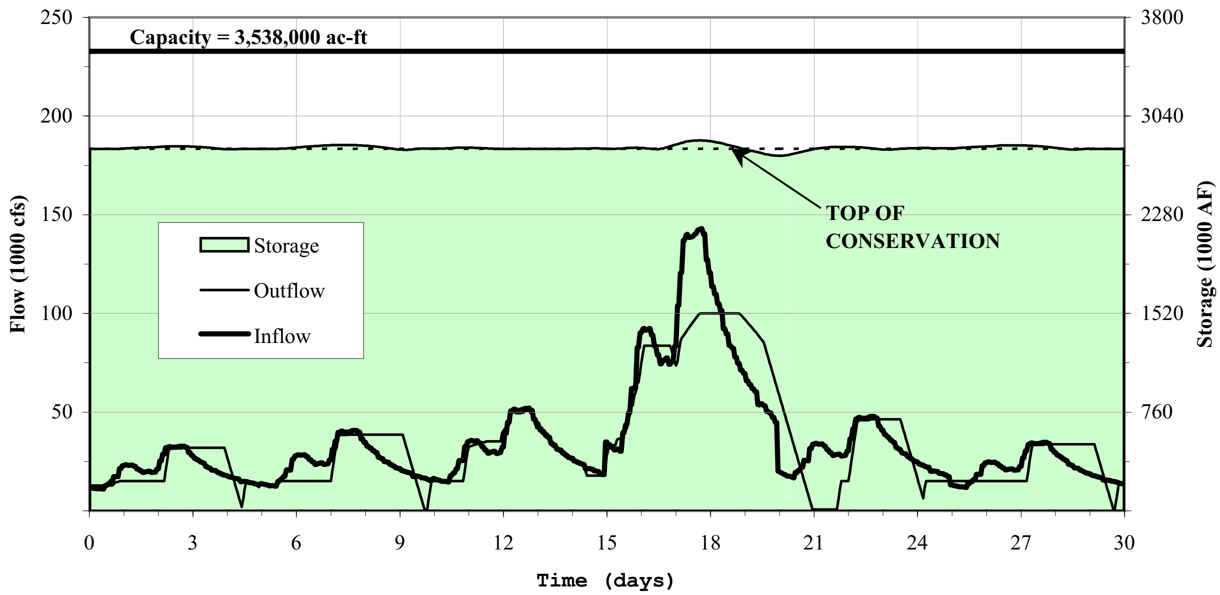
Yuba River Basin Project GRR	
Plate 5 **Reservoir Simulation Hydrographs** **Oroville** **(50% Chance Exceedence Event)**	
US Army Corps of Engineers	**August 2004**

prepared by B.J.W.

FEATHER RIVER
Oroville Inflow (10% Chance Exceedence Event)
Regulated and Unregulated Inflow



Oroville Operations (10% Chance Exceedence Event)



Notes:
1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

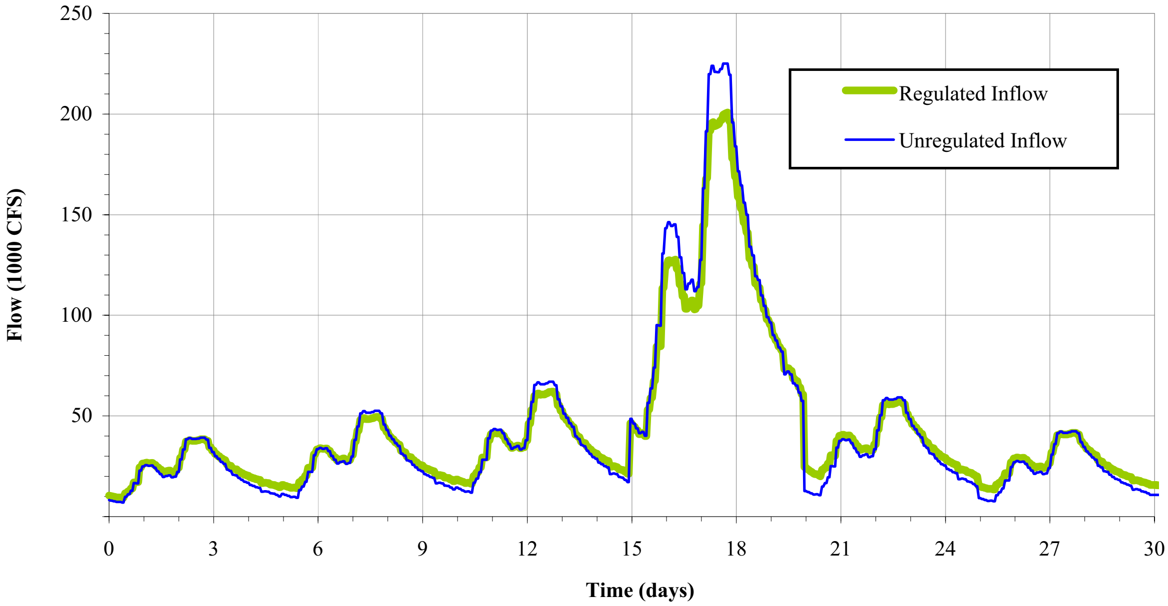
Yuba River Basin Project GRR	
Plate 6 Reservoir Simulation Hydrographs Oroville (10% Chance Exceedence Event)	
US Army Corps of Engineers	August 2004

prepared by B.J.W.

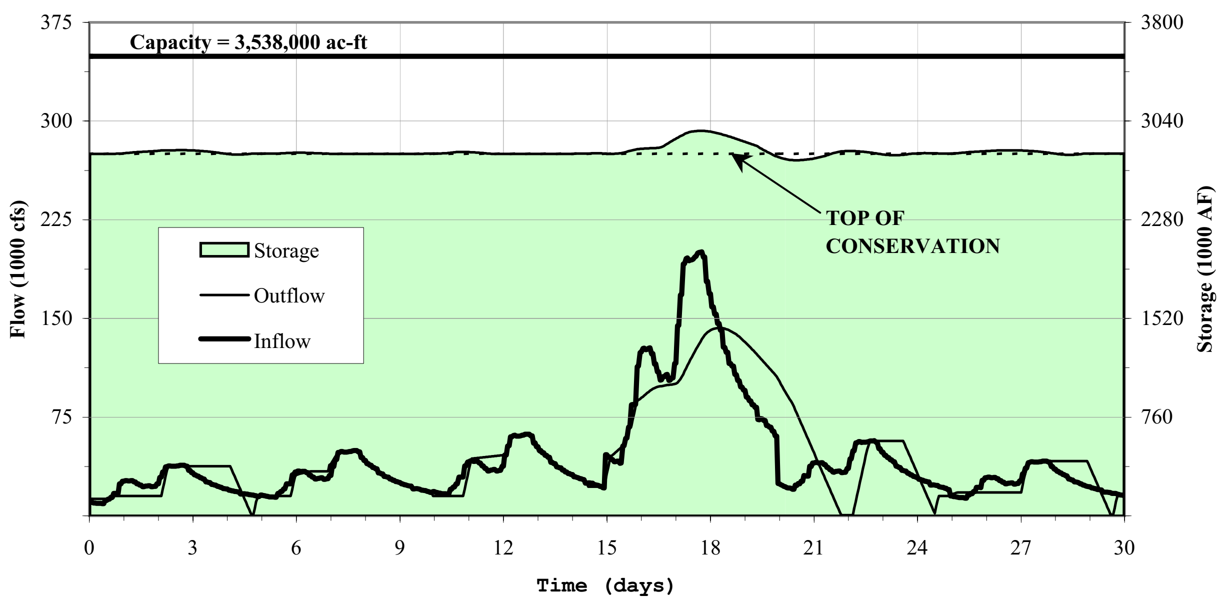
FEATHER RIVER

Oroville Inflow (4% Chance Exceedence Event)

Regulated and Unregulated Inflow



Oroville Operations (4% Chance Exceedence Event)

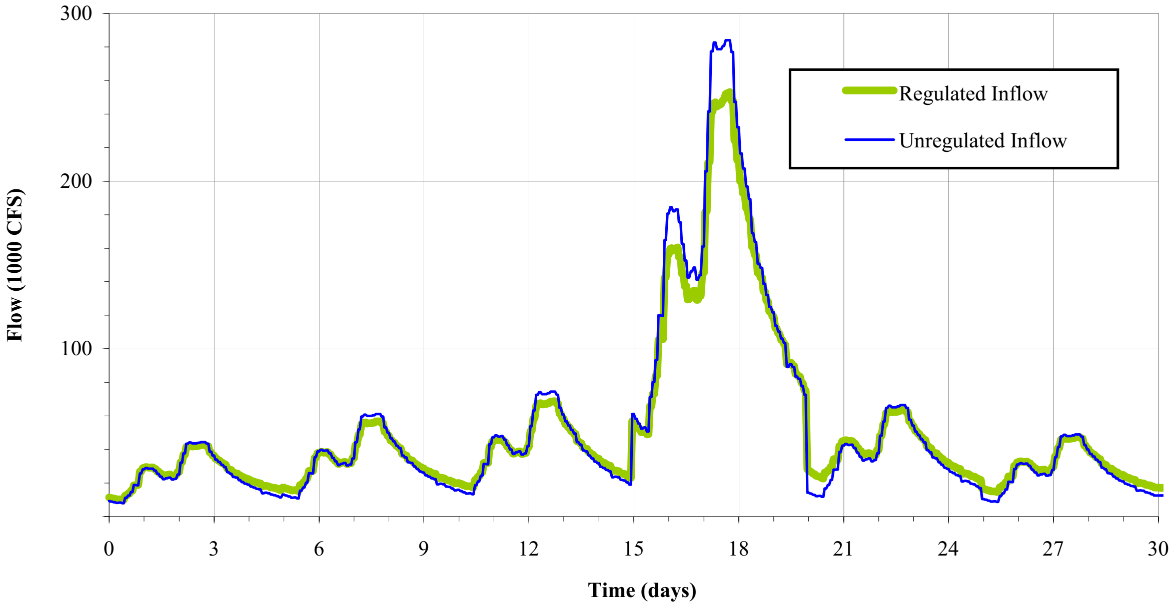


Notes:

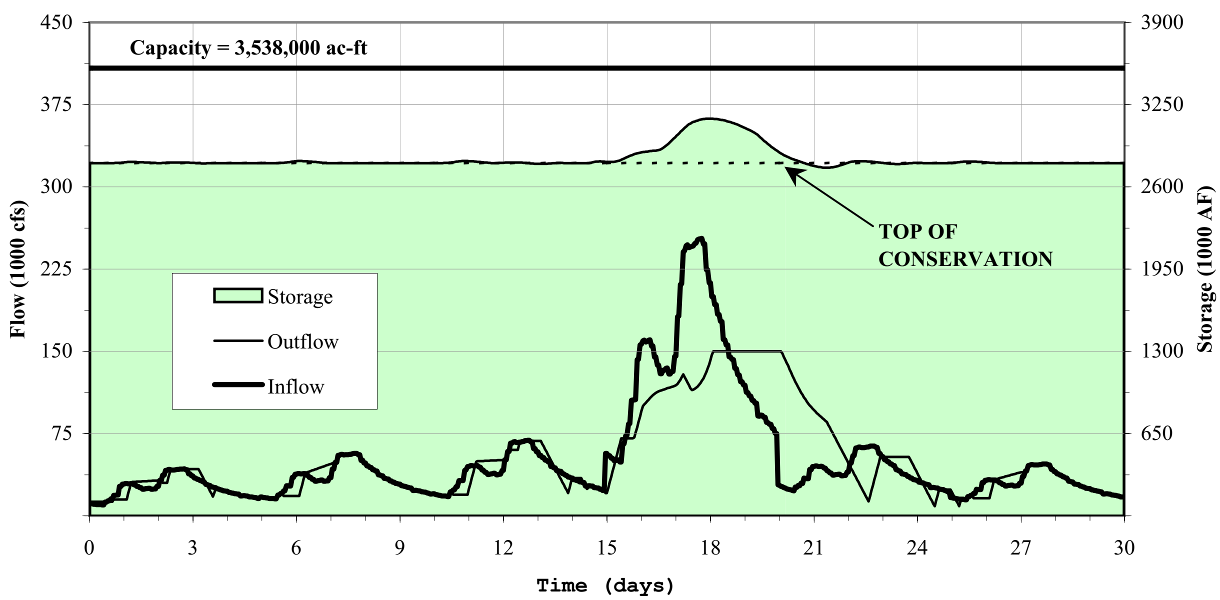
1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

Yuba River Basin Project GRR	
Plate 7 Reservoir Simulation Hydrographs Oroville (4% Chance Exceedence Event)	
US Army Corps of Engineers	August 2004

FEATHER RIVER
Oroville Inflow (2% Chance Exceedence Event)
Regulated and Unregulated Inflow

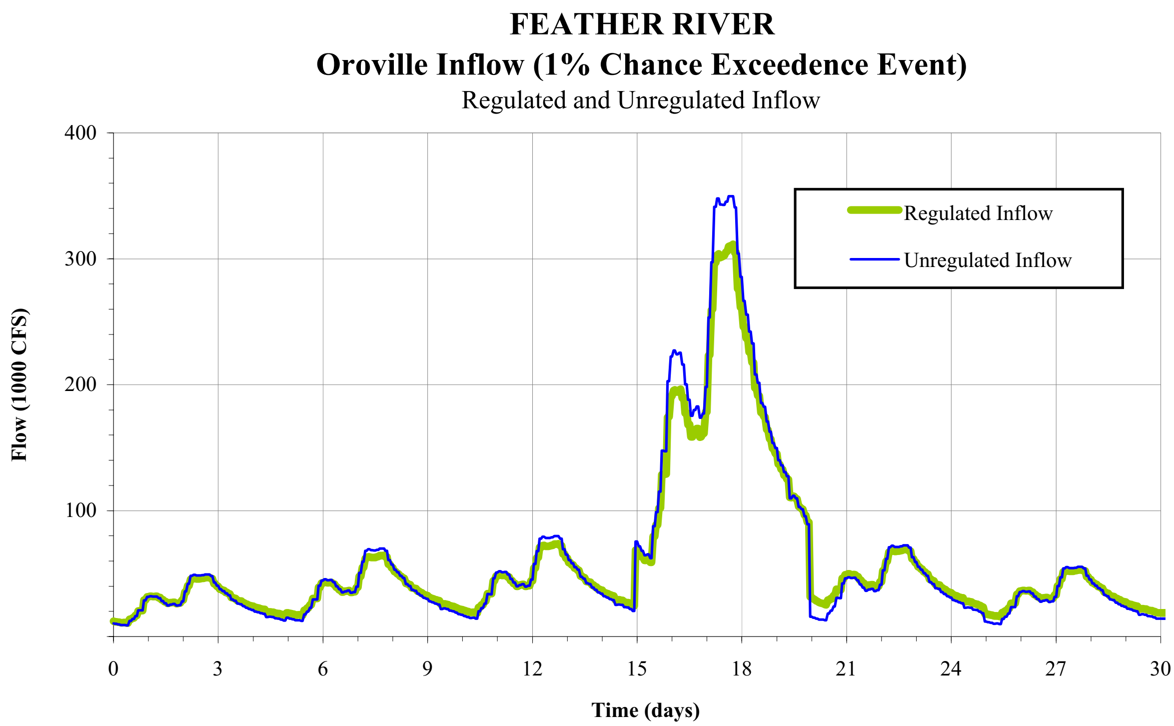


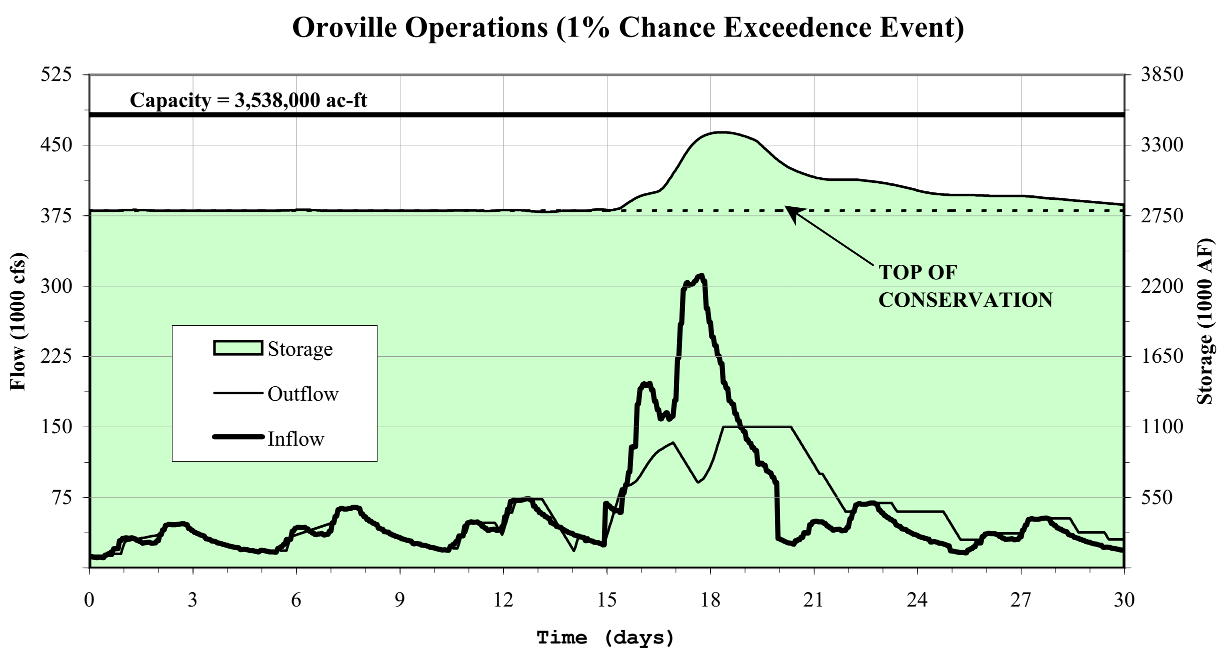
Oroville Operations (2% Chance Exceedence Event)



Notes:
1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

Yuba River Basin Project GRR	
Plate 8 Reservoir Simulation Hydrographs Oroville (2% Chance Exceedence Event)	
US Army Corps of Engineers	August 2004



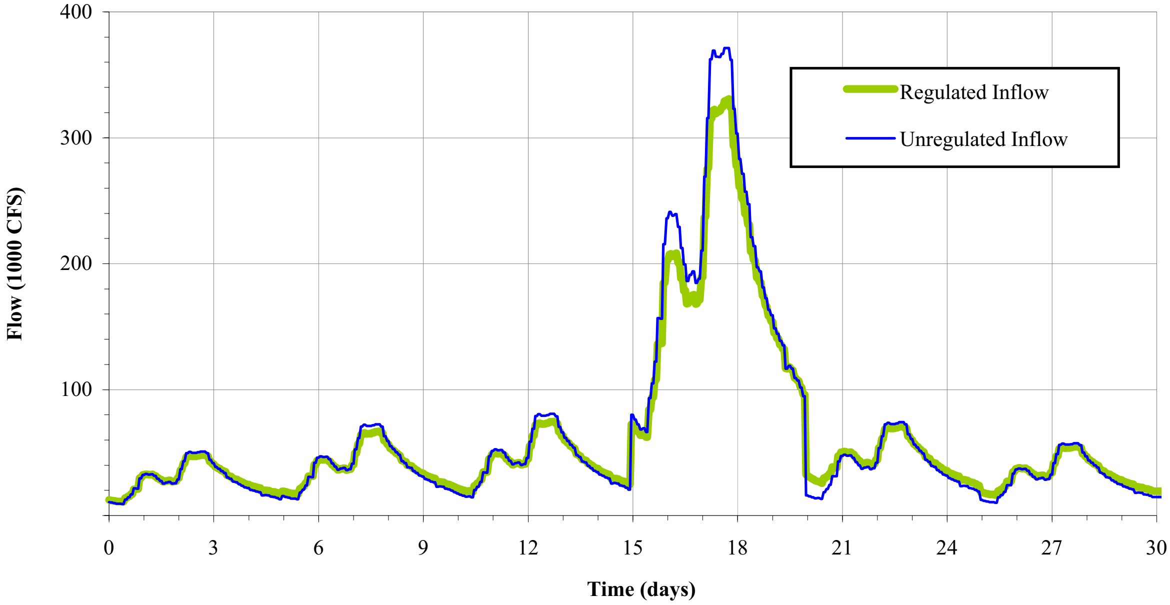


Notes:

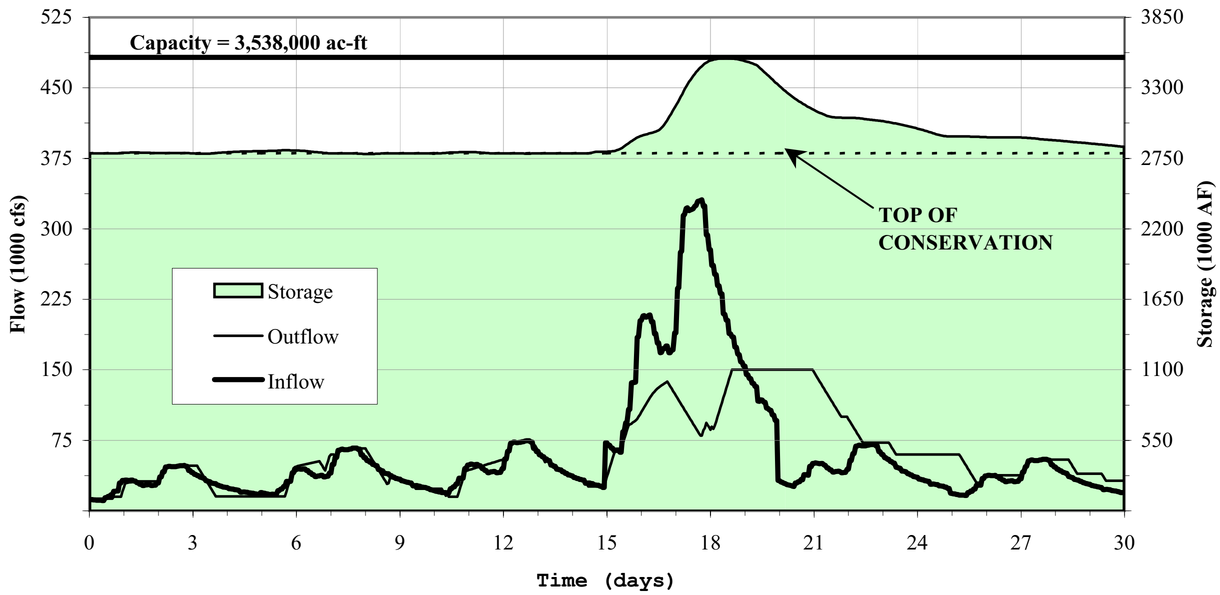
1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

Yuba River Basin Project GRR	
Plate 9 Reservoir Simulation Hydrographs Oroville (1% Chance Exceedence Event)	
US Army Corps of Engineers	August 2004

FEATHER RIVER
Oroville Inflow (0.8% Chance Exceedence Event)
Regulated and Unregulated Inflow



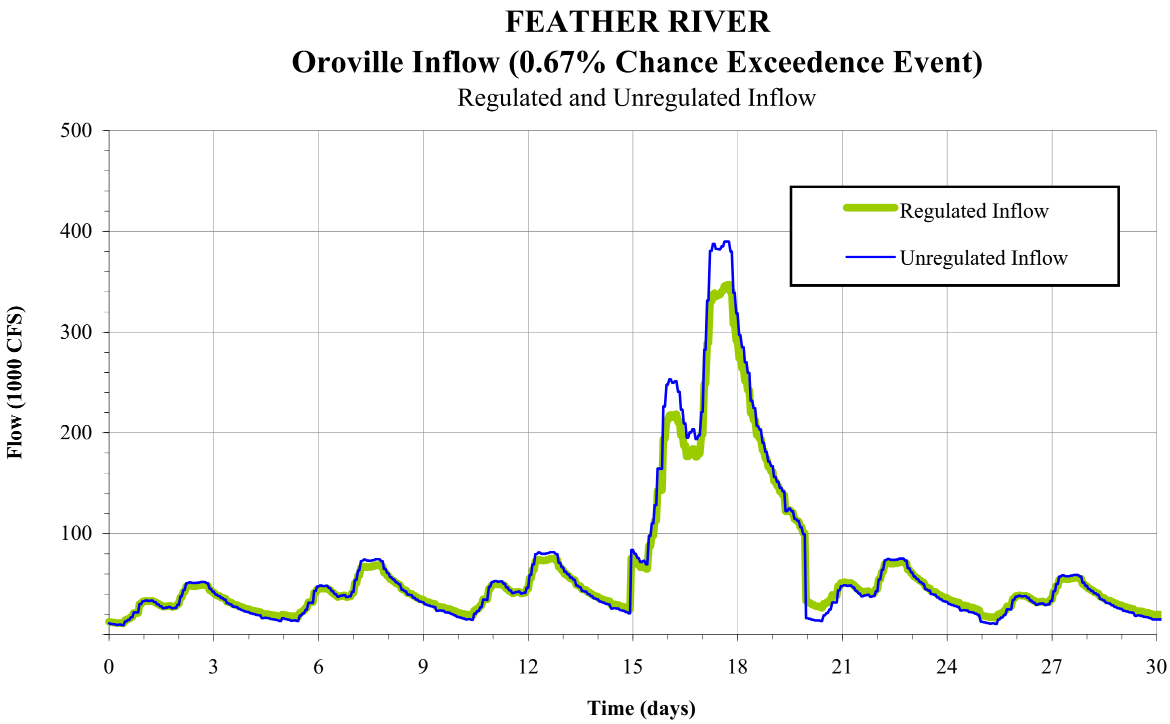
Oroville Operations (0.8% Chance Exceedence Event)

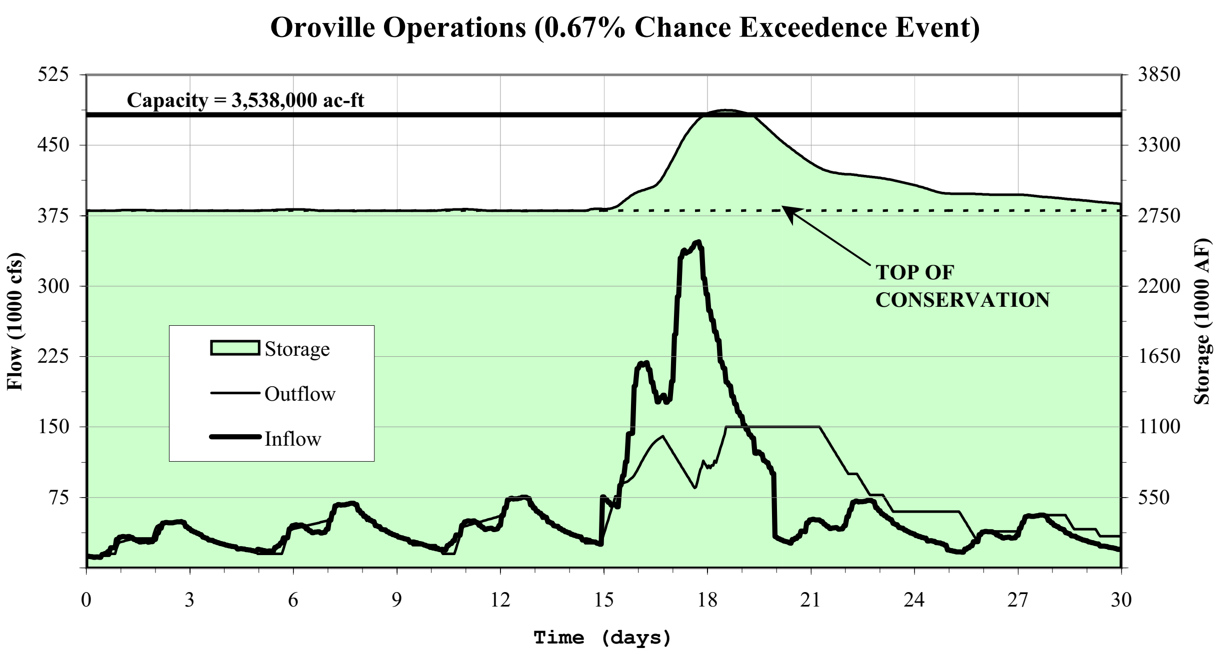


Notes:
1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

Yuba River Basin Project GRR	
Plate 10 Reservoir Simulation Hydrographs Oroville (0.8% Chance Exceedence Event)	
US Army Corps of Engineers	August 2004

prepared by B.J.W.





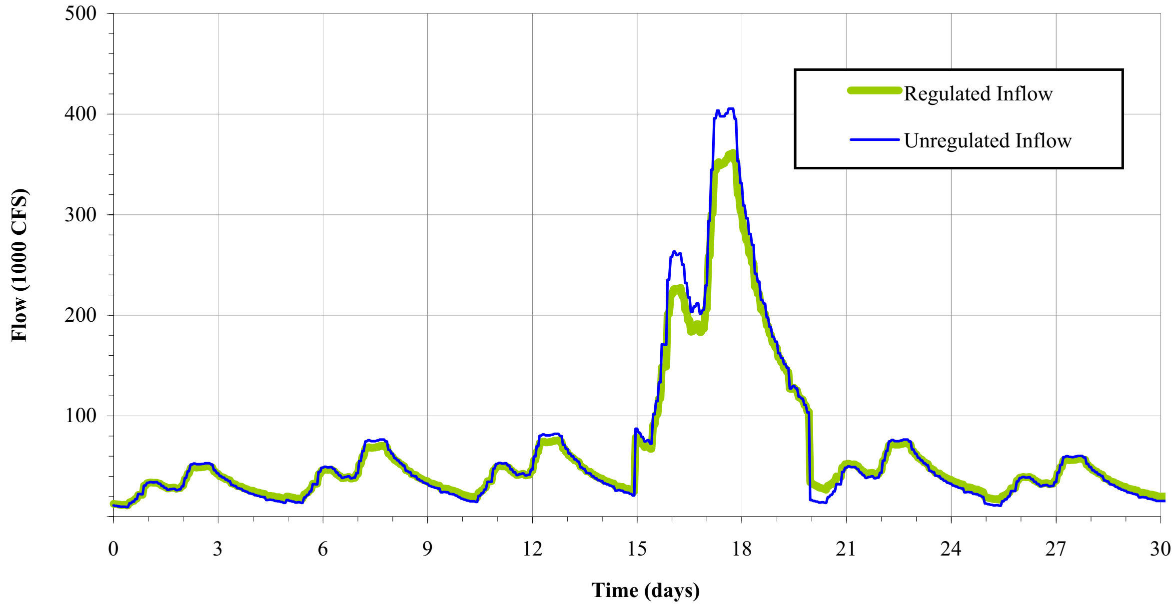
Yuba River Basin Project GRR	
Plate 11 Reservoir Simulation Hydrographs Oroville (0.67% Chance Exceedence Event)	
US Army Corps of Engineers	August 2004

Notes:

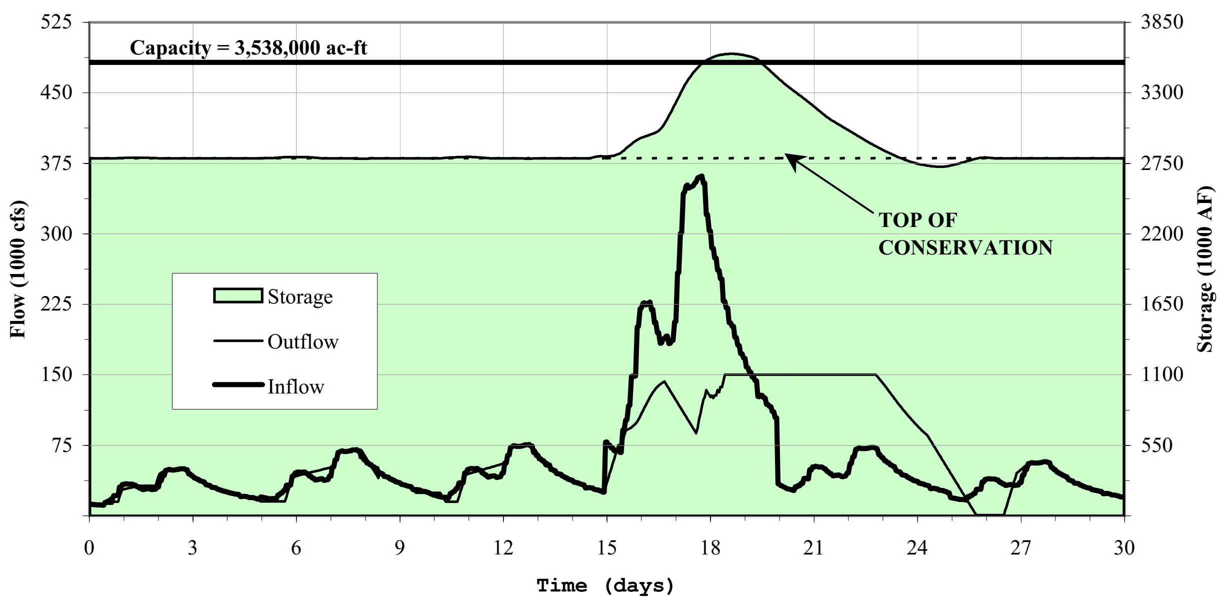
1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

prepared by B.J.W.

FEATHER RIVER
Oroville Inflow (0.57% Chance Exceedence Event)
Regulated and Unregulated Inflow



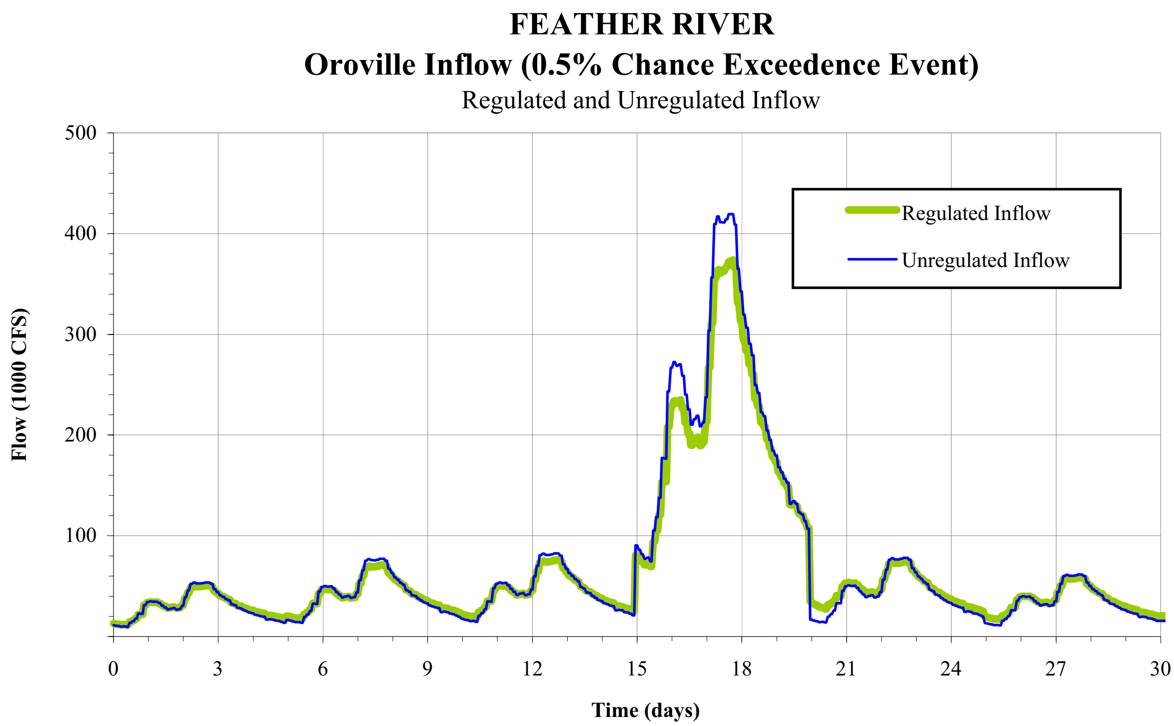
Oroville Operations (0.57% Chance Exceedence Event)

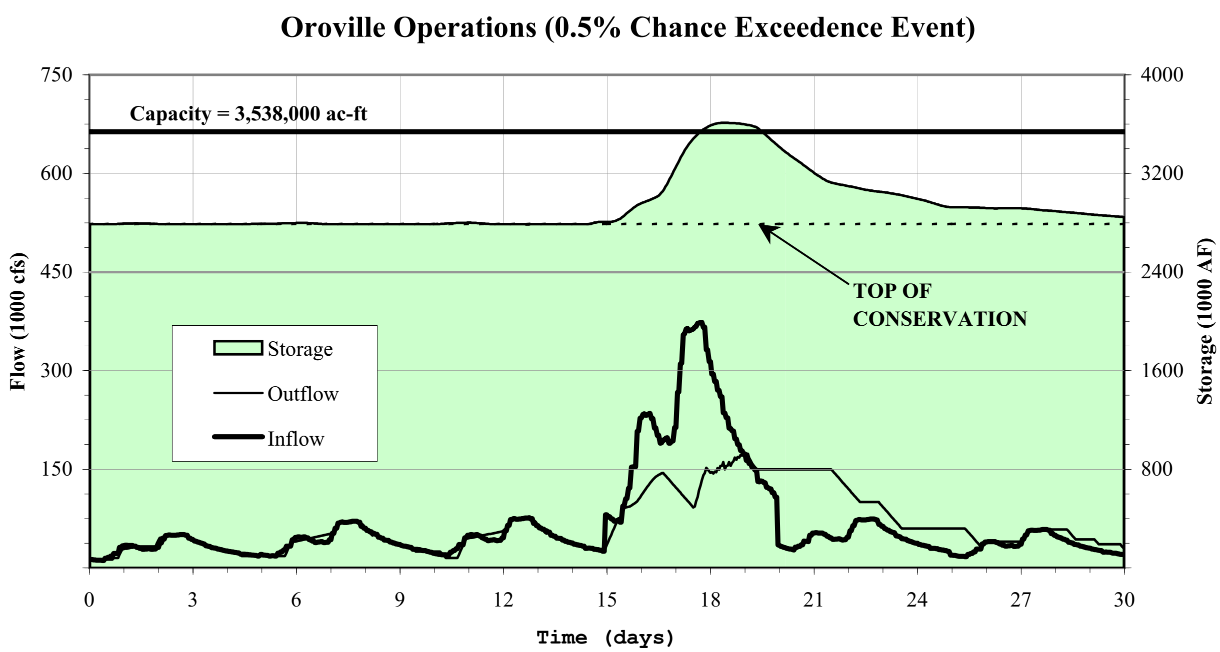


Notes:
1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

Yuba River Basin Project GRR	
Plate 12 Reservoir Simulation Hydrographs Oroville (0.57% Chance Exceedence Event)	
US Army Corps of Engineers	August 2004

prepared by B.J.W.



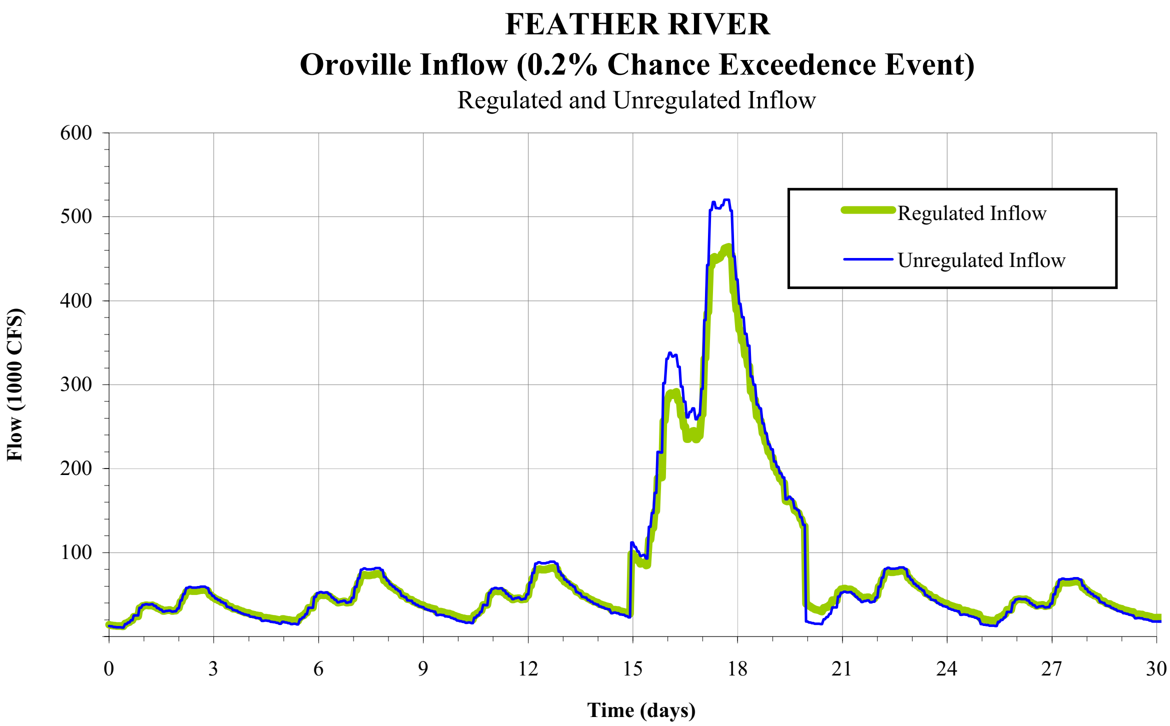


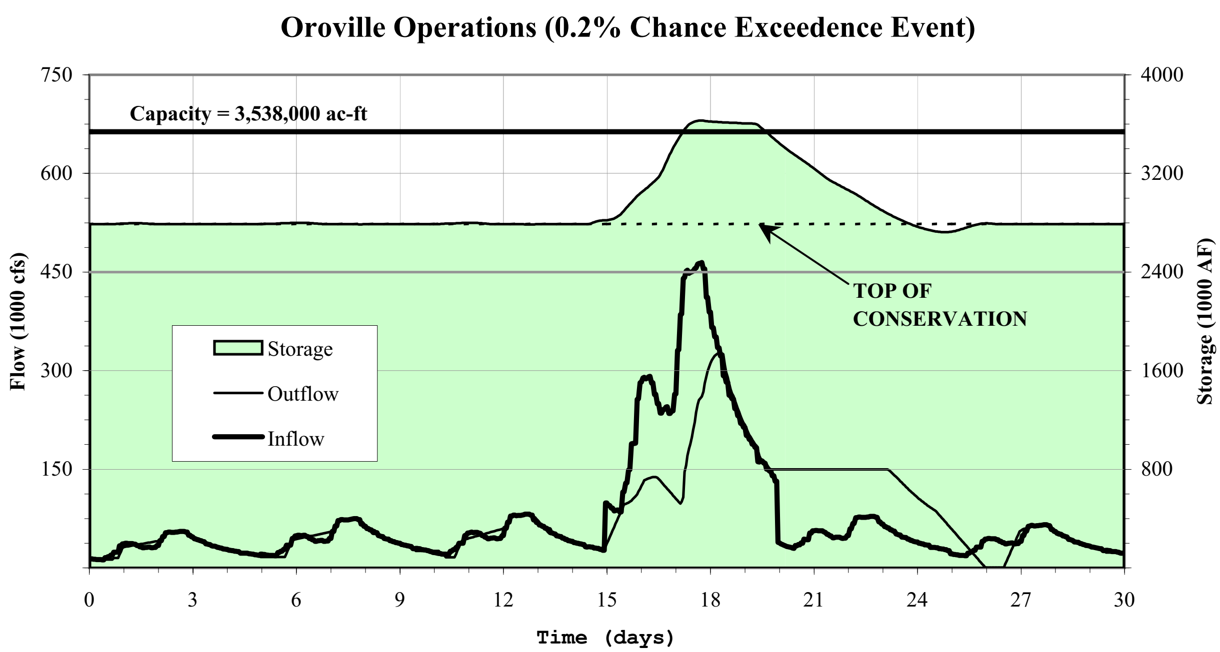
Notes:

1) Results are from Feather River Storm Centering A
2) 20% uncertainty in local flow assumed

Yuba River Basin Project GRR	
Plate 13 Reservoir Simulation Hydrographs Oroville (0.5% Chance Exceedence Event)	
US Army Corps of Engineers	August 2004

prepared by B.J.W.





Notes:
1) Results are from Feather River Storm Centering A
2) 20% uncertainty in local flow assumed

Yuba River Basin Project GRR
Plate 14 Reservoir Simulation Hydrographs Oroville (0.2% Chance Exceedence Event)
US Army Corps of Engineers   August 2004

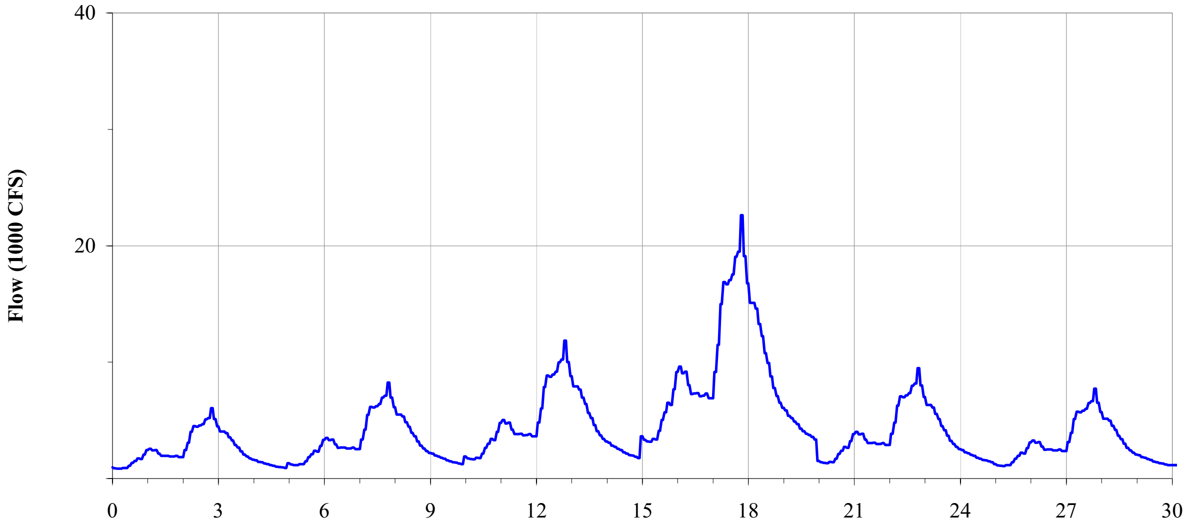
prepared by B.J.W.

NORTH YUBA RIVER

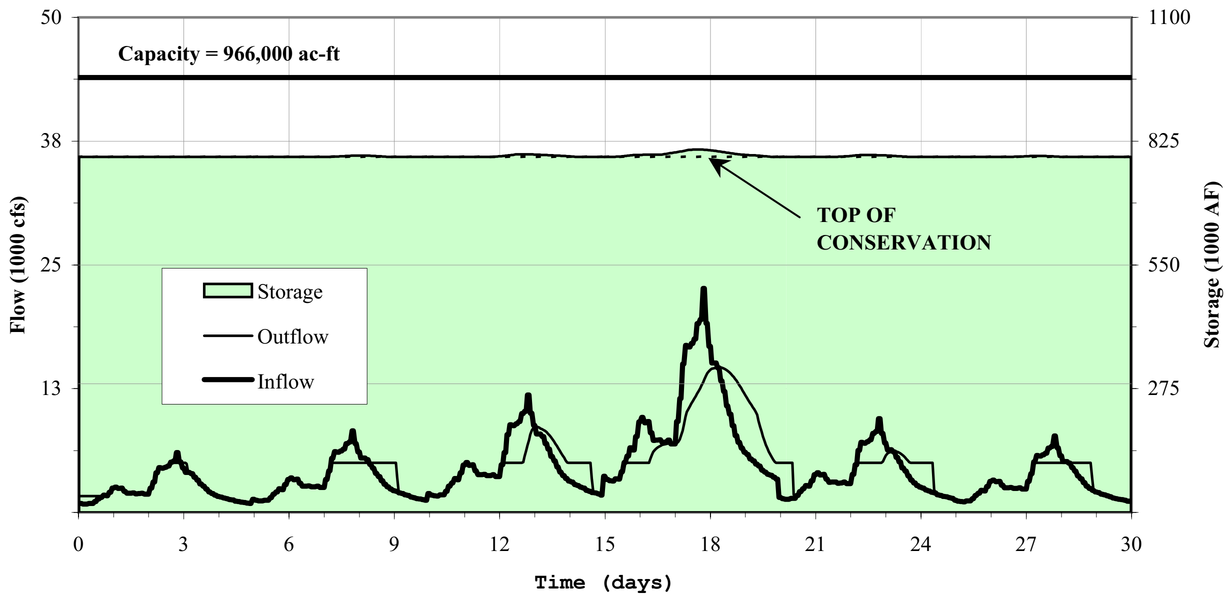
New Bullards Bar Inflow (50% Chance Exceedence Event)

Regulated and Unregulated Inflow

Note: This reservoir has no major headwater reservoirs



New Bullards Bar Operations (50% Chance Exceedence Event)

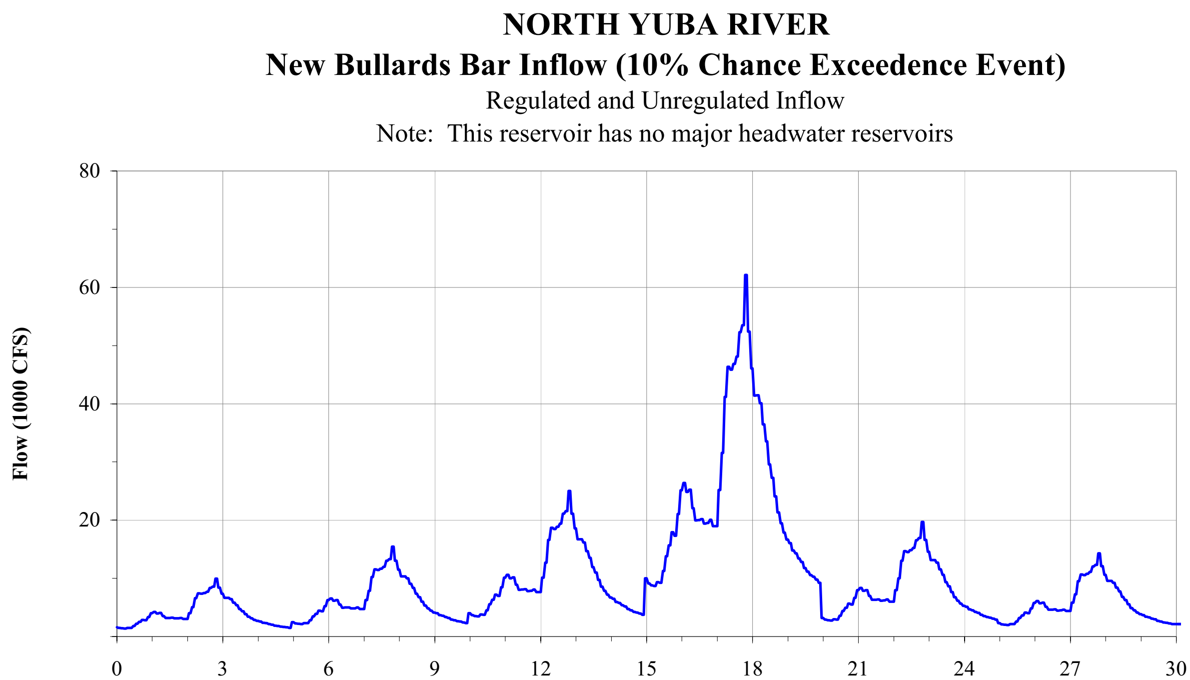


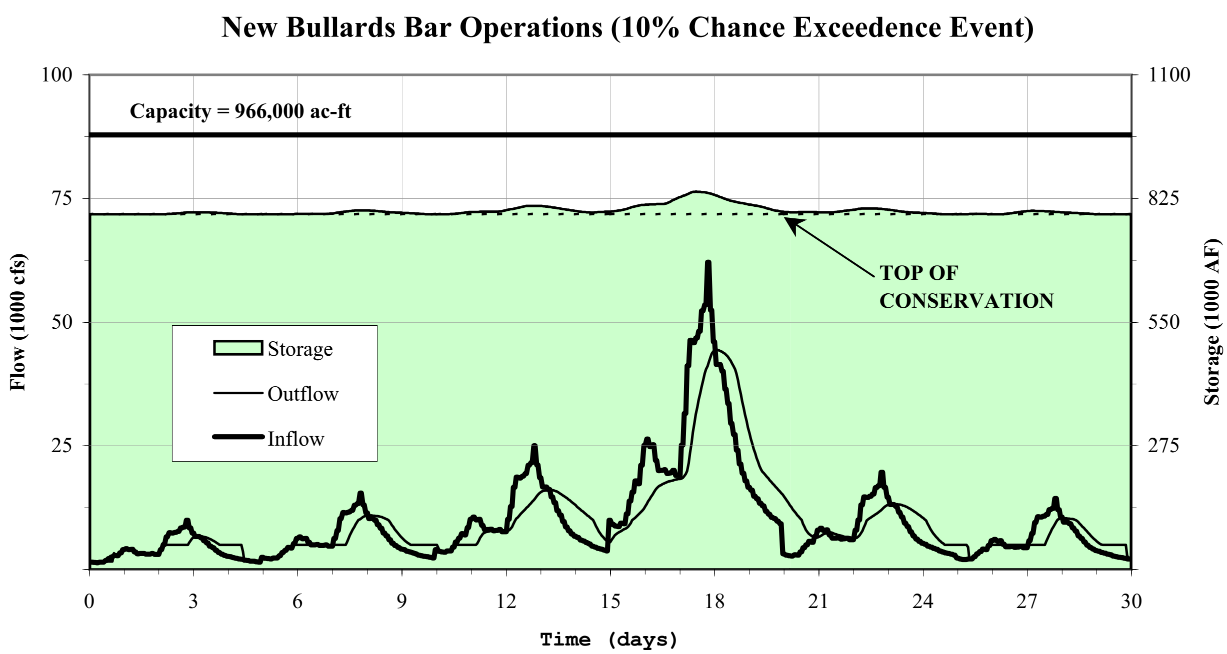
Notes:

1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

Yuba River Basin Project GRR	
Plate 15 Reservoir Simulation Hydrographs New Bullards Bar (50% Chance Exceedence Event)	
US Army Corps of Engineers	August 2004

prepared by B.J.W.





Notes:
1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

Yuba River Basin Project GRR

Plate 16
Reservoir Simulation Hydrographs
New Bullards Bar
(10% Chance Exceedence Event)

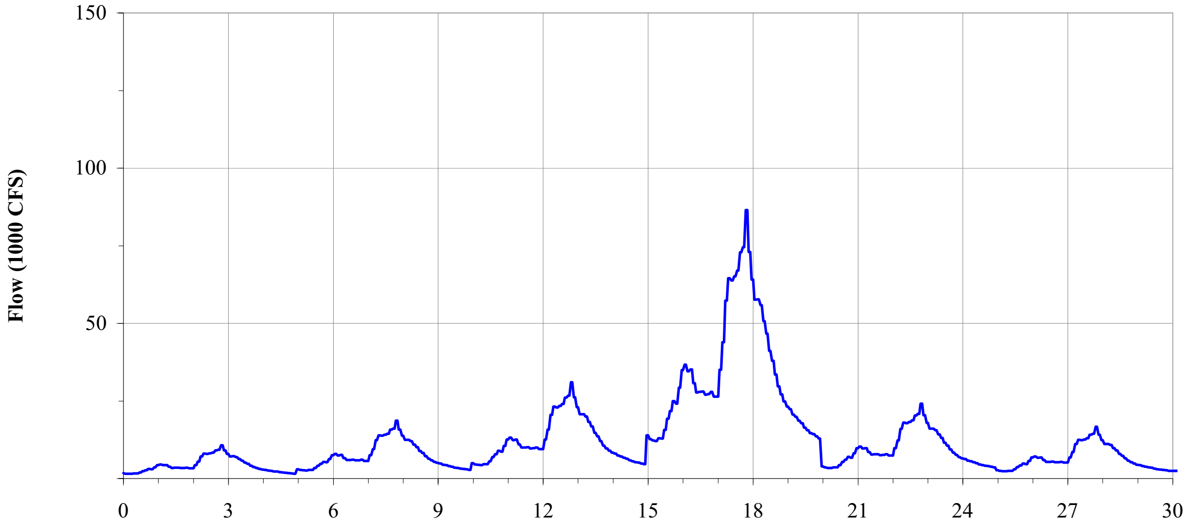
US Army Corps of Engineers **August 2004**

prepared by B.J.W.

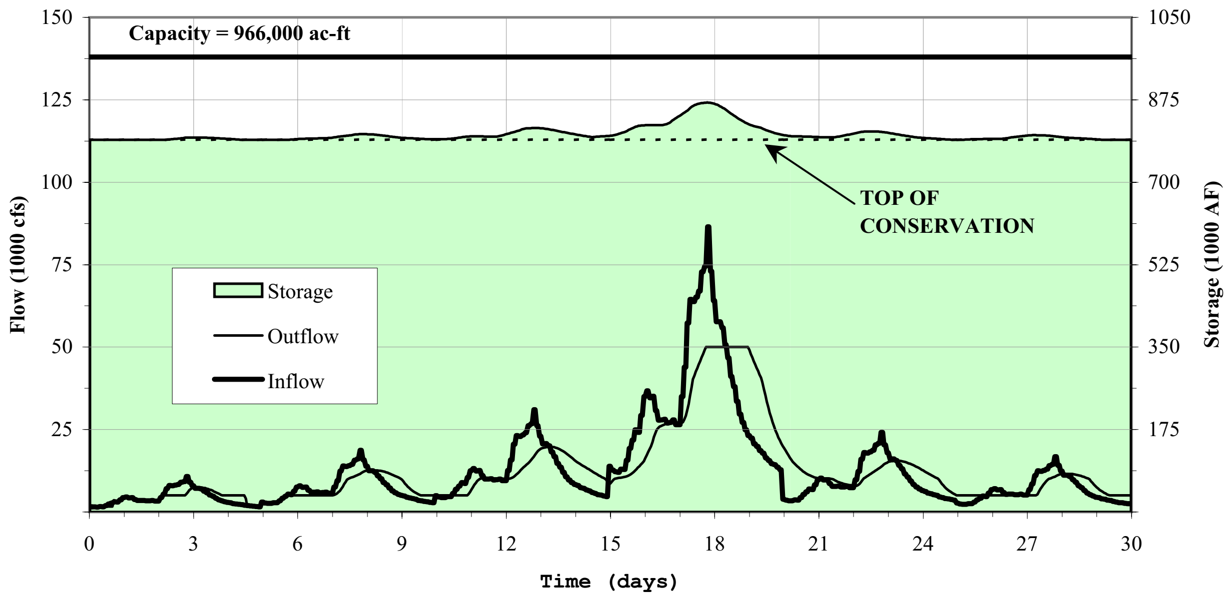
NORTH YUBA RIVER
New Bullards Bar Inflow (4% Chance Exceedence Event)

Regulated and Unregulated Inflow

Note: This reservoir has no major headwater reservoirs



New Bullards Bar Operations (4% Chance Exceedence Event)



Notes:

1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

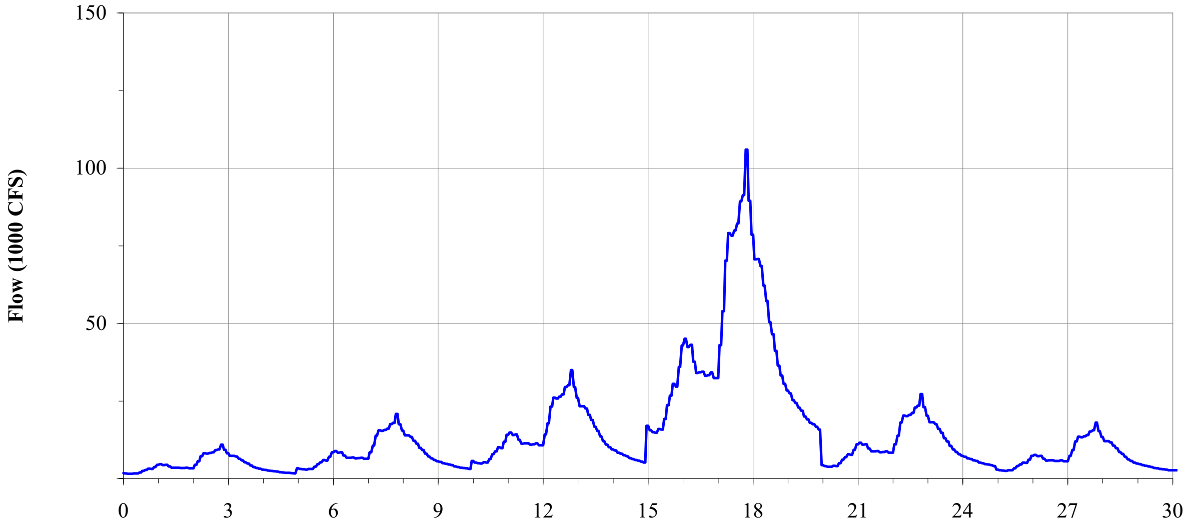
Yuba River Basin Project GRR	
Plate 17 **Reservoir Simulation Hydrographs** **New Bullards Bar** **(4% Chance Exceedence Event)**	
US Army Corps of Engineers	**August 2004**

prepared by B.J.W.

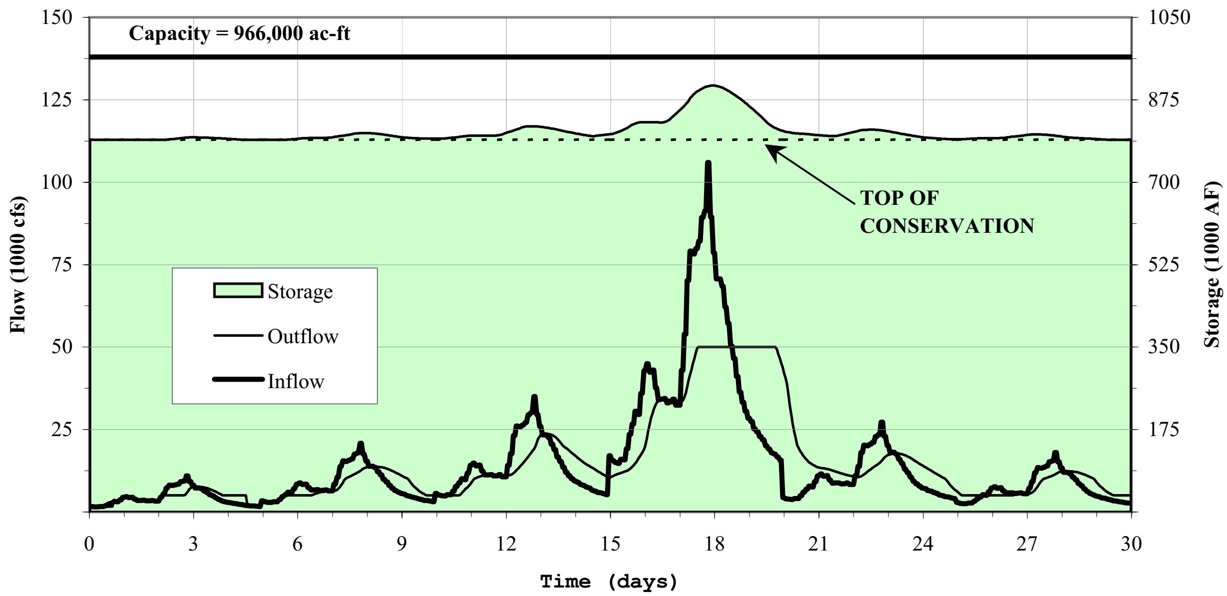
NORTH YUBA RIVER
New Bullards Bar Inflow (2% Chance Exceedence Event)

Regulated and Unregulated Inflow

Note: This reservoir has no major headwater reservoirs



New Bullards Bar Operations (2% Chance Exceedence Event)



Yuba River Basin Project GRR	
Plate 18 **Reservoir Simulation Hydrographs** **New Bullards Bar** **(2% Chance Exceedence Event)**	
US Army Corps of Engineers	**August 2004**

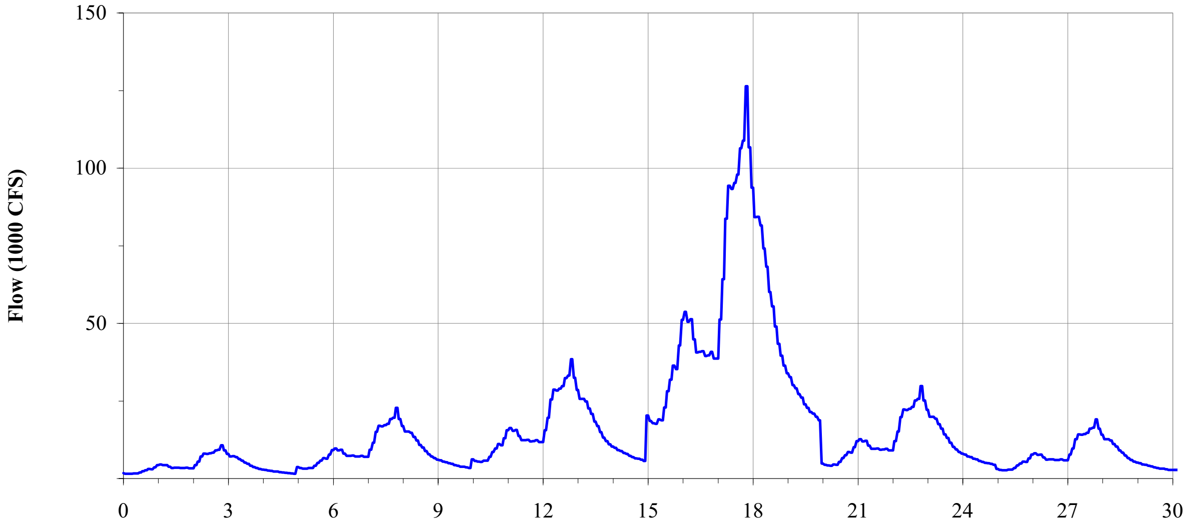
Notes:

1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

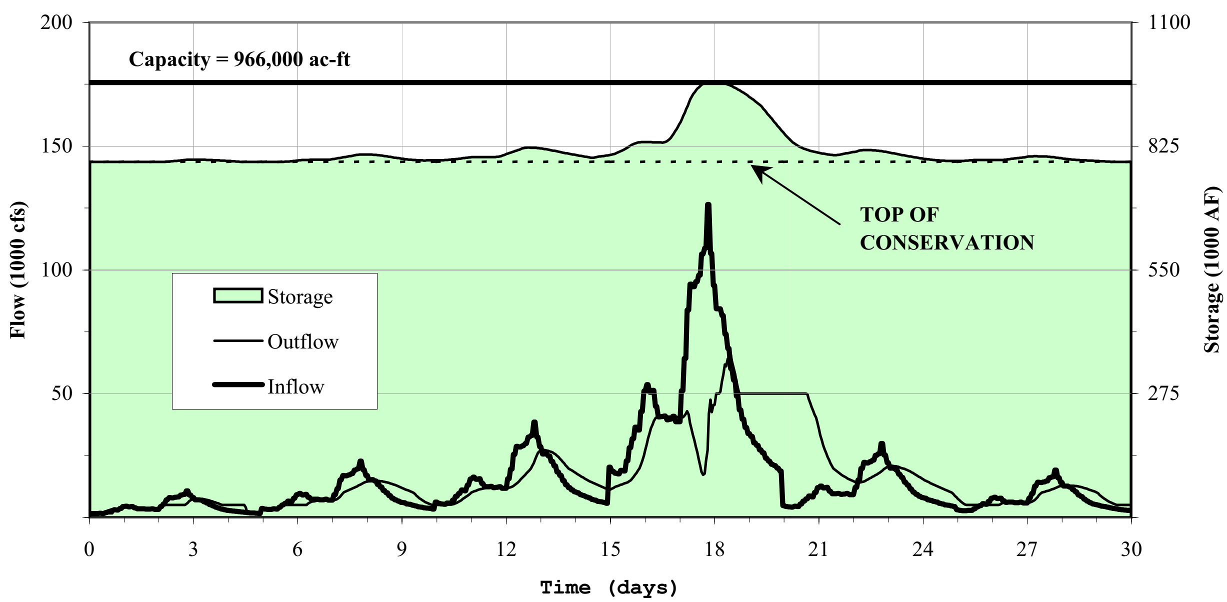
prepared by B.J.W.

NORTH YUBA RIVER
New Bullards Bar Inflow (1% Chance Exceedence Event)
Regulated and Unregulated Inflow

Note: This reservoir has no major headwater reservoirs



New Bullards Bar Operations (1% Chance Exceedence Event)



Notes:

1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

Yuba River Basin Project GRR	
Plate 19 Reservoir Simulation Hydrographs New Bullards Bar (1% Chance Exceedence Event)	
US Army Corps of Engineers	August 2004

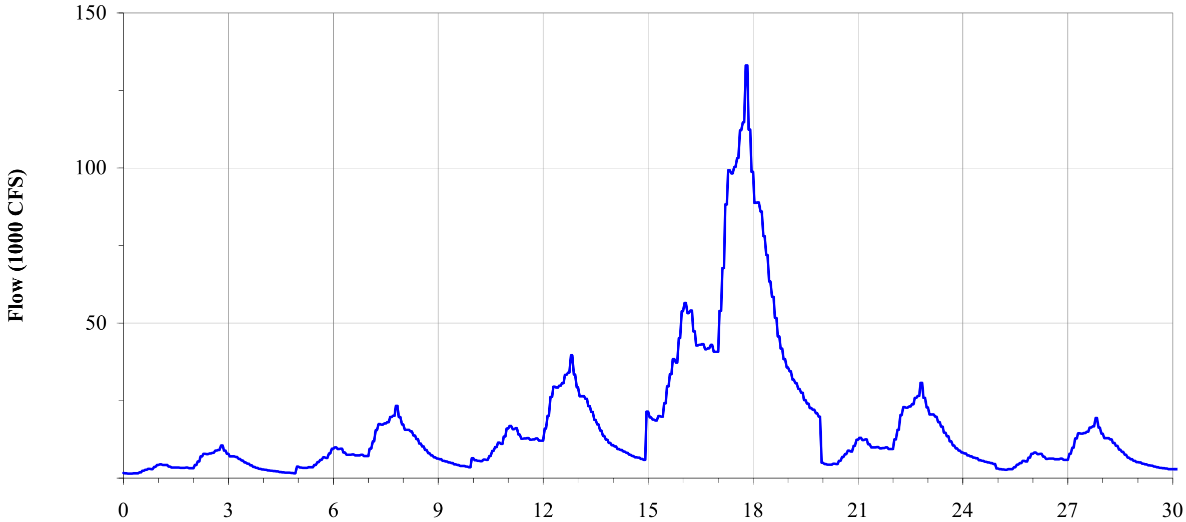
prepared by B.J.W.

NORTH YUBA RIVER

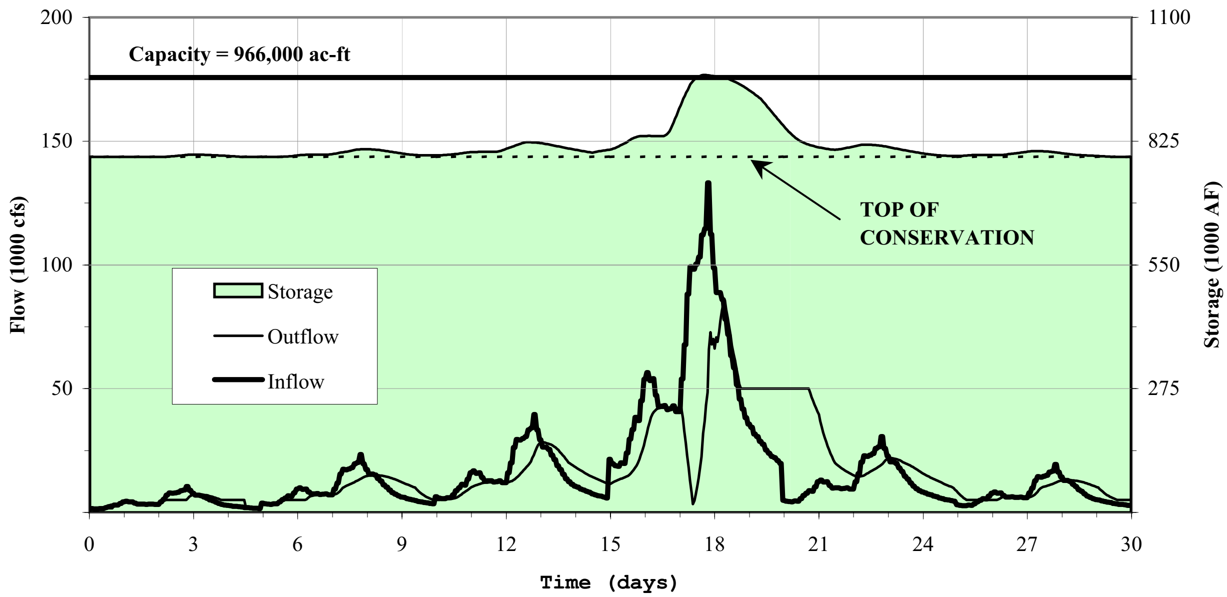
New Bullards Bar Inflow (0.8% Chance Exceedence Event)

Regulated and Unregulated Inflow

Note: This reservoir has no major headwater reservoirs



New Bullards Bar Operations (0.8% Chance Exceedence Event)



Notes:

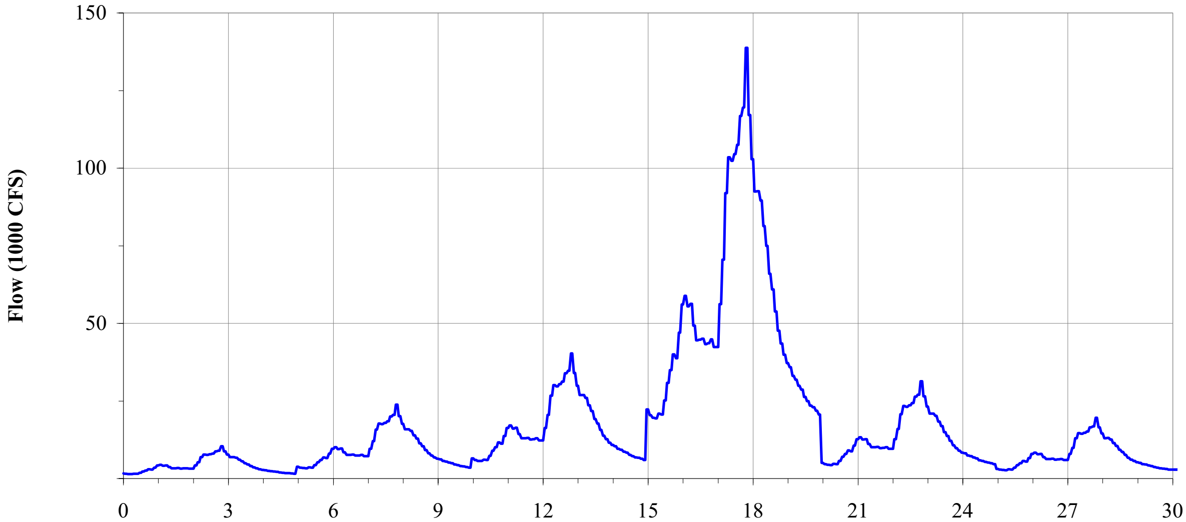
1) Results are from Feather River Storm Centering A

2) Complete certainty in local flow assumed

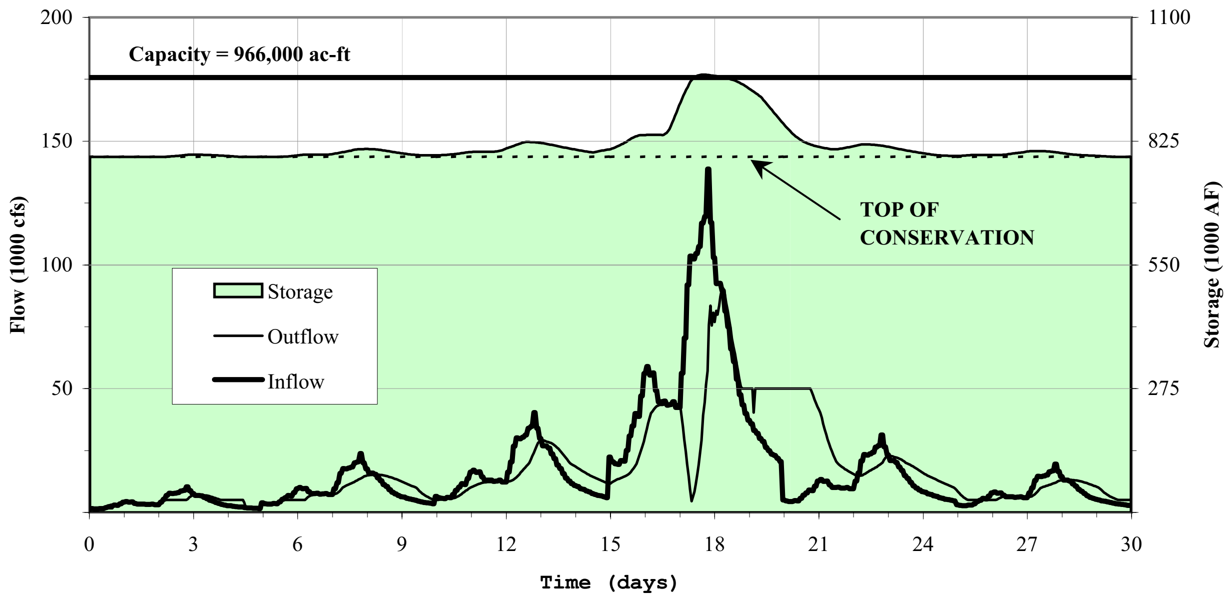
Yuba River Basin Project GRR	
Plate 20 **Reservoir Simulation Hydrographs** **New Bullards Bar** **(0.8% Chance Exceedence Event)**	
US Army Corps of Engineers	**August 2004**

prepared by B.J.W.

NORTH YUBA RIVER
New Bullards Bar Inflow (0.67% Chance Exceedence Event)
Regulated and Unregulated Inflow
Note: This reservoir has no major headwater reservoirs



New Bullards Bar Operations (0.67% Chance Exceedence Event)



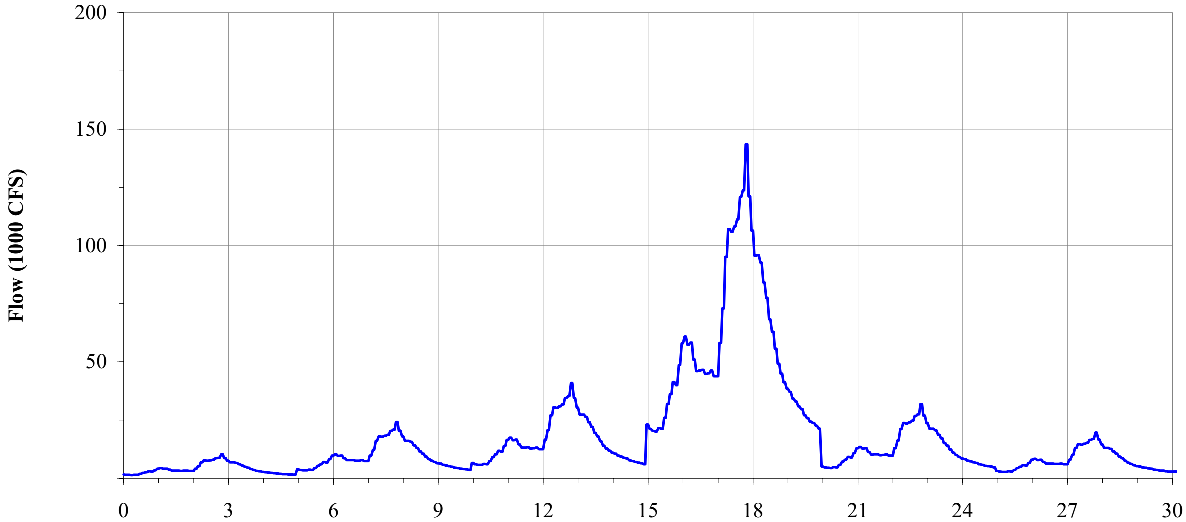
Notes:
1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

Yuba River Basin Project GRR	
Plate 21 Reservoir Simulation Hydrographs New Bullards Bar (0.67% Chance Exceedence Event)	
US Army Corps of Engineers	August 2004

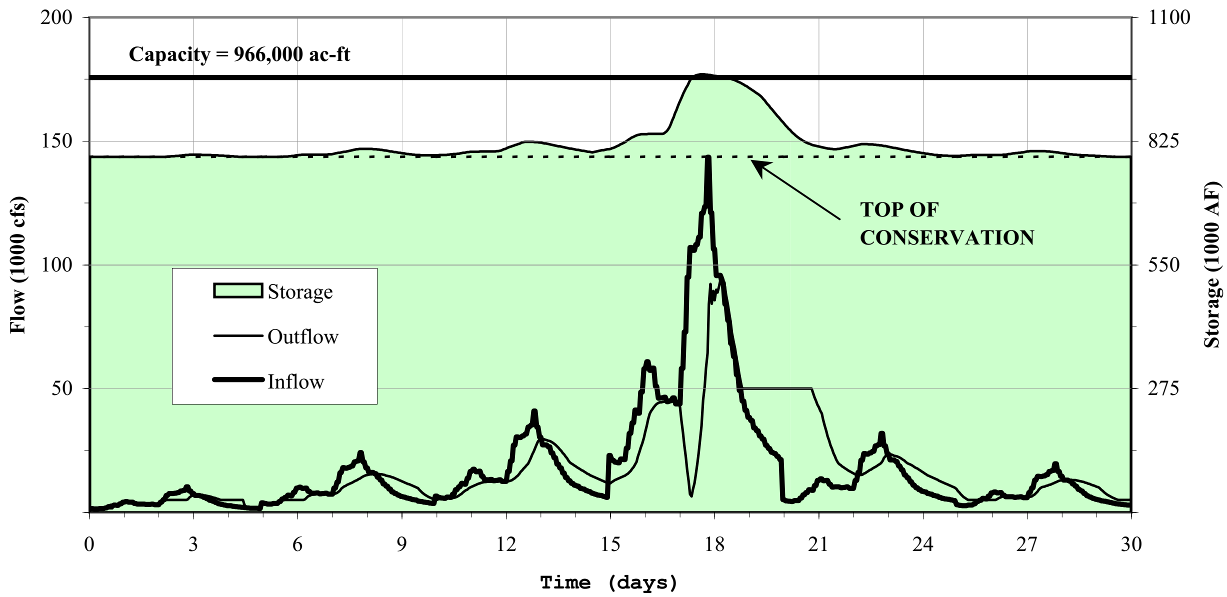
prepared by B.J.W.

NORTH YUBA RIVER
New Bullards Bar Inflow (0.57% Chance Exceedence Event)
Regulated and Unregulated Inflow

Note: This reservoir has no major headwater reservoirs



New Bullards Bar Operations (0.57% Chance Exceedence Event)

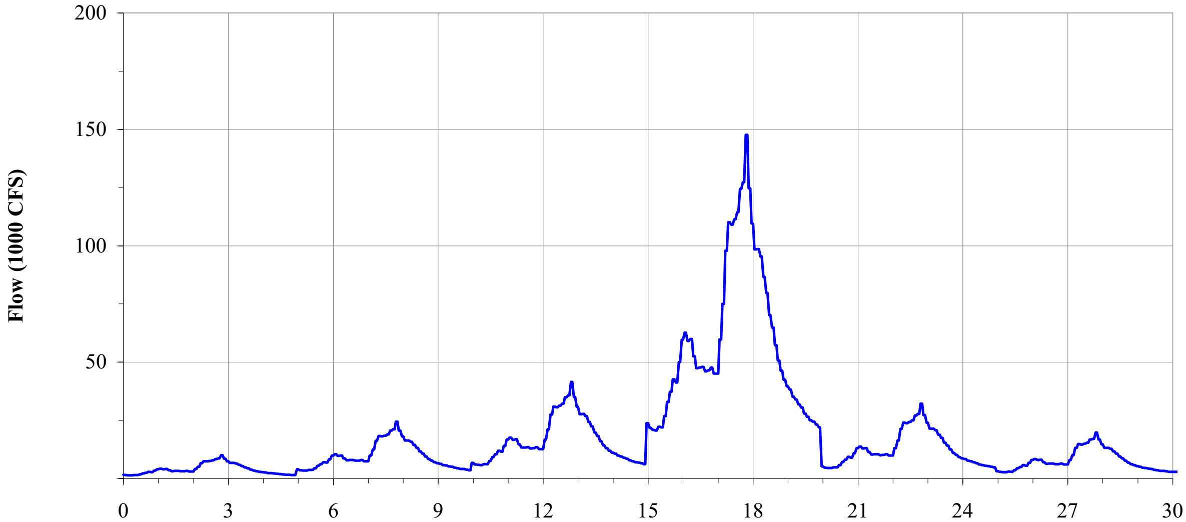


Notes:
1) Results are from Feather River Storm Centering A
2) Complete certainty in local flow assumed

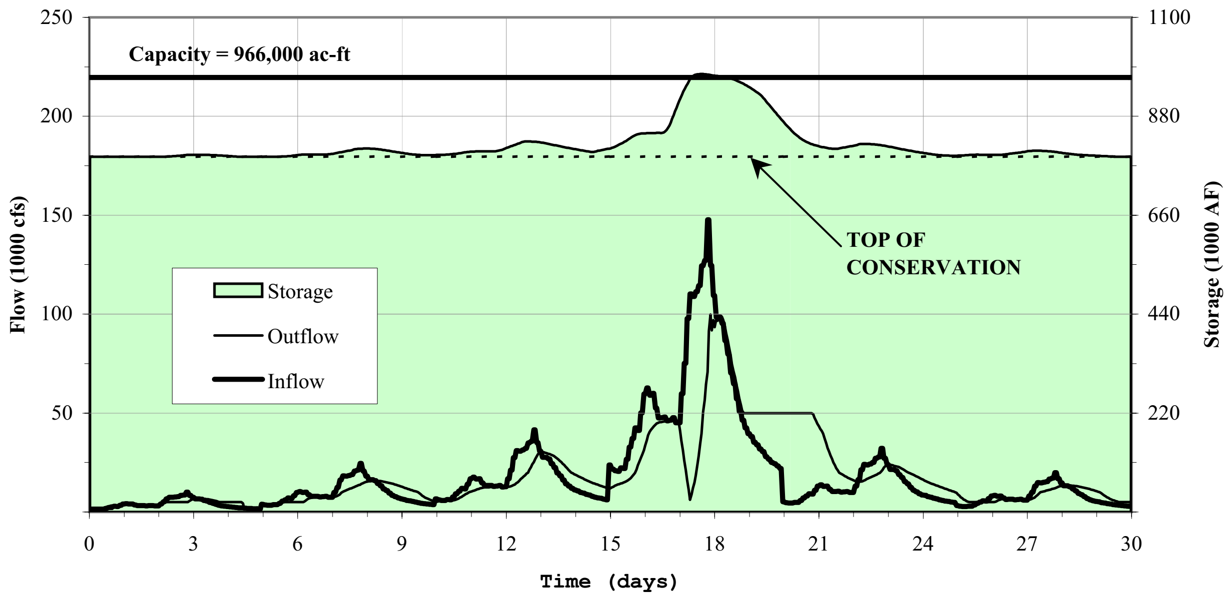
Yuba River Basin Project GRR	
Plate 22 Reservoir Simulation Hydrographs New Bullards Bar (0.57% Chance Exceedence Event)	
US Army Corps of Engineers	August 2004

prepared by B.J.W.

NORTH YUBA RIVER
New Bullards Bar Inflow (0.5% Chance Exceedence Event)
Regulated and Unregulated Inflow
Note: This reservoir has no major headwater reservoirs



New Bullards Bar Operations (0.5% Chance Exceedence Event)



Notes:
1) Results are from Feather River Storm Centering A
2) 20% uncertainty in local flow assumed

Yuba River Basin Project GRR	
Plate 23 Reservoir Simulation Hydrographs New Bullards Bar (0.5% Chance Exceedence Event)	
US Army Corps of Engineers	August 2004

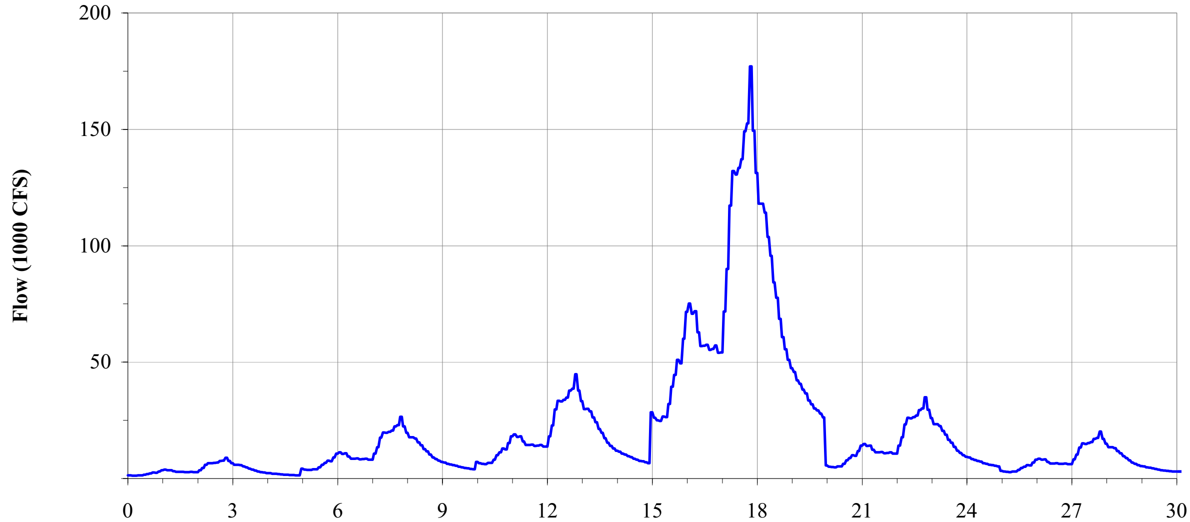
prepared by B.J.W.

NORTH YUBA RIVER

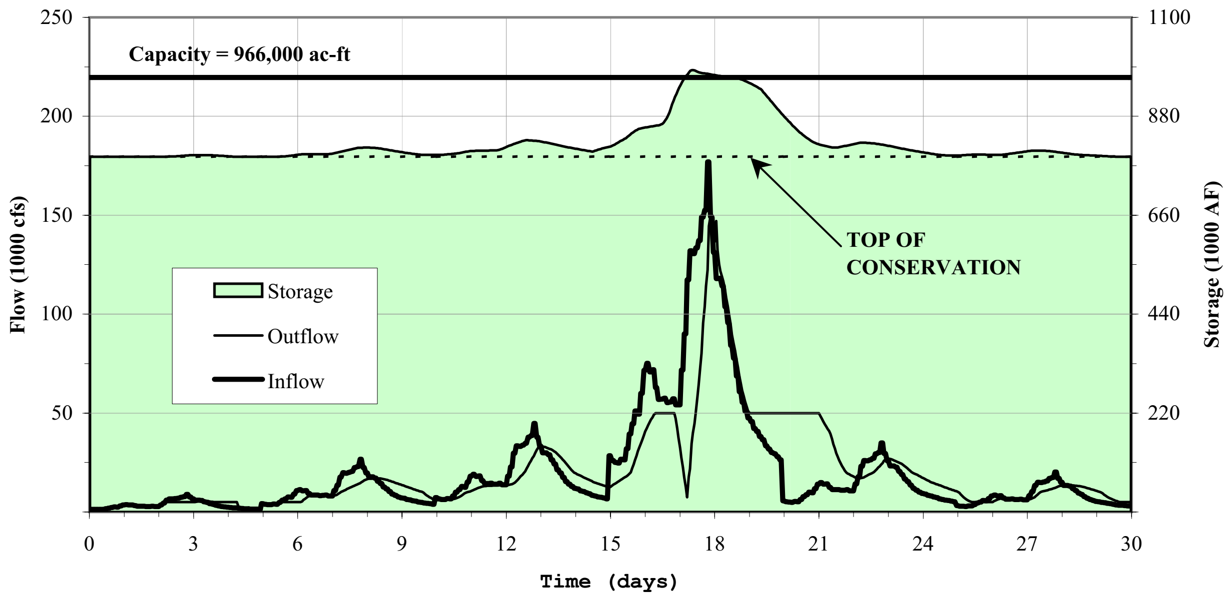
New Bullards Bar Inflow (0.2% Chance Exceedence Event)

Regulated and Unregulated Inflow

Note: This reservoir has no major headwater reservoirs



New Bullards Bar Operations (0.2% Chance Exceedence Event)



Notes:

1) Results are from Feather River Storm Centering A

2) 20% uncertainty in local flow assumed

Yuba River Basin Project GRR	
Plate 24 **Reservoir Simulation Hydrographs** **New Bullards Bar** **(0.2% Chance Exceedence Event)**	
US Army Corps of Engineers	**August 2004**

prepared by B.J.W.

American River Watershed Project

Post-Authorization Change Report
And Interim General Reevaluation Report

American River Watershed

Common Features Project
Natomas Basin
Sacramento and Sutter Counties, California

SAFCA
Sacramento Area Flood Control Agency

CENTRAL VALLEY FLOOD PROTECTION BOARD
PUBLIC SAFETY AND FLOOD MANAGEMENT
STATE OF CALIFORNIA

US Army Corps of Engineers®
Sacramento District

Appendix B – Hydrology Technical Documentation

August 2010

Appendix B

Appendix B1 – Synthetic Hydrology Technical Documentation

Appendix B2 – American River Hydrology and Folsom Dam Reservoir Operations

US Army Corps of Engineers
Sacramento District

American River Watershed
Common Features Project
Natomas Post-Authorization Change Report

Appendix B1
Synthetic Hydrology Technical Documentation

September 2008

AMERICAN RIVER WATERSHED COMMON FEATURES PROJECT
NATOMAS POST-AUTHORIZATION CHANGE REPORT
SYNTHETIC HYDROLOGY TECHNICAL DOCUMENTATION

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF PLATES

Note: Cover photo shows the mouth of the Natomas Cross Canal under the Garden Highway.

AMERICAN RIVER WATERSHED COMMON FEATURES PROJECT
NATOMAS POST-AUTHORIZATION CHANGE REPORT
SYNTHETIC HYDROLOGY TECHNICAL DOCUMENTATION

1.0 Documentation for Synthetic Flood Centerings

This chapter cites the documentation used to develop the hydrographs provided to Hydraulic Design Section as input for its calibrated HEC-RAS 4.0 model – the model used to develop water surface profiles for existing conditions (year 2007). Multiple flood centerings were tested to assure that the controlling hydrologic events were used for the hydraulic analysis. Each centering consisted of flow hydrographs developed for the specific frequency events: 50-, 10-, 4-, 2-, 1-, 0.5-, and 0.2 percent exceedence floods (8-Flood Series). The three flood centerings tested were the Sacramento Mainstem, Shanghai Bend-Yuba River, and the American River. The study area includes the Sacramento River from the Natomas Cross Canal down to Freeport and the American River from Folsom Dam down to its confluence with the Sacramento River, as well as the Natomas tributary drainage to the Natomas Cross Canal and to Steelhead Creek. **Plate 1**, the general map, shows the watersheds for the four Natomas tributaries to Steelhead Creek, the five Natomas tributaries to the Natomas Cross Canal, the American River south of the Natomas tributaries, the Feather River at its confluence with the Sacramento River, and the Sacramento River from upstream of Feather River down to its confluence with the American River. **Plate 2** shows where the hydraulic model input locations are for the five hydrographs contributing to the Natomas Cross Canal and the four hydrographs contributing to Steelhead Creek. Steelhead Creek is also known as the Natomas East Main Drainage Canal (NEMDC). The hydrographs are for an unsteady state simulation.

The three different flood centerings mentioned above are being tested in the hydraulic model to see which one produces the highest stages in which locations of the study area. Under certain conditions the American River is the controlling flood event for Steelhead Creek. The Shanghai Bend centering or the Sacramento Mainstem centering may be the controlling flood event for the Natomas Cross Canal. However, which flood centering series will produce the most critical flooding at which locations will not be known without hydraulic analysis.

1.1 <u>Sacramento Mainstem Centering</u>. The flood centering hydrographs were created using the methodology developed in the Comprehensive Study (the “Sacramento and San Joaquin River Basins Comprehensive Study,” Technical Studies Documentation, dated December 2002, abbreviated here as Comp Study and described in **Reference 1**). The Comprehensive Study models were developed for use in regional, broad concept studies, such as the Sacramento Common Features General Reevaluation study. **Reference 1, Appendix B**: “Synthetic Hydrology Technical Documentation,” describes the development of the unregulated flood hydrographs.

Unregulated flow frequency curves were developed at key mainstem and tributary locations in the Sacramento River basin. The unregulated frequency curves plot historic flood peaks and volumes with the statistical distributions of unimpaired flows (with no reservoir influence). The frequency curves display volumes, or average flow rates, for different time durations over a range of annual exceedence probabilities. These curves are used to translate: 1) hydrographs to frequencies; and 2) frequencies to flood volumes. As part of the Comprehensive Study (Comp Study), flow frequency curves were developed for 1-, 3-, 7-, 15-, and 30-day durations. A routing model was developed to route the unregulated daily flows from the tributary locations to downstream locations for use in constructing mainstem "index" frequency curves. Mainstem locations include the Sacramento River at the Latitude of Sacramento (including flows down the Yolo Bypass) and the Feather River downstream of the Yuba River (at Shanghai Bend). The maximum flows for each winter at the mainstem locations were used to develop flow frequency curves (for 1-, 3-, 7-, 15-, and 30-day durations) for those mainstem locations. No synthetic precipitation events were needed for the hydrology. This paragraph and the paragraphs below explain the development of the synthetic flood centerings for the latitude of Sacramento; the flood centerings for Shanghai Bend were developed similarly.

Based on analysis of historic floods over the Sacramento watershed, synthetic mainstem flood centerings were developed to stress widespread valley areas. The flow frequency curves for the Latitude of Sacramento (used for the Sacramento Mainstem Centering) provide the hypothetic flood volumes that the basin will produce during simulations of each of the eight synthetic exceedence frequency flood events (50-, 20-, 10-, 4-, 2-, 1-, 0.5-, and 0.2percent). The role of the mainstem centering is to distribute these flood volumes back into the basin, tributary by tributary, in accordance with patterns visible in historic flood events. **Reference 1, Appendix C**: "Reservoir Operations Modeling, Existing Design Operations and Reoperation Analysis," describes the development of the reservoir operations models to route the unregulated hydrographs through the headwater and major flood management reservoirs for input into the hydraulic model.

The Sacramento Mainstem flood hydrographs were developed using the flood patterns shown on **Table 1** to produce flood runoff hydrographs centered at the Latitude of Sacramento. **Table 1** shows the set of synthetic exceedence frequencies assigned to the set of tributaries listed in column 1 such that the regulated and routed hydrographs have the volumes for a flood series centered at the Latitude of Sacramento. The hydrographs have a duration of 30 days, with six 5-day waves. The pattern hydrograph used for the 5-day waves at each upstream tributary is that of the unregulated flood hydrograph for 30 December 1996 to 3 January 1997 (New Year 1997 flood) at that tributary index point. This flood pattern was used because, of the large historical floods over the Sacramento Basin, it is the flood event for which hourly hydrographs were available for the largest number of upstream tributary gages used for the Comp Study. The American River flood hydrographs are different from those used in the Comp Study. See **Section 1.3** for an explanation of the changes made for the American River centering.

Table 1
Sacramento River Mainstem Synthetic Flood Centering

Index Point	Percent Chance Exceedence						
	50%	10%	4%	2%	1%	0.50%	0.20%
Sacramento River at Shasta	84.42	17.03	8.09	4.41	2.21	1.13	0.44
Clear Cr. at Whiskeytown	80.91	17.03	10.79	6.47	3.24	1.66	0.65
Cow Cr. near Millville	80.91	16.18	9.71	5.39	2.70	1.38	0.60
Cottonwood Cr. near Cottonwood	80.91	17.03	10.79	6.47	3.24	1.66	0.65
Battle Cr. Below Coleman FH	80.91	16.18	9.71	5.39	2.70	1.38	0.60
Mill Cr. near Los Molinos	80.91	16.18	9.71	4.22	2.35	1.23	0.51
Elder Cr. near Paskenta	88.26	19.42	10.79	4.85	2.70	1.38	0.58
Thomes Cr. at Paskenta	88.26	19.42	10.79	4.85	2.70	1.38	0.58
Deer Cr. near Vina	88.26	16.18	9.71	4.22	2.35	1.23	0.51
Big Chico Cr. near Chico	88.26	16.18	9.71	4.22	2.35	1.23	0.51
Stony Cr. at Black Butte	88.26	19.42	10.79	4.85	2.70	1.38	0.58
Butte Cr. near Chico	66.70	13.63	6.08	2.75	1.38	0.71	0.30
Feather River at Oroville	53.60	11.78	4.42	2.41	1.20	0.62	0.24
Yuba R. at New Bullards Bar	55.09	12.52	4.86	2.10	1.05	0.54	0.21
Yuba R. at Englebright	55.09	12.52	4.86	2.10	1.05	0.54	0.21
Deer Cr. near Smartsville	55.12	12.52	4.86	2.10	1.05	0.54	0.21
Bear River near Wheatland	53.60	11.13	4.42	2.10	1.05	0.54	0.21
Cache Cr. at Clear Lake	52.19	12.52	6.95	4.45	2.22	1.14	0.45
N.F. Cache Cr. at Indian Vy.	52.19	12.52	6.95	4.45	2.22	1.14	0.45
American River at Folsom	55.09	12.52	4.86	2.51	1.26	0.64	0.25
Putah Cr. at Berryessa	52.19	12.52	6.95	4.45	2.22	1.14	0.45

The process of preparing flood hydrographs begins by using unregulated frequency curves to translate all of the exceedence frequencies in the synthetic patterns to average flow rates. The unregulated frequency curves were prepared using 1-, 3-, 7-, 15-, and 30-day durations. Values for the 5-, 10-, 20-, and 25-day durations were obtained through interpolation. The values from the frequency curves represent the average flow anticipated over a specific time interval. For instance, the 5-day value is the average flow expected during the highest 5-days of flooding during any of the eight synthetic exceedence events. Likewise the 10-day value is the average over the highest 10 days of flooding. Flood volumes were computed by multiplying the average flows by their respective durations. These values represented the total volumes of water anticipated during the highest 5, 10, 15, 20, 25, or 30 days of flows. Furthermore, these flood volumes were portioned into time segments by subtracting volumes of the shorter durations from the next longer duration. For example, the 5-day volume was subtracted from the 10-day volume and the remainder was equal to the amount of flood volume that is produced by the tributary between the 5-day and 10-day maximum periods. This procedure was repeated for the 10-, 15-, 20-, 25-, and 30-day durations and resulted in a set of eight synthetic exceedence frequency flood volumes produced by the tributary.

The basic pattern of all synthetic flood hydrographs was a 30-day hourly time series consisting of 6 waves, each 5 days in duration. Volumes were ranked and distributed into the basic pattern. The highest wave volume was always distributed into the fourth, or main, wave.

The second and third highest volumes preceded and followed the main wave, respectively. The fourth highest volume was distributed into the second wave and the fifth highest was distributed into the final of the six waves. The sixth and smallest wave volume was distributed into the first wave of the series. The shape of each wave is identical and the magnitude is determined by the total volume that the wave must convey. The process of converting flow frequency curves into the synthetic series of 30-day hydrographs is depicted on **Plate 3**.

There are several reasons for using a 30-day duration for the synthetic flood hydrographs. The Sacramento River watershed is so large that 5 days is not long enough for a flood wave to travel from the most distant headwater down to the mouth of the Sacramento River. The multi-wave flood hydrograph includes the smaller antecedent waves from storms that prime the watershed for the highest wave. Also, the multi-wave hydrograph is needed to (1) provide the extra flood volume needed to simulate reservoir operation during an extended period of wet weather, and (2) fill the floodplains with enough flood volume to run levee failure scenarios.

Figure 1 shows an example of the 30-day hydrograph with the 5-day waves, for unregulated and regulated conditions. The figure shows the 1 percent exceedence hydrographs, for unregulated and regulated conditions, for the Sacramento River at the confluence with the Feather River, for the Sacramento Mainstem Centering. The hydrograph for unregulated conditions is not a true representation of the hydrograph with six 5-day waves; it is the result from routed contributions of upstream tributaries. See **Figure 2** for an example of a tributary hydrograph with six 5-day waves – the Comp Study hydrograph for Folsom Lake inflow.

Figure 1

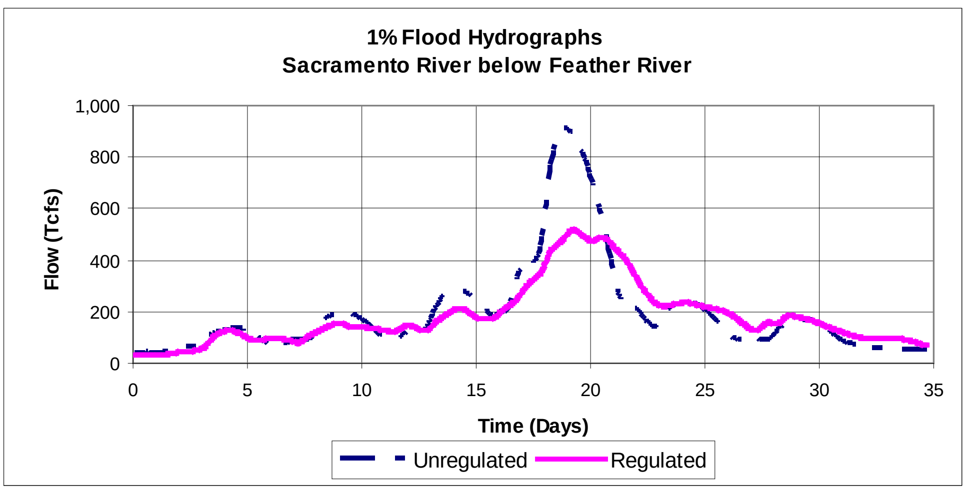
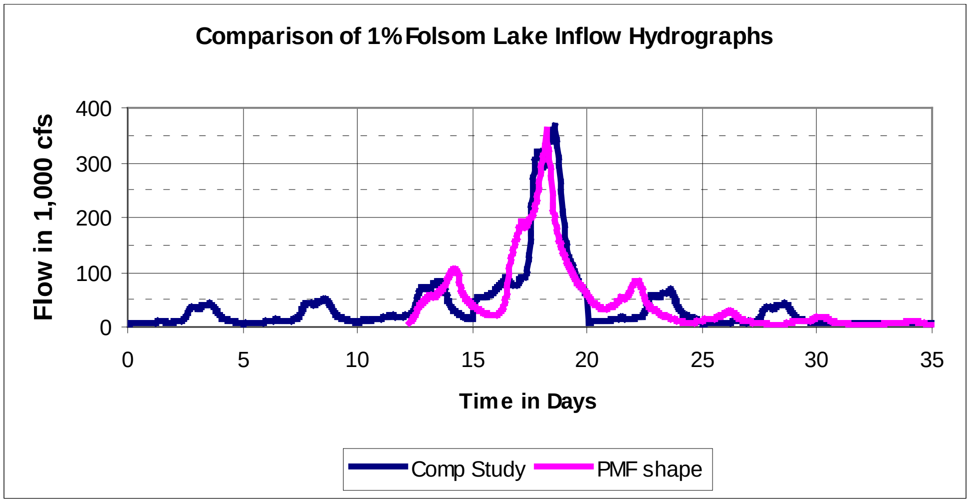


Figure 2



1.2 Shanghai Bend-Yuba River Centering. This flood centering, with a specific centering on the Yuba River and slightly more frequent concurrent event on the Feather River above Oroville, produces the maximum inundation areas along the lower reaches of the Feather and Yuba rivers. It also produces the maximum inundation area at Verona, near the confluence of the Feather River with the Sacramento River. This flood centering was not developed as part of the original Comp Study, but the Comp Study methodology described in **Reference 1** was used to develop the storm centering and flood hydrographs, which were routed through the reservoir system. **Reference 2**, the "Yuba River Basin Project General Reevaluation Report," App. A, Synthetic Hydrology and Reservoir Operations Technical Documentation, dated August 2004, corrected June 2008, documents the hydrology and modeling efforts conducted for the Feather and Yuba rivers using the Comp Study methodology. **Table 2** shows the flood patterns for the Shanghai Bend-Yuba River centering. The American River flood hydrographs are different from those used in the Comp Study. See **Section 1.3** for an explanation of the changes made.

Table 2
Feather River above Shanghai Bend Synthetic Flood Centering A
With a Specific Centering on the Yuba River

Index Point	Percent Chance Exceedence						
	50%	10%	4%	2%	1%	0.50%	0.20%
Sacramento River at Shasta	101.01	20.20	8.08	5.77	2.89	1.44	0.58
Clear Cr. at Whiskeytown	344.83	68.97	27.59	19.70	9.85	4.93	1.97
Cow Cr. near Millville	196.08	39.22	15.69	11.20	5.60	2.80	1.12
Cottonwood Cr. near Cottonwood	344.83	68.97	27.59	19.70	9.85	4.93	1.97
Battle Cr. Below Coleman FH	196.08	39.22	15.69	11.20	5.60	2.80	1.12
Mill Cr. near Los Molinos	76.34	15.27	6.11	4.36	2.18	1.09	0.44
Elder Cr. near Paskenta	140.85	28.17	11.27	8.05	4.02	2.01	0.80
Thomes Cr. at Paskenta	140.85	28.17	11.27	8.05	4.02	2.01	0.80
Deer Cr. near Vina	76.34	15.27	6.11	4.36	2.18	1.09	0.44
Big Chico Cr. near Chico	76.34	15.27	6.11	4.36	2.18	1.09	0.44
Stony Cr. at Black Butte	140.85	28.17	11.27	8.05	4.02	2.01	0.80
Butte Cr. near Chico	76.34	15.27	6.11	4.36	3.18	1.09	0.44
Feather River at Oroville	54.95	10.87	4.35	2.17	1.06	0.53	0.21
Yuba R. at New Bullards Bar	50.00	10.00	4.00	2.00	1.00	0.5	0.20
Yuba R. at Englebright	50.00	10.00	4.00	2.00	1.00	0.5	0.20
Deer Cr. near Smartsville	125.00	25.00	10.00	5.00	2.50	1.25	0.50
Bear River near Wheatland	125.00	25.00	10.00	5.00	2.50	1.25	0.50
Cache Cr. at Clear Lake	153.85	30.77	12.31	6.15	3.08	1.54	0.62
N.F. Cache Cr. at Indian Vy.	153.85	30.77	12.31	6.15	3.08	1.54	0.62
American River at Folsom	76.34	15.27	6.11	3.05	1.53	0.76	0.31
Putah Cr. at Berryessa	153.85	30.77	12.31	6.15	3.08	1.54	0.62

1.3 American River Centering. The flood patterns for the American River specific tributary centering are shown on **Table 3**. The concurrent flood hydrographs for this centering were developed using the Comp Study methodology and hydrograph shapes, based on the January 1997 New Years flood event. However, the American River specific flood hydrographs were developed using a different shape and different volumes**.** For consistency with the ongoing American River Watershed Study, the Folsom Dam inflow hydrograph shape used for the American River Common Features GRR is based upon the Probable Maximum Flood (PMF) for Folsom Dam. Use of this PMF-shape flood hydrograph predates the Comp Study. Development of the revised Folsom Dam PMF is discussed in **Reference 3**, “Folsom Dam and Lake Revised PMF Study,” American River Basin, California, Hydrology Office Report, dated October 2001. The PMF was computed using the most recent Probable Maximum Precipitation criteria, presented in **Reference 4**, “Hydrometeorological Report No. 59, Probable Maximum Precipitation for California,” U.S. Dept. of Commerce, NOAA, U.S. Dept of the Army Corps of Engineers, Feb 1999).

Table 3
American River Tributary Synthetic Flood Centering

Index Point	Percent Chance Exceedence						
	50%	10%	4%	2%	1%	0.50%	0.20%
Sacramento River at Shasta	250.00	50.00	20.00	10.00	5.00	2.50	1.00
Clear Cr. at Whiskeytown	555.56	111.11	44.44	22.22	11.11	5.56	2.22
Cow Cr. near Millville	178.57	35.71	14.29	7.14	3.57	1.79	0.71
Cottonwood Cr. near Cottonwood	555.56	111.11	44.44	22.22	11.11	5.56	2.22
Battle Cr. below Coleman FH	178.57	35.71	14.29	7.14	3.57	1.79	0.71
Mill Cr. near Los Molinos	121.95	24.39	9.76	4.88	2.44	1.22	0.49
Elder Cr. near Paskenta	138.89	27.78	11.11	5.56	2.78	1.39	0.56
Thomes Cr. at Paskenta	138.89	27.78	11.11	5.56	2.78	1.39	0.56
Deer Cr. near Vina	121.95	24.39	9.76	4.88	2.44	1.22	0.49
Big Chico Cr. near Chico	138.89	27.78	11.11	5.56	2.78	1.39	0.56
Stony Cr. at Black Butte	121.95	24.39	9.76	4.88	2.44	1.22	0.49
Butte Cr. near Chico	138.89	27.78	11.11	5.56	2.78	1.39	0.56
Feather River at Oroville	92.59	18.52	7.41	3.7	1.85	0.93	0.37
Yuba R. at New Bullards Bar	69.44	13.89	5.56	2.78	1.39	0.69	0.28
Yuba R. at Englebright	69.44	13.89	5.56	2.78	1.39	0.69	0.28
Deer Cr. near Smartsville	116.28	23.26	9.30	4.65	2.33	1.16	0.47
Bear River near Wheatland	116.28	23.26	9.30	4.65	2.33	1.16	0.47
Cache Cr. at Clear Lake	192.31	38.46	15.38	7.69	3.85	1.92	0.77
N.F. Cache Cr. at Indian Vy.	192.31	38.46	15.38	7.69	3.85	1.92	0.77
American River at Folsom	50.00	10.00	4.00	2.00	1.00	0.50	0.20
Putah Cr. at Berryessa	192.31	38.46	15.38	7.69	3.85	1.92	0.77

Also, the American River Watershed Study unregulated flow frequency curves for the American River were revised when the period of record was updated through 2004. See **Reference 5**, “Rain Flood Flow Frequency Analysis, American River California,” Office Report, U.S. Army Corps of Engineers, Sacramento District, dated August 2004. Revision of the flood frequency curves changed the flood volumes used for the American River hydrographs for the 8-Flood Series. **Figure 2** is a graphical presentation of the flood inflow hydrographs to Folsom Lake, comparing the Comp Study 1 percent flood with the PMF-shape 1 percent flood. The graph presents the maximum 72-hour period as coincident for the two flood hydrographs for days 17 through 19.

Because the PMF-shape hydrographs for the Folsom Lake inflow are different from the Comp Study hydrographs, a volume comparison was made between the hydrographs for various exceedence events. This comparison was made to ensure that use of the PMF-shape hydrographs would not cause problems and inconsistencies. **Table 4** presents a volume comparison between the two different hydrograph shapes for the American River flood series above Folsom Dam. The table shows that the differences in volume are minor.

Table 4
Hydrograph Volume Comparison for
Inflow Hydrographs to Folsom Lake

% Event Flood	1-Day Volume (in day cfs)	3-Day Volume (in day cfs)	7-Day Volume (in day cfs)
10% (PMF Shape) 10% (Comprehensive Study) % Difference	101,000 113,000 12%	71,000 70,000 -1%	43,000 46,000 7%
4% (PMF Shape) 4% (Comprehensive Study) % Difference	156,000 174,000 10%	110,000 108,000 -2%	66,000 67,000 1%
2% (PMF Shape) 2% (Comprehensive Study) % Difference	207,000 229,000 10%	145,000 142,000 -2%	87,000 86,000 -1%
1% (PMF Shape) 1% (Comprehensive Study) % Difference	266,000 292,000 9%	187,000 181,000 -3%	112,000 107,000 -5%
0.5% (PMF Shape) 0.5% (Comprehensive Study) % Difference	334,000 363,000 8%	235,000 226,000 -4%	141,000 131,000 -8%
0.2% (PMF Shape) 0.2% (Comprehensive Study) % Difference	440,000 475,000 7%	309,000 300,000 -3%	185,000 169,000 -9%
The flow comparison is presented in Table 4 in "% Difference", which shows how much the Comprehensive Study hydrograph volume differs from the PMF shape hydrograph volume. Hydrographs are for unregulated inflow conditions.			

The PMF-shape hydrographs were routed through Folsom Dam for three without-project alternatives. In preparation for routing the PMF-shape hydrographs through Folsom Dam, the maximum 72-hour period of the PMF-shape was lined up to occur at the same time as the Comp Study American River hydrograph. See **Figure 2** above. For the PMF-shape hydrographs, the maximum 3-day flow occurs closer to the beginning of the hydrograph. As a result, outflow from Folsom Dam for the PMF-shape hydrographs does not begin until 6 p.m. of day 12 after the start of the Comp Study hydrographs for the other Sacramento River tributaries. A constant flow of 2,000 cfs was used for outflow from Folsom Dam for days 1 through 6pm of day 12 for the PMF shape flood hydrographs.

2.0 Development of Historical Flood Hydrographs for Natomas Tributaries

Historical flow hydrographs for the Natomas tributaries were developed as upstream boundary conditions on the Natomas Cross Canal and Steelhead Creek (also known as Natomas East Main Drainage Canal), for testing of the hydraulic model. The upstream boundary locations for the Natomas tributaries are shown on **Plate 2.** Six large historical flood events were chosen for which Natomas tributary flood hydrographs would be developed. The six flood events are 15 - 19 February 1986, 8 - 12 January 1995, 29 December 1996 - 3 January 1997, 22 - 26 January 1997, 2 - 6 February 1998, and 30 December 2005 - 3 January 2006. The selection of flood events was based on the amount of available precipitation data and whether any flow data, either a hydrograph or mean day flow, were available for the Dry Creek at Roseville gaging station. Hydrographs for the six floods on the Sacramento, Feather, and American rivers were available for use in the hydraulic model. The effect of any additional contribution from the Natomas tributaries could then be tested in the model. Also, from the frequency analysis presented in the Natomas General Reevaluation Report Hydrology Appendix (**Reference 6**), frequencies could be assigned to these flood events for the Natomas tributaries, which could then be compared with the magnitudes of these events on the mainstem Sacramento and American rivers for the Coincident Frequency Analysis.

This chapter discusses the computation of historical flood hydrographs first for the Steelhead Creek tributaries and then for the Natomas Cross-Canal tributaries. The historical flood hydrographs were easier to develop for Steelhead Creek because calibrated HEC-1 models had been developed in previous studies for the tributaries, an extensive network of precipitation gages covers the watershed, and hydrographs or mean day flows exist for the six flood events for the Dry Creek at Roseville gage. A mean day flow record is available for four of the six floods at the Arcade Creek near Del Paso Heights gage. **Table 5** shows what flow data are available for which storm events. Station locations are shown on **Plate 1**.

Table 5
Available Flow Data for 6 Historical Flood Events

Stream---> Gage Location--->	Dry Cr Royer Park	Dry Cr Vernon St.	Magpie Cr Del Paso Hghts	Arcade Cr Del Paso Hghts
CDEC Code or USGS Number	CDEC RYP	CDEC VRS	USGS 11447330	CDEC ACK
FLOOD EVENT	D.A. (sq.mi.) 58.63*	D.A. (sq.mi.) 77.75*	D.A. (sq.mi.) 2.30*	D.A. (sq.mi.) 31.83*
15-19 February 1986	N/A	Hydrograph	N/A	N/A
8-12 January 1995	N/A	Hydrograph	N/A	N/A
29 Dec 96 - 3 Jan 97	N/A	Mean Day	Mean Day	Mean Day
22-26 January 1997	N/A	Mean Day	Mean Day	Mean Day
2-6 February 1998	N/A	Mean Day	N/A	Mean Day
30 Dec 05 - 3 Jan 06	hydrograph	Hydrograph	N/A	Mean Day

N/A = Not Available

* = drainage area in HEC-1 model, not drainage area associated with DWR or USGS gage

Some of the precipitation gages used for the December 2005 storm isohyetal map were not available for the earlier flood events. These are mostly the stations on the Wunderground Web site and are not included in **Table 6**. **Table 6** below lists the National Climatic Data Center (NCDC) stations and California Data Exchange Center (CDEC) stations used to develop the storm isohyetal maps for one or more of the six historical flood events. **Table 6** also lists the station precipitation amounts for the 6 storms. **Plate 4** shows the locations of the precipitation gages listed in **Table 6** and the streamflow gages listed in **Table 5**.

Table 6

Precipitation Gages - Storm Totals for 6 Historical Storm Events

STATION	DATA SOURCE	CDEC STATION CODE	STORM EVENT AND PRECIPITATION (INCHES)					
			1986 15-19 FEB	1995 8-12 JAN	1996 - 97 29 DEC - 2 JAN	1997 22-26 JAN	1998 2-6 FEB	2005 - 06 30 DEC - 3 JAN
Arcade Cr-Winding Way	CDEC	AMC	N/A	N/A	** 3.93	** 6.34	** 5.79	** 4.93
Arden	CDEC	ARW	** 9.09	5.74	** 3.34	** 5.59	** 5.00	4.49
Auburn	NCDC	---	12.83	8.96	7.28	7.95	5.70	N/A
Auburn Dam Ridge	CDEC	ADR	N/A	N/A	** 6.93	** 7.84	** 5.55	4.60
CSUS	CDEC	CSU	N/A	N/A	N/A	N/A	N/A	4.80
Camp Far West	CDEC	CFW	N/A	N/A	N/A	N/A	N/A	4.63
Caperton Reservoir	CDEC	CPR	N/A	N/A	** 4.65	** 5.67	** 5.63	** 4.64
Chicago	CDEC	CHG	** 7.96	N/A	3.82	5.75	2.68	4.69
Cresta Park	CDEC	CRP	9.37	N/A	3.86	6.50	4.88	4.49
Englebright Dam	CDEC	ENG	N/A	5.48	6.20	6.56	4.83	N/A
Folsom Dam	CDEC	FLD	9.53	N/A	2.13	3.58	3.03	4.72
Folsom WTP	CDEC	FWP	N/A	N/A	N/A	N/A	5.94	N/A
Grass Valley #2	NCDC	---	** 14.9	9.51	14.73	10.77	8.69	N/A
Grass Valley	CDEC	GVY	N/A	N/A	N/A	N/A	N/A	10.72
Hurley	CDEC	HUR	N/A	N/A	2.78	3.56	3.91	4.55
Lincoln	CDEC	LCN	N/A	** 5.19	N/A	3.46	** 5.15	4.34
Loomis Observatory	CDEC	LMO	N/A	N/A	3.74	6.38	4.89	3.89
Navion	CDEC	NVN	** 9.54	N/A	N/A	6.07	5.94	N/A
Newcastle-Pineview Sch.	CDEC	NCS	N/A	N/A	** 4.96	** 6.74	** 5.94	4.93
Orangevale	CDEC	ORN	** 6.67	N/A	3.94	5.67	6.26	4.85
Rancho Cordova	CDEC	RNC	7.76	N/A	3.54	5.50	5.24	4.61
Represa	NCDC	---	7.03	5.24	3.52	4.47	4.53	3.89
Rio Linda	CDEC	RLN	** 7.28	N/A	** 2.92	** 4.77	** 5.32	** 3.90
Roseville City Hall	#	---	9.34	N/A	N/A	N/A	N/A	N/A
Roseville Fire Stn	CDEC	RSV	N/A	N/A	3.62	** 5.63	N/A	3.76
Roseville WTP	CDEC	RTP	** 8.76	N/A	** 4.30	** 6.30	** 5.95	** 5.01
Royer Park	CDEC	RYP	N/A	N/A	** 3.86	** 6.50	** 6.10	** 4.08
Sac Exec AP	NCDC	---	6.72	5.11	2.79	5.65	4.69	4.70
Sac Metro AP	CDEC	SMF	N/A	4.30	5.51	5.74	3.70	3.56
Sacramento 5 ESE	NOAA	---	7.68	5.89	2.22	4.71	4.54	5.02
Sacramento City	#	---	8.12	N/A	N/A	N/A	N/A	N/A
Sacramento Post Office	CDEC	SPO	N/A	5.89	2.46	4.75	4.60	N/A
Sierra College	#	---	9.05	N/A	N/A	N/A	N/A	N/A
Sunrise Blvd	#	---	6.82	N/A	N/A	N/A	N/A	N/A
Van Maren	CDEC	VNM	** 8.90	N/A	** 3.98	** 5.95	** 5.98	N/A
Wheatland 2NE	NCDC	---	4.90	4.40	N/A	N/A	N/A	N/A

N/A = Not Available or Missing Record

** = Recording Rain Gage pattern used to distribute this storm in HEC-1 Model

= Data from Dry Creek Basin Hydrology Report dated April 1988

2.1 Steelhead Creek Historical Flood Hydrographs.

a. December 2005 Flood. The December 2005 – January 2006 rainflood event was used to validate the HEC-1 models for Dry and Arcade creeks in **Reference 6**, the Natomas GRR Hydrology Appendix, dated October 2006. **Plate 5** shows the December 2005 – January 2006 storm isohyetal map, and **Figure 3** shows the comparison between the observed and computed hydrographs for Dry Creek at Vernon Street. The HEC-1 model was used to compute flood hydrographs at the streamgage locations, route the flows down to the downstream index locations, add the local flow above Steelhead Creek, and compute flood hydrographs for Upper NEMDC and Old Magpie Creek above and below their respective pumping stations. The computed flood hydrographs for Dry Creek at Steelhead Creek, Arcade Creek at Steelhead Creek, Upper NEMDC above and below the NEMDC Stormwater Pumping Station, and Old Magpie Creek above and below Pump 157, were provided to Hydraulic Design Section as historical flood input for this flood event. The pumping station locations are shown on **Plate 1**.

Figure 3 presents the flood hydrograph from the HEC-1 run for Dry Creek at Roseville compared with the observed hydrograph. **Table 7** presents a comparison for the peak, and 1-, 3-, and 5-day volumes between the computed hydrographs and the observed hydrographs for the Dry Creek and Arcade Creek gaging stations.

Figure 3

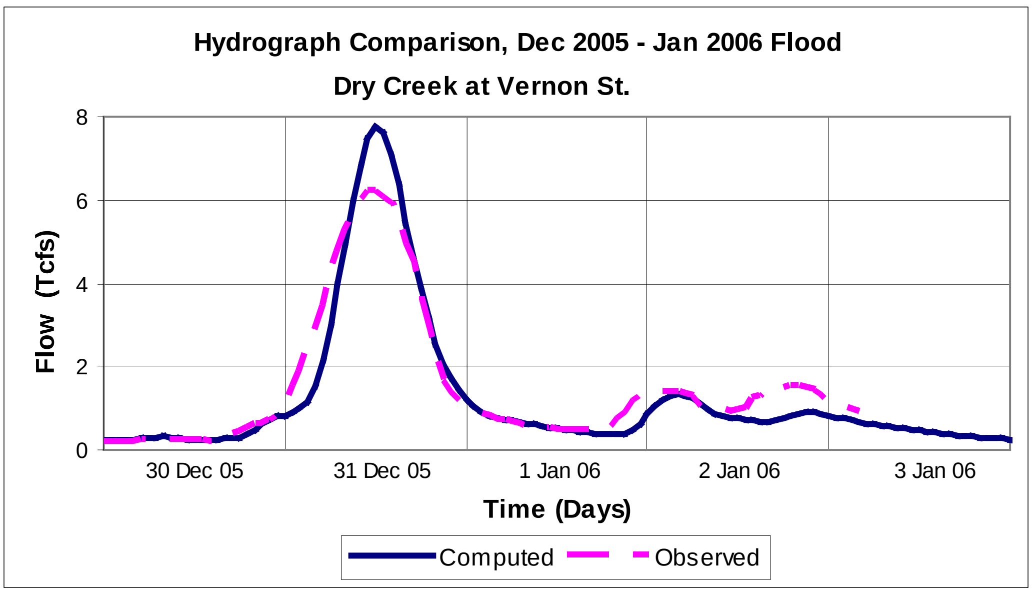


Table 7
30 December 2005 - 3 January 2006 Flood Volume Comparison
For Three Steelhead Creek Tributary Streamflow Gaging Stations

Hydrograph	Peak (cfs)	1-Day Vol. (avg cfs)	3-Day Vol. (avg cfs)	5-Day Vol. (avg cfs)
Dry Creek at Royer Park				
Observed Hydrograph	5,240	3,040	1,620	------
2006 HEC-1 Run	6,230	2,870	1,330	916
% Difference	18.9%	-5.6%	-17.9%	------
Dry Creek at Vernon St.				
Observed Hydrograph	6,250	3,820	1,930	1,424
2006 HEC-1 Run	7,760	3,920	1,810	1,252
% Difference	24.2%	2.6%	-6.2%	-12.1%
Arcade Cr. near Del Paso Heights				
Observed Hydrograph	3,460	1,900	835	536
2006 HEC-1 Run	3,240	1,870	846	561
% Difference	-6.4%	-1.6%	1.3%	4.6%

b. February 1986 Flood**.** According to **Reference 7,** Dry Creek, Placer and Sacramento Counties, California, Hydrology Office Report, revised April 1988, runoff from a large storm event like that of February 1986, can only be estimated, due to a lack of adequate streamflow data. The Dry Creek gage does not function correctly for flows above 2,000 cfs. Peak flows above that are estimated using highwater marks and slope-area measurements by the State of California. The peak flow of 13,100 cfs and associated one-day flow of 5,800 cfs listed in **Reference 7** for the February 1986 flood for Dry Creek at the Vernon Street gage are based upon a flood reconstitution, using the HEC-1 model and rainfall recording data. The flood reconstitution HEC-1 run could not be located, but available data included the reconstituted flood hydrograph for Dry Creek at Roseville, 5-day storm totals, and rainfall recording data for several stations.

Plate 6 shows the isohyetal map created for the 15 - 19 February 1986 storm, based on the station precipitation totals listed on **Table 6**. **Plate 6** may not necessarily be an accurate isohyetal map of the storm, but it shows approximate isolines of the 5-day storm amounts used in the HEC-1 model to develop the flood hydrographs for the Natomas tributaries. Eight precipitation gages used for storm distribution patterns are identified with "**" in the February 1986 rainfall column of **Table 6**. For subbasins above the Dry Creek at Roseville gage, the base flow parameters in the HEC-1 model are:

STARTQ = 9 cfs/sq.mi.
QRCSN = -0.1
RTIOR = 1.05

No base flow was used for the lower elevation subbasins in the Steelhead Creek watershed. Loss rates used were zero initial loss and 0.10 inch per hour constant loss. The watershed was wet from three days of rain prior to 15 February, the start of the maximum five-day flow.

The HEC-1 model was run to develop flood hydrographs for this storm for the four tributaries to Steelhead Creek. **Figure 4** presents the flood hydrograph from the HEC-1 run for Dry Creek at Roseville compared with the previously reconstituted flood hydrograph from **Reference 7**. **Table 8** presents a comparison for the peak, and 1-, 3-, and 5-day volumes for the two hydrographs.

Figure 4

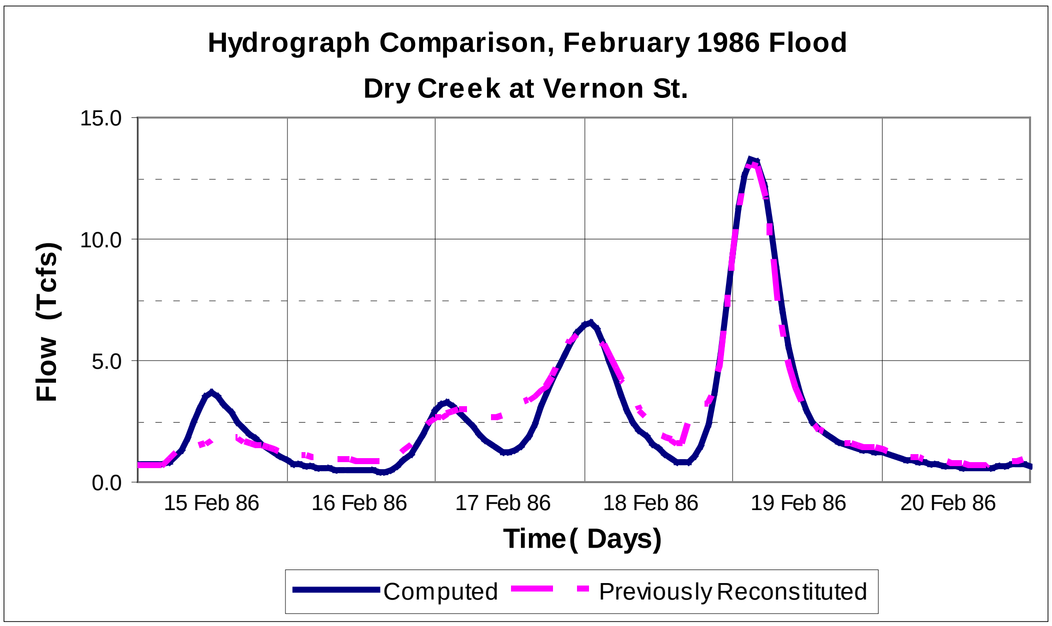


Table 8
15 – 19 February 1986 Flood Volume Comparison
Dry Creek at Roseville Gage

Hydrograph	Peak (cfs)	1-Day Vol. (avg cfs)	3-Day Vol. (avg cfs)	5-Day Vol. (avg cfs)
Ref 7 Hydrograph (1988)	13,100	5,930	4,160	2,980
2008 HEC-1 Run	13,000	5,980	3,810	2,850
% Difference	-0.8%	0.8%	-8.4%	-4.4%

c. <u>January 1995 Flood</u>. The 8 - 12 January 1995 storm had a very intense 6-hour period of rainfall the evening of 9 January that produced the peak flow of record on Dry Creek. **Reference 8**, "Use of Radar-Rainfall Estimates to Model the January 9 - 10, 1995 Floods in Sacramento, CA," paper presented October 1995, explains how data from a network of rain gages were combined with radar-rainfall estimates from the National Weather Service WSR-88D radar observations to reconstitute the flood hydrograph for Dry Creek at Roseville and estimate flood hydrographs for other locations in the watershed. The HEC-1 model used a 5-minute time increment for one hundred small subbasins above the Dry Creek at Roseville gage for a 3-day hydrograph. Each subbasin or small group of subbasins had its own rainfall distribution pattern.

The Natomas GRR study is more concerned with 5-day volumes than those of shorter duration, so the rainfall period was extended back one day, to include 8 January. The Natomas GRR HEC-1 model listed in **Reference 6,** Attachment 1 was used instead of the 5-minute HEC-1 model described in **Reference 8**. The **Reference 6** model has 28 subbasins above the Dry Creek at Roseville gage instead of the 100 subbasins in the **Reference 8** model. The nearly one hundred 5-minute rainfall distribution patterns in the **Reference 8** HEC-1 model were reduced to eight patterns to distribute the January 1995 storm for the Natomas GRR HEC-1 model. The 5-minute rainfall distribution patterns were converted to hourly increments, and extended back to 8 January using the CDEC rainfall gage for Lincoln (LCN). **Plate 7** is not an accurate isohyetal map of the storm, but it shows approximate isolines of the 5-day storm amounts used in the HEC-1 model to develop the flood hydrographs for the Natomas tributaries. The isolines were based on the station precipitation totals listed on **Table 6** and subbasin storm totals in the **Reference 8** HEC-1 model. Very little rain fell on 11-12 January. The HEC-1 model for this American River GRR study was run for a 5-day time period. For subbasins above the Dry Creek at Roseville gage, the base flow parameters in the HEC-1 model are:

STARTQ = 3 cfs/sq.mi.
QRCSN = -0.1
RTIOR = 1.10

No base flow was used for the rest of the Steelhead Creek watershed. Loss rates used were zero initial loss and 0.10 inch per hour constant loss.

The HEC-1 model was run to develop flood hydrographs for this storm for the four tributaries to Steelhead Creek. **Figure 5** presents the flood hydrograph from the HEC-1 run for Dry Creek at Roseville compared with the observed flood hydrograph shown on Figure 12 of **Reference 8**, the radar-rainfall report. The rainfall distribution patterns used in the HEC-1 model produced a hydrograph with two peaks flows, not one. The higher peak is still similar in magnitude and timing to the observed peak, and the three-day volumes are nearly the same. **Table 9** presents a comparison for the peak, and 1-, and 3-day volumes for the two hydrographs. The computed Dry Creek hydrograph has only a single peak by the time it is routed down to Steelhead Creek and added to the local flow.

Figure 5

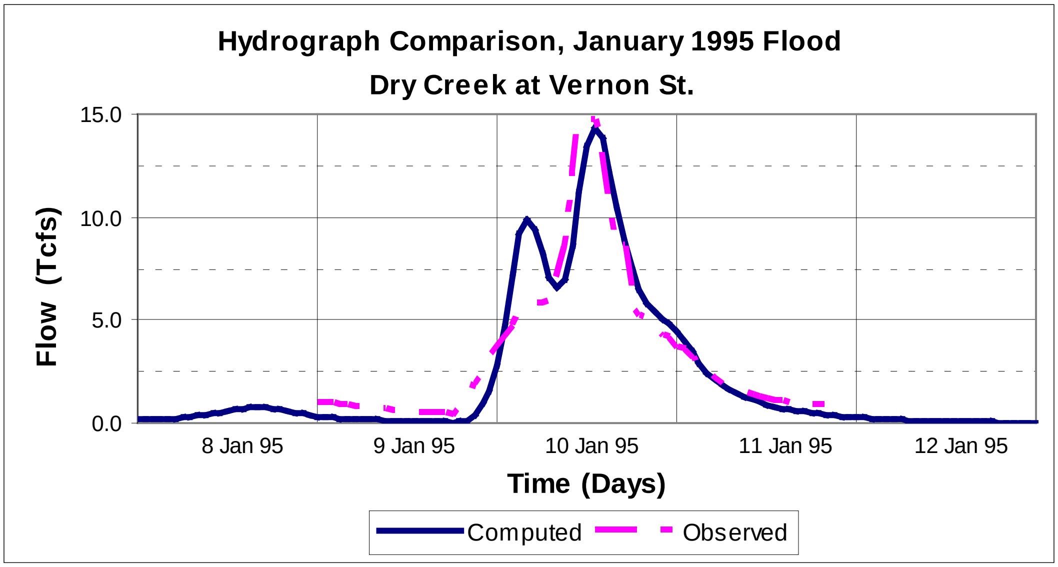


Table 9
8 – 12 January 1995 Flood Hydrograph Comparison
Dry Creek at Roseville Gage

Hydrograph	Peak (cfs)	1-Day Vol. (avg cfs)	3-Day Vol. (avg cfs)	5-Day Vol. (avg cfs)
Observed Hydrograph	14,800	7,580	3,380	------
2008 HEC-1 Run	14,400	8,390	3,360	2,120
% Difference	-2.7%	10.7%	-0.6%	------

d. <u>29 Dec 1996 – 3 Jan 1997 Flood</u>. Recording rainfall data for numerous stations were available on the CDEC website for January 1997. **Table 6** lists the storm totals for these and the daily rainfall stations. The 5-day storm period for the 1997 New Years storm is from 29 December 1996 to 2 January 1997. An isohyetal map was created, based on the storm amounts for this time period, shown on **Table 6**, and subbasin storm amounts were estimated for the HEC-1 model. Nine precipitation stations, identified with "**" in the Dec '96 – Jan '97 rainfall column of **Table 6**, were used as rainfall distribution patterns in the HEC-1 model. For subbasins above the Dry Creek at Roseville gage, the base flow parameters in the HEC-1 model are:

STARTQ = 3 cfs/sq.mi.
QRCSN = -0.1
RTIOR = 1.05

No base flow was used for the rest of the Steelhead Creek watershed. Loss rates used were zero initial loss and 0.10 inch per hour constant loss.

The HEC-1 model was run to develop flood hydrographs for this storm for the four tributaries to Steelhead Creek. These hydrographs are of greater importance than merely as reconstituted hydrographs for this flood event. The shapes of these computed hydrographs for the 5-day period 30 Dec 1996 to 3 Jan 1997 are used as the 5-day pattern hydrographs in the Coincident Frequency Analysis. The 5-day flood hydrograph patterns used in the Comprehensive Study as Sacramento River tributary input hydrographs, prior to their re-distribution to the upstream reservoirs for the Comp Study reservoir operations modeling, are either the observed or computed unregulated tributary hydrographs for that 5-day period, 30 Dec 1996 to 3 Jan 1997. With all the tributary hydrographs for the same 5-day period, timing for high flows on the Natomas tributaries should historically match their actual timing with respect to timing of the other streams, including the Sacramento River at Verona flood hydrograph for the New Year 1997 flood event.

The observed flows for this flood event at the stream gages on Dry and Arcade creeks and the flood hydrographs routed to the downstream index points showed the flood to be a 30 percent chance or more frequent event for Natomas, compared with the large, low frequency flows occurring on many other Sacramento River tributaries. It would be difficult to justify basing the shapes of floods up to the 0.2 percent event upon a 30 percent chance event, so the HEC-1 model was revised. The observed storm amounts were raised by between 15 and 45 percent, to compute a somewhat rarer flood event, on which to base the synthetic flood hydrographs. With enhanced rainfall and higher runoff, the 8-Flood Series flood patterns are based on a 15 percent chance 5-day flood event. Exceedence estimates of the 5-day volumes for the six historic floods are discussed in **Section 2.1.g**. **Plate 8** shows the revised isohyetal map with the higher rainfall amounts used to develop subbasin storm totals in the HEC-1 model to develop Natomas tributary flood hydrographs

Figure 6 presents the flood hydrograph from the HEC-1 run with the increased rainfall for Dry Creek at Roseville compared with the observed mean day flow hydrograph for the Vernon Street gage. **Figure 7** presents the flood hydrograph from the HEC-1 run for Arcade Creek near Del Paso Heights USGS gage compared with the observed mean day flow hydrograph for the gage. The bars on **Figures 5 and 6** represent the observed peak flows for Dry and Arcade creeks at their respective gaging stations. **Table 10** presents a comparison for the peak, and 1-, and 3-day volumes between the computed hydrograph and the mean day flow hydrograph published for the gage. The 5-day period, 30 December 1996 to 3 January 1997, is the period for which the computed 5-day hydrographs for Dry and Arcade creeks at their confluences with Steelhead Creek and Upper NEMDC and Old Magpie Creek above their respective pumping stations are the pattern hydrographs used for the 8-Flood synthetic series.

Figure 6

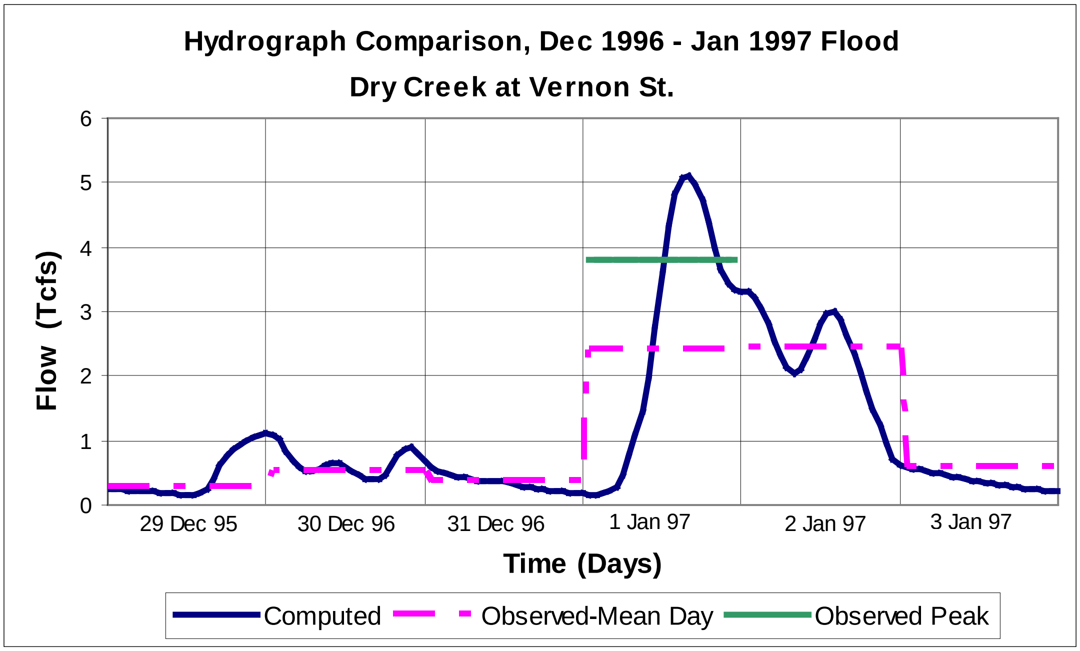


Figure 7

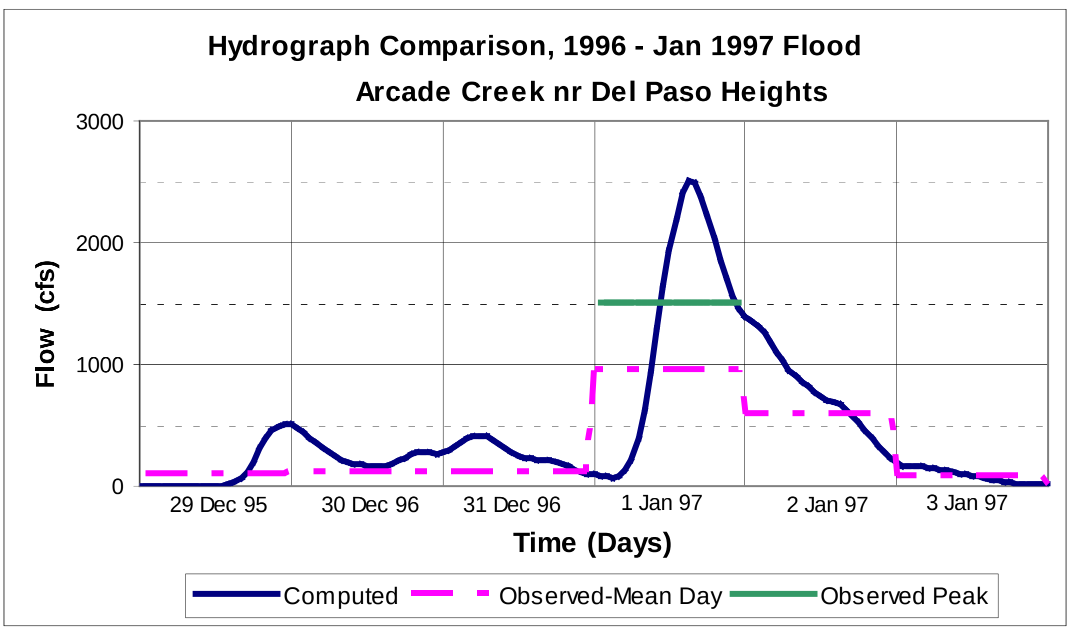


Table 10

29 December 1996 – 3 January 1997 Flood Volume Comparison

For Three Steelhead Creek Tributary Streamflow Gaging Stations

Hydrograph	Peak (cfs)	1-Day Vol. (avg cfs)	3-Day Vol. (avg cfs)	5-Day Vol. (avg cfs)
Dry Creek at Vernon St.				
Observed Hydrograph	3,800	2,440	1,810	1,262
2008 HEC-1 Run	5,120	3,470	1,770	1,303
% Difference	34.7%	42.2%	-2.2%	3.3%
Magpie Cr. near Del Paso Heights				
Observed Hydrograph	N/A	81	35	25
2008 HEC-1 Run	320	108	47	31
% Difference	------	33.3%	35.6%	22.0%
Arcade Cr. near Del Paso Heights				
Observed Hydrograph	1,510	945	551	373
2008 HEC-1 Run	2,507	1,630	778	558
% Difference	66.0%	72.5%	41.2%	49.5%

e. Mid-January 1997 Flood. The mid-January 1997 flood was not an especially rare flood event for the higher elevation tributaries to the Sacramento River. However, for the Natomas tributaries, the mid-January rainfall was greater than for the New Year 1997 storm a few weeks earlier. The greater mid-January rainfall is reflected in the higher peak flows and runoff volumes for this event on the Natomas tributaries. Compare the difference between the Dry Creek hydrographs shown on **Figure 6** and **Figure 8**. The peak flow on Arcade Creek was 150 percent of the peak flow there three weeks earlier. The rainfall from **Table 6** for the 22-26 January 1997 storm was used to develop a storm isohyetal map for the HEC-1 model. **Plate 9** may not necessarily be an accurate isohyetal map of the storm, but it shows approximate isolines of the 5-day storm amounts used in the HEC-1 model to develop the flood hydrographs for the Natomas tributaries. The observed mean day flood hydrographs for Vernon Street, Magpie Creek and Arcade Creek near Del Paso Heights were used as the observed hydrographs for the comparison between observed and computed flood hydrographs in **Table 11**. Ten precipitation stations, identified with "**" in the 22-26 January 1997 rainfall column of **Table 6**, were used as storm distribution patterns. For subbasins above the Dry Creek at Roseville gage, the base flow parameters in the HEC-1 model are:

STARTQ = 3 cfs/sq.mi.
QRCSN = -0.1
RTIOR = 1.05

No base flow was used for the rest of the Steelhead Creek watershed. Loss rates used were zero initial loss and 0.10 inch per hour constant loss.

The HEC-1 model was run to develop flood hydrographs for this storm for the four tributaries to Steelhead Creek. **Figure 8** presents the flood hydrograph from the HEC-1 run for Dry Creek at Roseville compared with the mean day hydrograph observed for the Vernon Street gage. Timing of the observed peak flows of 7,950 cfs and 7,250 cfs is based on the time that the highest stages occurred. The computed peak flows are not the same as the observed peak flows, but the observed peak flows are only one hour earlier than the computed peak flows, which is better timing than for the New Year 1997 flood hydrograph reproduction. There is not much difference between the computed and the observed 5-day flood volumes for Dry Creek. **Table 11** presents a comparison for the peak, and 1-, 3-, and 5-day volumes for the three gaging stations.

Figure 8

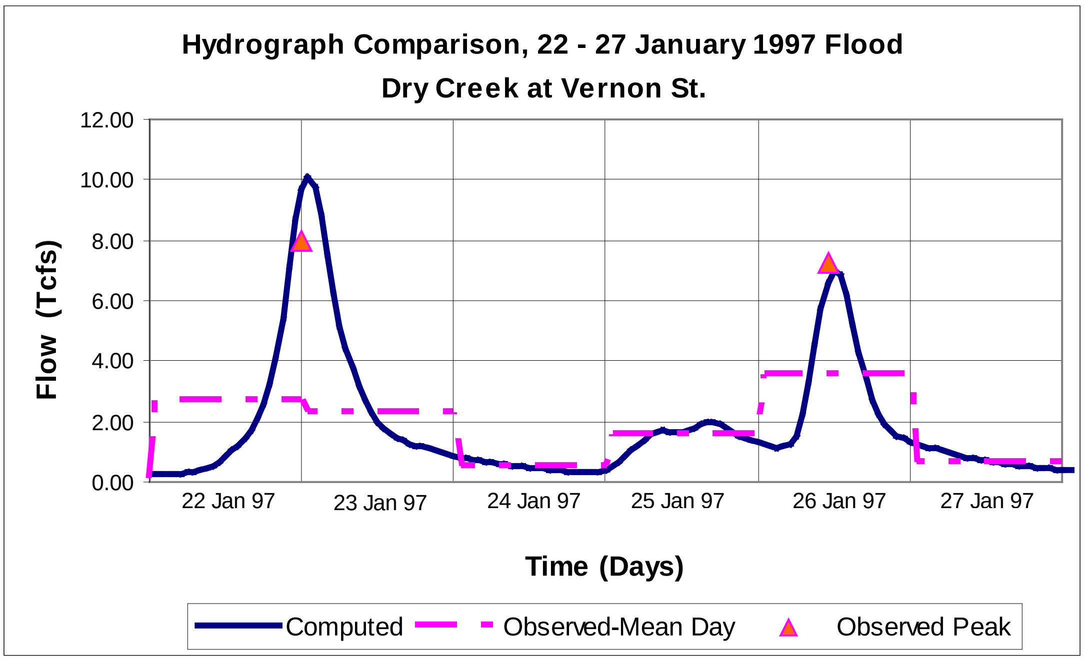


Table 11

22 - 26 January 1997 Flood Volume Comparison

For Three NEMDC Tributary Streamflow Gaging Stations

Hydrograph	Peak (cfs)	1-Day Vol. (avg cfs)	3-Day Vol. (avg cfs)	5-Day Vol. (avg cfs)
Dry Creek at Vernon St.				
Observed Hydrograph	7,950	3,550	1,886	2,142
2008 HEC-1 Run	10,060	4,810	2,200	2,204
% Difference	26.5%	35.5%	16.6%	2.9%
Magpie Cr. near Del Paso Heights				
Observed Hydrograph	560	128	47	47
2008 HEC-1 Run	570	107	45	49
% Difference	1.8%	-16.4%	-4.5%	3.2%
Arcade Cr. near Del Paso Heights				
Observed Hydrograph	2,270	1,090	591	679
2008 HEC-1 Run	3,410	1,730	714	748
% Difference	50.2%	58.7%	20.8%	10.2%

f. February 1998 Flood. Another large storm occurred over the Natomas tributaries watershed in February 1998. The storm amounts for 2 - 6 February 1998 on **Table 6** were used to create a storm isohyetal map for the event, and subbasin storm amounts were used in the HEC-1 model. **Plate 10** may not necessarily be an accurate isohyetal map of the storm, but it shows approximate isolines of the 5-day storm amounts used in the HEC-1 model to develop the flood hydrographs for the Natomas tributaries. The observed mean day flood hydrographs for the Vernon Street and Arcade Creek near Del Paso Heights gages were used for the comparison between the observed and computed flood hydrographs. Ten precipitation stations, identified with "**" in the 2-6 February 1998 rainfall column of **Table 6**, were used as storm distribution patterns. For subbasins above the Dry Creek at Roseville gage, the base flow parameters in the HEC-1 model are:

STARTQ = 3 cfs/sq.mi.
QRCSN = -0.1
RTIOR = 1.05

No base flow was used for the rest of the Steelhead Creek watershed. Loss rates used were zero initial loss and 0.10 inch per hour constant loss.

The HEC-1 model was run to develop flood hydrographs for this storm for the four tributaries to Steelhead Creek. **Figure 9** presents the flood hydrograph from the HEC-1 run for Dry Creek at Roseville compared with the mean day hydrograph observed for the Vernon Street gage. The observed peak flow at Vernon Street gage occurred two hours earlier than the computed peak flow in the HEC-1 run. There is not much difference between the computed and the observed 5-day flood volumes for the Dry and Arcade creek gages. **Table 12** presents a comparison for the peak, and 1-, 3-, and 5-day volumes for the two gaging stations.

Figure 9

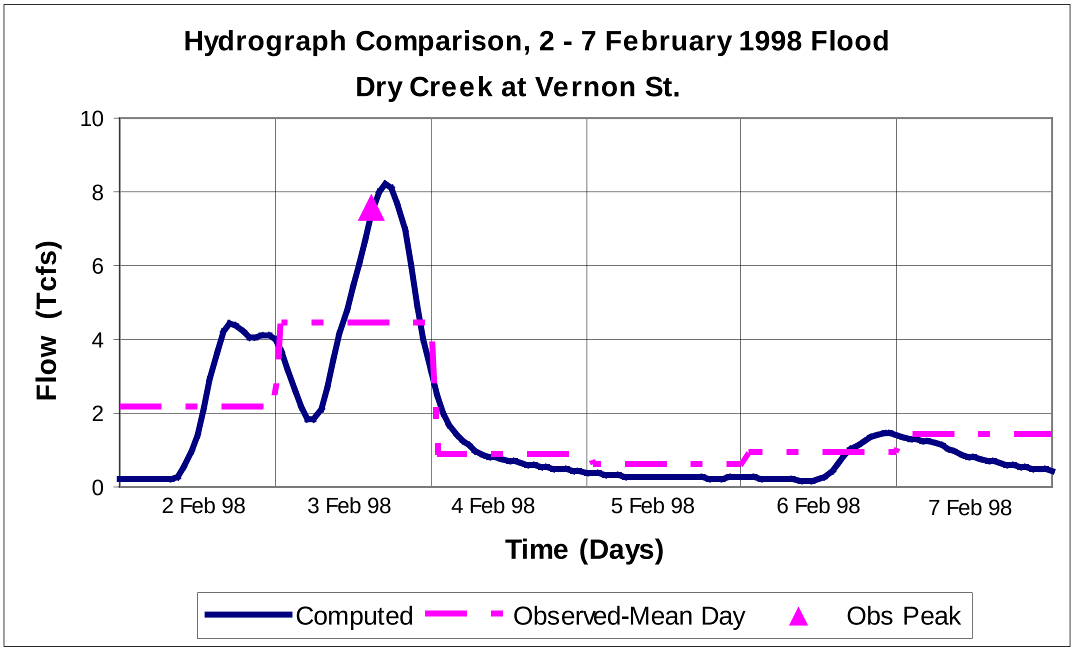


Table 12
2 - 6 February 1998 Flood Volume Comparison
For Two Steelhead Creek Tributary Streamflow Gaging Stations

Hydrograph	Peak (cfs)	1-Day Vol. (avg cfs)	3-Day Vol. (avg cfs)	5-Day Vol. (avg cfs)
Dry Creek at Vernon St.				
Observed Hydrograph	7,549	4,420	2,489	1,791
2008 HEC-1 Run	8,240	4,840	2,620	1,822
% Difference	9.2%	9.5%	5.2%	1.7%
Arcade Cr. Near Del Paso Heights				
Observed Hydrograph	3,320	1,910	1,069	715
2008 HEC-1 Run	3,190	2,100	1,120	718
% Difference	-3.9%	9.9%	4.7%	0.4%

g. 5-Day Volume Frequency Relationships. **Table 13** lists the 5-day flood volumes for the 8-Flood Series for the Steelhead Creek and Natomas Cross Canal tributaries at their downstream index points. The NEMDC Sum in **Table 13** below is the maximum 120 hours of the Steelhead Creek hydrograph developed by adding the 4 tributary hydrographs together at their respective downstream index points. The NEMDC Sum is not necessarily the sum of the four tributary hydrograph volumes, because the maximum 120 hours for the tributary hydrographs do not have the exact same starting and ending times. The 5-day volume frequency curves for Steelhead Creek and Natomas Cross Canal are shown on **Plates 11 and 12**.

Table 13

Summary Table - 8-Flood Series - Five-Day Duration Volumes

Stream at	D.A.	8-Flood Series Five-Day Volumes (in Acre-Feet)							
at Mouth	(sq.mi.)	50%	20%	10%	4%	2%	1%	0.50%	0.20%
Steelhead Cr									
Dry Cr. at NEMDC	116.48	9,250	15,450	19,800	26,600	31,000	35,600	39,800	47,200
Upper NEMDC	27.13	2,010	3,230	4,110	5,300	6,190	7,120	7,980	9,360
OldMag at NEMDC (5-DAY)	4.57	380	594	747	952	1,103	1,260	1,410	1,640
Arcade Cr. At NEMDC	40.14	3,400	5,310	6,650	8,430	9,710	11,050	12,300	14,260
NEMDC Sum	188.32	14,970	24,600	31,340	41,320	48,020	54,980	61,360	71,750
Cross Canal									
Coon Creek at WPRR	112.61	8,760	15,640	20,360	29,430	34,360	39,410	44,040	51,430
Markham Rav. at WPRR	32.36	1,840	3,310	4,370	5,660	6,700	7,760	8,810	10,480
Auburn Rav. at WPRR	79.97	6,770	11,250	14,290	19,460	22,500	25,660	28,600	33,250
Pl.Grove Cr. at WPRR	46.69	4,140	6,500	8,110	10,360	11,880	13,390	15,080	17,420
Curry Creek at WPRR	16.59	1,190	2,000	2,560	3,300	3,850	4,420	4,950	5,810
Cross Canal Sum	288.22	22,690	38,710	49,680	68,160	79,230	90,580	101,420	118,320

The 5-day volumes in **Table 13** and the volume frequency curves on **Plate 11** were used to estimate the percent exceedence of the 5-day volumes for Steelhead Creek for the six historical flood events described above. **Table 14** lists the 5-day volumes for the Steelhead Creek tributaries computed using the HEC-1 program and the storm isohyetal maps for the 6 historical floods, along with the estimated percent exceedence of the 5-day volume for Steelhead Creek hydrographs.

Table 14

5-Day Volume Frequency Relationships for Six Historical Storms

Steelhead Creek Tributaries

Steelhead Cr Index Pt	5-Day Volume (ac-ft)	5-Day Volume % Chance Event (%)	Steelhead Cr Index Pt	5-Day Volume (ac-ft)	5-Day Volume % Chance Event (%)
Feb 1986 Storm			Mid-Jan 1997 Storm		
Dry Cr. At Mouth	38,400	0.6%	Dry Cr. At Mouth	28,500	2.6%
Arcade Cr.-Del Paso Hghts	10,700	0.6%	Arcade Cr.-Del Paso Hghts	7,420	4.6%
Arcade Cr. at Mouth	12,200	0.6%	Arcade Cr. At Mouth	8,300	4.4%
Upper NEMDC abv. Pump	7,090	1.0%	Upper NEMDC abv. Pump	4,230	9.3%
Old Magpie Cr. abv. Pump	1,420	0.6%	Old Magpie Cr. Abv. Pump	810	8.0%
Steelhead Sum	58,300	0.7%	Steelhead Sum	41,600	3.6%
Jan 1995 Storm			Feb 1998 Storm		
Dry Cr. At Mouth	29,800	2.2%	Dry Cr. At Mouth	24,100	5.1%
Arcade Cr.-Del Paso Hghts	8,300	2.7%	Arcade Cr.-Del Paso Hghts	7,380	5.7%
Arcade Cr. at Mouth	9,540	2.3%	Arcade Cr. At Mouth	8,100	4.9%
Upper NEMDC abv. Pump	5,430	3.6%	Upper NEMDC abv. Pump	4,540	7.3%
Old Magpie Cr. abv. Pump	930	4.6%	Old Magpie Cr. Abv. Pump	780	9.0%
Steelhead Sum	45,700	2.4%	Steelhead Sum	37,500	5.4%
New Year 1997 Storm			New Year 2006 Storm		
Dry Cr. At Mouth	17,400	14.5%	Dry Cr. At Mouth	17,700	13.8%
Arcade Cr.-Del Paso Hghts	5,300	15.6%	Arcade Cr.-Del Paso Hghts	5,430	14.6%
Arcade Cr. at Mouth	6,100	13.5%	Arcade Cr. At Mouth	6,370	11.8%
Upper NEMDC abv. Pump	3,370	18.4%	Upper NEMDC abv. Pump	2,820	28.0%
Old Magpie Cr. abv. Pump	600	19.5%	Old Magpie Cr. Abv. Pump	700	13.0%
Steelhead Sum	27,500	14.6%	Steelhead Sum	27,600	14.4%

A sensitivity analysis of storm centerings and runoff discussed in the Natomas GRR Hydrology Appendix showed there was less than a 5 percent difference in runoff on Steelhead Creek for a 1 percent storm centering on the Steelhead drainage and a concurrent storm on Steelhead Creek with the specific centering on Cross Canal drainage. The difference in runoff was also less than 5 percent for the Natomas Cross Canal. To simplify Natomas flood centerings for the Coincident Frequency Analysis, an n-percent chance flood is assumed to be centered on the combined drainages of Steelhead Creek and Natomas Cross Canal. So, if the 5-day flood hydrograph for Steelhead Creek for the New Year 1997 flood is a 15 percent exceedence event, it is assumed to be a 15 percent exceedence event for the Natomas Cross Canal 5-day runoff volume as well. Based on the flood volumes listed in **Table 13**, the 5-day volume of the New Year 1997 flood for the Natomas Cross Canal should be about 43,300 acre-feet. Based on this combined 5-day flood volume for the Cross Canal, 5-day flood hydrographs needed to be computed for the five Cross Canal tributaries for the New Year 1997 flood, to be used in the Coincident Frequency Analysis. Computation of the Natomas Cross Canal tributary hydrographs for the New Year 1997 flood and other five historic floods is discussed in **Section 2.2**.

2.2 Natomas Cross-Canal Historical Flood Hydrographs.

a. Computing 5-Day Volumes for 6 Historical Floods on Natomas Cross Canal. There are several problems with developing historical flood hydrographs for the Natomas Cross Canal tributaries. One is the lack of precipitation stations in the Cross Canal watershed. See **Plate 2**, the watershed map showing the precipitation station locations. Also, there are no flow gages – only a few stage gages on Pleasant Grove Creek at and upstream of Fiddyment Road, and in the upper watersheds of Coon Creek and Auburn Ravine. Coon Creek and Auburn Ravine stage gage locations can be found at **Reference 9,** on the map of Sacramento County ALERT gages. The Pleasant Grove Creek stage gage locations can be found at **Reference 10**, the map of City of Roseville Flood Alert gages. The isohyetal lines on the isohyetal maps for the six historic storms (**Plates 5 through 10**) were extended from Steelhead Creek drainage north through the Cross Canal drainage.

The Civil Engineering Solutions HEC-1 models and the isohyetal maps (**Plates 5 through 10**) were used to compute preliminary runoff hydrographs for the Cross Canal tributaries for the six historical floods. The storm isohyetal maps and subbasins storm amounts for the Cross Canal tributaries were adjusted until the 5-day runoff volumes for the Cross Canal tributaries matched the percent exceedence of the 5-day Steelhead Creek tributary volumes for the same event. (See **Table 14**.) The Pleasant Grove Creek and Markham Ravine drainages are similar to Arcade Creek in east-to-west alignment, drainage area, and elevation range (below 300 feet), so that the percent exceedence event for the Arcade Creek 5-day flood volumes were used as guidance to estimate the flood volumes for those two Cross Canal tributaries. For the larger tributaries, Coon Creek and Auburn Ravine, with large contributing drainage above 300 feet (extending up to 2,000 feet for Coon Creek), the percent exceedence 5-day volumes for the six historical floods were based on the percent exceedence flood volumes for Dry Creek at Steelhead Creek. Curry Creek is adjacent to Upper NEMDC, which was used as a model in case the 5-day volumes on Curry Creek needed adjustment.

Table 15 lists the computed 5-day flood volumes from the above adjusted modeling runs for the Natomas Cross Canal tributaries, as well as the ratios of peak-to-5-day-volume for the computed hydrographs on the Steelhead Creek and Cross Canal tributaries. The HEC-1 models developed by Civil Engineering Solutions, Inc., for the Natomas Cross canal tributaries, discussed in the Natomas GRR Hydrology Appendix (**Reference 6**), assumed that future housing and urbanization projects were in place. At the present time, they have yet to be constructed. One review comment on the Hydrology Appendix was that the Cross Canal tributary peak flows computed for the Hydrology Appendix had much higher peak flows in proportion to their flood volumes and contributing drainage areas. The relationship for Cross Canal peak flows should be more in line with the ratios of peak flow to flood volume and to drainage area for the Steelhead Creek tributaries.

Table 15
Ratio of Peaks to 5-Day Volumes
for 6 Historical Floods on Natomas Tributaries

Stream at Mouth	D.A. (sq.mi.)	8-Flood Series - Peaks, Volumes and Ratios: Peak to Volume						Average Peak to Volume
		Feb-86	Jan-95	NY 1997	MidJan 97	Feb-98	NY 2006	
Steelhead Cr								
Dry Cr. At Steelhead Cr.	Peak (cfs)	10,040	12,080	5,110	7,830	7,350	6,900	
5-day Vol. (ac-ft)		38,400	29,800	17,400	28,500	24,100	17,700	
Drainage Area 116.48 sq.mi.	PK/Vol.	0.26	0.41	0.29	0.27	0.30	0.39	0.32
Upper NEMDC	Peak (cfs)	3,830	3,840	2,610	2,610	1,610	2,110	
5-day Vol. (ac-ft)		7,090	5,430	3,370	4,230	4,540	2,820	
Drainage Area 27.13 sq.mi.	PK/Vol.	0.54	0.71	0.77	0.62	0.35	0.75	0.62
Old Magpie Cr. above Pump	Peak (cfs)	831	918	603	673	389	573	
5-day Vol. (ac-ft)		1,420	930	600	810	780	700	
Drainage Area 4.57 sq.mi.	PK/Vol.	0.59	0.99	1.01	0.83	0.50	0.82	0.79
Arcade Cr. At Steelhead Cr.	Peak (cfs)	3,720	4,950	2,640	3,470	3,200	3,360	
5-day Vol. (ac-ft)		12,200	9,540	6,100	8,300	8,100	6,370	
Drainage Area 40.14 sq.mi.	PK/Vol.	0.30	0.52	0.43	0.42	0.40	0.53	0.43
Steelhead Cr. Sum	Peak (cfs)	14,060	17,840	8,470	11,300	11,050	10,860	
5-day Vol. (ac-ft)		58,300	45,700	27,500	41,600	37,500	27,600	
Drainage Area 188.32 sq.mi.	PK/Vol.	0.24	0.39	0.31	0.27	0.29	0.39	0.32
Cross Canal		Feb-86	Jan-95	NY 1997	MidJan 97	Feb-98	NY 2006	Average
Coon Creek at WPRR	Peak (cfs)	11,700	26,500	8,250	13,700	10,150	9,970	
5-day Vol. (ac-ft)		35,500	29,100	17,600	20,700	18,050	13,460	
Drainage area 112.61 sq.mi.	PK/Vol.	0.33	0.91	0.47	0.66	0.56	0.74	0.61
Markham Rav. At WPRR	Peak	6,510	4,830	2,520	4,810	2,550	4,120	
5-day Vol. (ac-ft)		8,620	4,850	3,700	5,280	5,130	3,440	
Drainage Area 32.36 sq.mi.	PK/Vol.	0.76	1.00	0.68	0.91	0.50	1.20	0.84
Auburn Rav. At WPRR	Peak (cfs)	11,700	10,200	4,290	6,840	5,490	5,700	
5-day Vol. (ac-ft)		26,450	21,000	12,500	16,360	14,100	10,200	
Drainage Area 79.97 sq.mi.	PK/Vol.	0.44	0.49	0.34	0.42	0.39	0.56	0.44
Pl.Grove Cr. At WPRR	Peak (cfs)	7,870	9,100	4,550	7,360	4,610	5,470	
5-day Vol. (ac-ft)		14,900	11,400	6,560	9,090	9,330	6,160	
Drainage Area 46.69 sq.mi.	PK/Vol.	0.53	0.80	0.69	0.81	0.49	0.89	0.70
Curry Creek at WPRR	Peak (cfs)	2,520	2,500	1,570	1,680	1,020	1,290	
5-day Vol. (ac-ft)		4,650	3,330	2,130	2,890	3,000	1,730	
Drainage Area 16.59 sq.mi.	PK/Vol.	0.54	0.75	0.74	0.58	0.34	0.75	0.62
Cross Canal Sum	Peak (cfs)	30,700	43,600	16,100	23,200	20,800	21,300	
5-day Vol. (ac-ft)		89,800	72,900	42,500	54,300	49,500	35,000	
Drainage Area 288.22 sq.mi.	PK/Vol.	0.34	0.60	0.38	0.43	0.42	0.61	0.46

Upper NEMDC (Steelhead tributary) and Curry Creek (Cross Canal tributary) are adjacent basins on the valley floor and have similar ratios of computed peak to 5-day volume for each of the six flood events. The 6-event averaged ratio of peak/5-day volume (**Table 15**, right-hand column) is the same, 0.62, for Upper NEMDC and Curry Creek.

Arcade Creek (Steelhead tributary) and Pleasant Grove Creek and Markham Ravine (Cross Canal tributaries) are similar in orientation and elevation. However, because of the highly urbanized HEC-1 models used for Pleasant Grove Creek and Markham Ravine, the 6-event averaged ratio of peak/5-day volume for Pleasant Grove Creek is 60 percent higher than for Arcade Creek and for Markham Ravine is nearly two times that of Arcade Creek.

Dry Creek (Steelhead tributary) and Coon Creek and Auburn Ravine (Cross Canal tributaries) have larger drainage areas as well as headwaters at much higher elevations than the other Natomas tributaries. Because of the highly urbanized HEC-1 models used for Auburn Ravine and Coon Creek, the 6-event averaged ratio of peak/5-day volume for Auburn Ravine is 38 percent higher than for Dry Creek and is 91 percent higher for Coon Creek than for Dry Creek.

Table 16 shows the ratios of peak-to-drainage-area for the computed hydrographs on the Steelhead Creek and Cross Canal tributaries.

Table 16
Ratio of Peaks to Drainage Areas
for 6 Historical Floods on Natomas Tributaries

Stream at Mouth	D.A. (sq.mi.)	8-Flood Series - Ratios of Peaks to Drainage Areas Feb-86	Jan-95	NY 1997	MidJan 97	Feb-98	NY 2006	Average Peak to D.A.
Steelhead Cr								
Dry Cr. At Steelhead Cr.	Peak (cfs)	10,040	12,080	5,110	7,830	7,350	6,900	
Drainage Area (sq.mi.)								
116.48	Pk/D.A.	86.2	103.7	43.9	67.2	63.1	59.2	70.6
Upper NEMDC	Peak (cfs)	3,830	3,840	2,610	2,610	1,610	2,108	
Drainage Area (sq.mi.)								
27.13	Pk/D.A.	141.2	141.5	96.2	96.2	59.3	77.7	102.0
Old Magpie Cr. above Pump	Peak (cfs)	831	918	603	673	389	573	
Drainage Area (sq.mi.)								
4.57	Pk/D.A.	181.8	200.9	131.9	147.3	85.1	125.4	145.4
Arcade Cr. At Steelhead Cr.	Peak (cfs)	3,720	4,950	2,640	3,470	3,200	3,360	
Drainage Area (sq.mi.)								
40.14	Pk/D.A.	92.7	123.3	65.8	86.4	79.7	83.7	88.6
Steelhead Cr. Sum	Peak (cfs)	14,060	17,840	8,470	11,300	11,050	10,860	
Drainage Area (sq.mi.)								
188.32	Pk/D.A.	74.7	94.7	45.0	60.0	58.7	57.7	65.1
Cross Canal		Feb-86	Jan-95	NY 1997	MidJan 97	Feb-98	NY 2006	Average
Coon Creek at WPRR	Peak (cfs)	11,700	26,500	8,250	13,700	10,150	9,970	
Drainage Area (sq.mi.)								
112.61	Pk/D.A.	103.9	235.3	73.3	121.7	90.1	88.5	118.8
Markham Rav. At WPRR	Peak (cfs)	6,510	4,830	2,520	4,810	2,550	4,120	
Drainage Area (sq.mi.)								
32.36	Pk/D.A.	201.2	149.3	77.9	148.6	78.8	127.3	130.5
Auburn Rav. At WPRR	Peak (cfs)	11,700	10,200	4,290	6,840	5,490	5,700	
Drainage Area (sq.mi.)								
79.97	Pk/D.A.	146.3	127.5	53.6	85.5	68.7	71.3	92.2
Pl.Grove Cr. At WPRR	Peak (cfs)	7,870	9,100	4,550	7,360	4,610	5,470	
Drainage Area (sq.mi.)		14,900	11,400	6,560	9,090	9,330	6,160	
46.69	Pk/D.A.	168.6	194.9	97.5	157.6	98.7	117.2	139.1
Curry Creek at WPRR	Peak (cfs)	2,520	2,500	1,570	1,680	1,020	1,290	
Drainage Area (sq.mi.)								
16.59	Pk/D.A.	151.9	150.7	94.6	101.3	61.5	77.8	106.3
Cross Canal Sum	Peak (cfs)	30,700	43,600	16,100	23,200	20,800	21,300	
Drainage Area (sq.mi.)								
288.22	Pk/D.A.	106.5	151.3	55.9	80.5	72.2	73.9	90.0

The 6-event averaged ratio of peak/drainage area (**Table 16**, right-hand column) is nearly the same for the adjacent stream drainages, Upper NEMDC and Curry Creek, with ratios of 102 and 106.3, respectively. These basins are in close agreement for ratios of both peak to 5-day volume and peak to drainage area. The computed historical reproduction hydrographs for Curry Creek do not appear to need adjustment.

The 6-event averaged ratio of peak/drainage area for Arcade Creek is 88.6. While Markham Ravine and Pleasant Grove Creek are the tributaries to the Natomas Cross Canal most similar to Arcade Creek, the 6-event averaged ratio of peak/drainage area for Markham Ravine is 47 percent higher than for Arcade Creek and for Pleasant Grove Creek is 57 percent higher than for Arcade Creek. These higher ratios for the Cross Canal tributaries can be explained by the HEC-1 models that included future urbanization on those watersheds. The peak flows for present conditions on Markham Ravine and Pleasant Grove Creek should be lower.

The 6-event averaged ratio of peak/drainage area for Dry Creek is 70.6. The Cross Canal tributaries most similar to Dry Creek are Auburn Ravine and Coon Creek. The 6-event averaged ratio of peak/drainage area for Auburn Ravine is 31 percent higher than that for Dry Creek while the averaged ratio for Coon Creek is 68 percent higher than for Dry Creek. The peak flows for present conditions on Auburn Ravine and Coon Creek should be lower.

Based on the differences in the ratios presented in **Tables 15 and 16**, the hydrographs for Auburn Ravine, Coon Creek, Markham Ravine, and Pleasant Grove Creek were reshaped with lower peak flows. This process is explained in **Section 2.2.b**.

b. <u>Re-shaping the Natomas Cross Canal Historical Hydrographs</u>. Once the 5-day runoff volumes for the six historic floods on the Natomas Cross Canal tributaries were determined, the flood hydrographs were re-shaped (except for Curry Creek), with lower peak flows, more in line with the peak to volume and to drainage area ratios for the Steelhead Creek tributaries (**Tables 15 and 16** above). The same Steelhead Creek tributaries were used for the hydrograph patterns: Arcade Creek at Steelhead Creek as a pattern for Pleasant Grove Creek and Markham Ravine at their downstream WPRR index points, and Dry Creek at Steelhead Creek as a pattern for Auburn Ravine and Coon Creek at their downstream WPRR index points. The computed flood volumes for the Cross Canal tributaries remained the same, but volume lost by re-shaping for lower peak flows was offset by the addition of recession flow. The timing of the peak flows on the Cross Canal tributaries was not changed. Examples of re-shaping of the Cross Canal tributary hydrographs for the New Year 1997 flood are shown on **Figure 10**, Pleasant Grove Creek at WPRR, based on Arcade Creek, and **Figure 11**, Coon Creek at WPRR, based on Dry Creek at Steelhead Creek.

The figures show how the high peak flows on the Cross Canal tributaries were reduced by hydrograph re-shaping. Rapid hydrograph fluctuations were filled in. Recession base flow was added to the hydrographs for the Cross Canal tributaries with major contributing drainage above 300 feet (Coon Creek and Auburn Ravine). Minor waves in the flood hydrographs were not adjusted. While the Arcade Creek hydrograph appears to have base flow, the higher flow trailing after the main wave is due to water being pumped from interior drainage areas upstream of the mouth of Arcade Creek.

Figure 10

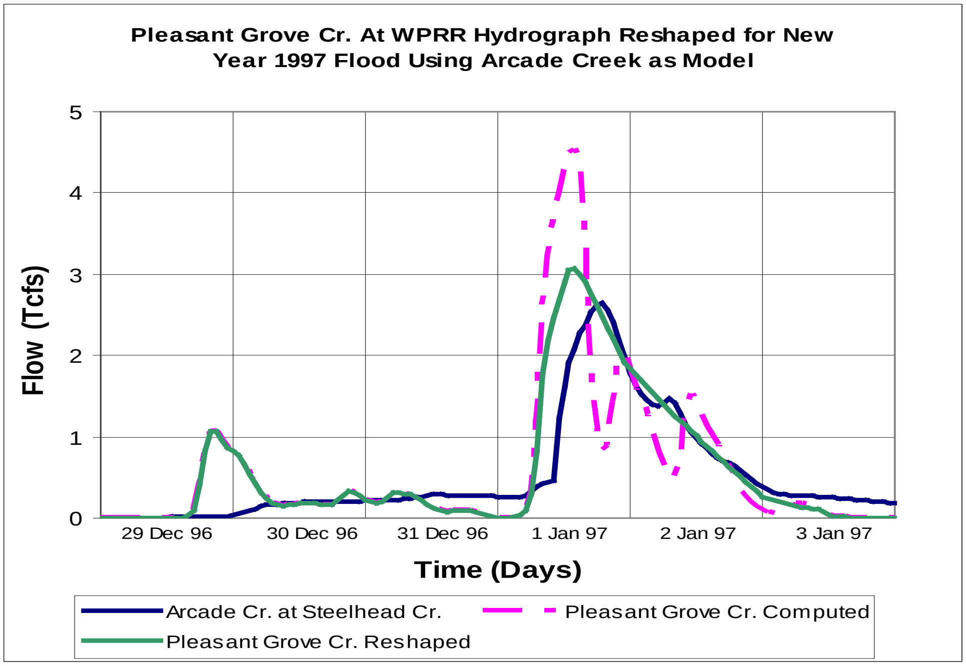
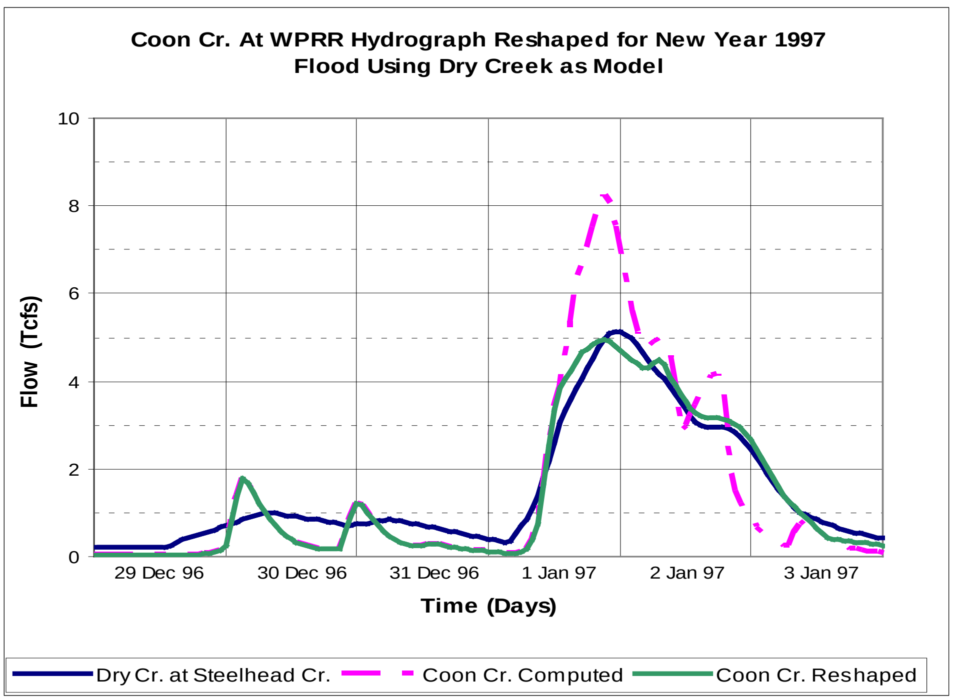
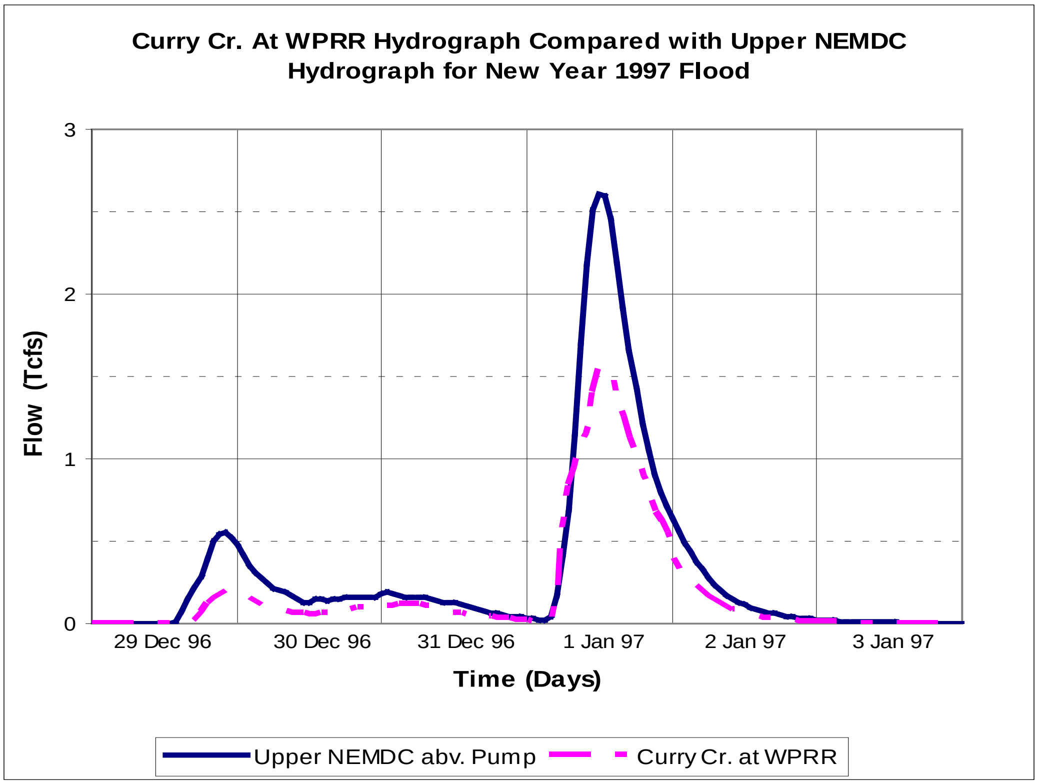


Figure 11



The smaller valley tributaries, Upper NEMDC and Old Magpie Creek, have higher peak flows in proportion to their flood volumes and drainage areas, but those peak flows would not have as much effect on the downstream Steelhead Creek hydrograph, even if they contributed directly to Steelhead Creek instead of being pumped in; their drainage areas and flood volumes are small compared with the larger tributaries, Dry and Arcade creeks. The contribution from Curry Creek to flows at the Natomas Cross Canal does not have a large effect either. The Rio Linda rainfall gage was used to distribute the precipitation over these two drainages for the six historical storms. The ratios of peak to flood volume and to drainage area for Curry Creek are very similar to the ratios for Upper NEMDC. The historical flood hydrograph for Curry Creek was not re-shaped. **Figure 12** presents the flood hydrographs for Curry Creek and Upper NEMDC for the New Year 1997 flood.

Figure 12



2.3 Use of Historical Flood Hydrographs on Natomas Tributaries. The Natomas tributary hydrographs for the six historic floods were provided to Hydraulic Design Section to be used for upstream boundary conditions in the hydraulic modeling. The historic flood hydrographs were at the following locations: Coon Creek at WPRR, Markham Ravine at WPRR, Auburn Ravine at WPRR, Pleasant Grove Creek at WPRR, Curry Creek at WPRR, Upper NEMDC above and below the NEMDC Stormwater Pumping Station, Dry Creek above Steelhead Creek confluence, Old Magpie Creek above and below Pump Station 157, and Arcade Creek above Steelhead Creek confluence. **Plate 13** shows the New Year 1997 computed flood hydrographs for Curry

Creek and the Steelhead Creek tributaries and the reshaped flood hydrographs for Pleasant Grove Creek, Auburn Ravine, Markham Ravine, and Coon Creek.

3.0 Development of 8-Flood Series Hydrographs for Natomas Tributaries

Development of the 8-Flood Series hydrographs for the Natomas tributaries follows Comprehensive Study methodology. The Comprehensive Study used 30-day hydrographs consisting of six 5-day waves, with the 4th wave being the highest. The process includes: 1) obtaining the average flood flow rates from the unregulated frequency curves, 2) separating these average flows into wave volumes, and 3) distributing volumes into the 6-wave series.

All of the Natomas tributaries at their respective downstream index points are unregulated. The index points for Upper NEMDC and Old Magpie Creek are upstream of their respective pumping stations. The 5-day volume frequency curves for the Natomas tributaries are shown on **Plates 11 and 12. Plates 14 and 15** present the 10-day volume frequency curves. The 5-day volumes for the 8-Flood Series for the Natomas tributaries are listed on **Table 13** in **2.1.g. Table 17** below lists the 10-day volumes for the 8-Flood Series.

Table 17

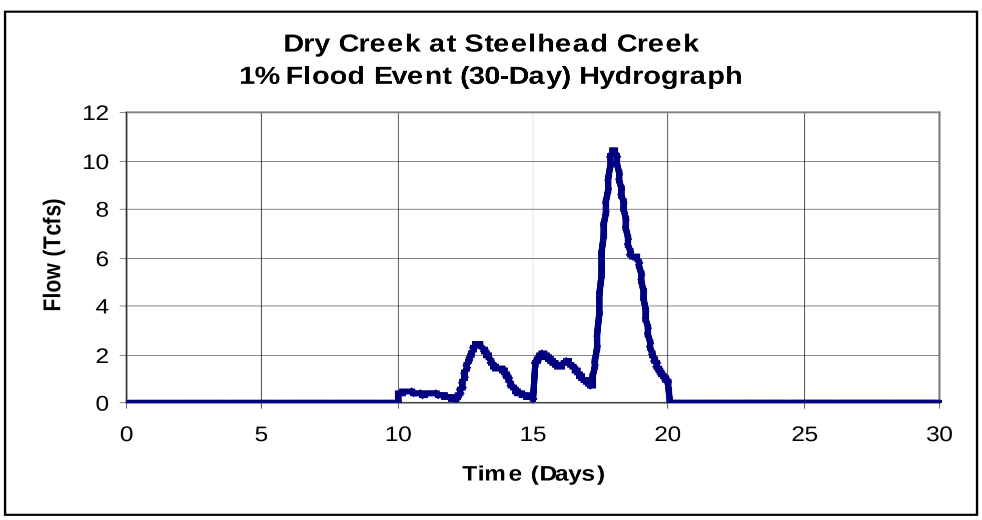
Summary Table - 8-Flood Series - Ten-Day Duration Volumes

Stream at at Mouth	D.A. (sq.mi.)	8-Flood Series Five-Day Volumes (in Acre-Feet)							
		50%	20%	10%	4%	2%	1%	0.50%	0.20%
Steelhead Cr									
Dry Cr. at NEMDC	116.48	11,000	18,300	23,600	32,700	38,200	43,900	49,100	58,700
Upper NEMDC	27.13	2,400	3,840	4,920	6,400	7,510	8,700	9,760	11,500
OldMag at NEMDC (5-DAY)	4.57	470	724	891	1,200	1,390	1,590	1,770	2,070
Arcade Cr. at NEMDC	40.14	4,220	6,570	8,190	10,300	11,900	13,600	15,100	17,600
NEMDC Sum	188.32	18,090	29,434	37,601	50,600	59,000	67,790	75,730	89,870
Cross Canal									
Coon Creek at WPRR	112.61	10,900	19,500	25,400	38,300	44,700	51,400	57,600	67,300
Markham Rav. at WPRR	32.36	2,380	4,170	5,450	7,320	8,610	9,920	11,200	13,300
Auburn Rav. at WPRR	79.97	8,600	14,200	18,100	25,300	29,300	33,400	37,300	43,400
Pl.Grove Cr. at WPRR	46.69	5,160	8,060	10,200	13,100	15,000	17,000	19,200	22,100
Curry Creek at WPRR	16.59	1,490	2,490	3,180	4,120	4,820	5,540	6,230	7,330
Cross Canal Sum	288.22	28,530	48,420	62,330	88,140	102,430	117,260	131,530	153,430

For consistency with the Comprehensive Study, the computed New Year 1997 flood hydrographs for the Natomas tributaries at their respective downstream index points, or upstream of their respective pumping stations for Old Magpie Creek and Upper NEMDC, were used as the pattern hydrographs for the synthetic 8-Flood Series. For the Comprehensive Study, the basic pattern of all synthetic flood hydrographs was a 30-day hourly time series consisting of six waves, each 5 days in duration. Flood volumes were ranked and distributed into the basic pattern. The highest wave volume was distributed into the fourth, or main, wave. The second highest volume preceded the main wave. So, the two highest waves are in the middle ten days of the 30-day hydrograph. The upstream tributary index points used for the Comprehensive Study are listed on **Table 1**. They flow out of the mountains to the east, west, and north of the Sacramento Valley and have high flows during the rainy season. The Natomas tributaries flow out of the foothills or originate on the valley floor. Flows on these tributaries can be high during and immediately after a rainstorm. Without additional rainfall, the flows drop to base flow or to urban runoff levels. The average flows are a lot lower than for the Comp Study tributaries on **Table 1**. The Natomas tributary flows for the four smaller waves would be so minor, that zero runoff was assumed for the 30-day hydrographs except for the middle 10 days (Waves 3 and 4).

The 1 percent flood hydrograph for Dry Creek at Steelhead Creek was developed in the following way. The 5-day flood pattern hydrograph for 30 Dec 1996 to 3 Jan 1997 for Dry Creek at its downstream index point is shown on **Figure 11** and **Plate 13**. The 5-day flood volume for this pattern hydrograph is 17,400 acre-feet. The 5-day flood volume for the 1 percent flood for Dry Creek is 35,600 acre-feet. The ratio of the 1 percent event 5-day volume to the New Year 1997 5-day volume is 35,600 / 17,400 or 2.046. This ratio was applied to the hourly ordinates of the computed 5-day New Year 1997 hydrograph for Dry Creek at Steelhead Creek, to define the 1 percent flood hydrograph for Wave 4 at the Dry Creek index point. The difference between the 1 percent 5-day volume (35,600 ac-ft) for Dry Creek at Steelhead Creek index point and the 1 percent 10-day volume (43,900 ac-ft) for the Dry Creek index point is 8,300 acre-feet. The ratio of 8,300 ac-ft to the New Year 1997 5-day volume for Dry Creek at Steelhead Creek is 8,300 / 17,400, or 0.477. This ratio was applied to the New Year 1997 flood hydrograph at the Dry Creek index point, to define the hydrograph for Wave 3 of the 30-day 1 percent event flood hydrograph at the Dry Creek index point. **Figure 13** below shows the shape of the 30-day 1 percent event hydrograph for Dry Creek at Steelhead Creek, with zero flow for waves 1 – 2 and 5 – 6. Wave 4 is higher than Wave 3.

Figure 13



The rest of the floods in the 8-Flood Series for Dry Creek, as well as the hydrographs for the other eight Natomas tributaries, were developed using the same method. These hydrographs are consistent in shape and timing with the synthetic flood hydrographs for the Sacramento River tributary index points listed on **Table 1**.

The 30-day hydrographs for Upper NEMDC above the NEMDC Stormwater Pumping station and Old Magpie Creek above Pump 157 were routed through their respective pumping stations for each of the 8-Flood Series.

The Natomas tributary 30-day hydrographs for the 8-Flood Series were provided to Hydraulic Design Section for use as upstream boundary conditions for the hydraulic model. For Upper NEMDC and Old Magpie Creek, hydrographs for above and below their respective pumping stations were provided to Hydraulic Design Section.

4.0 Natomas Cross Canal (NCC) and Steelhead Creek (SHC) Coincident Frequency Study

The Comprehensive Study hydrology included coincident flood centerings for the Sacramento River tributaries large enough to have an influence on the flows downstream of their confluences with the mainstem. Flood hydrograph contributions from the tributary Natomas Cross Canal (NCC) and Steelhead Creek (SHC) are negligible in comparison with the mainstem flood flows, such that the tributary flow or stage hydrographs do not need to be considered when developing stage-frequency functions for the mainstem channels. However, the mainstem channel stages still need to be considered when developing stage-frequency functions on the tributaries. For this phase of the analysis, the Sacramento Mainstem flood series is used as the mainstem for the Natomas Cross Canal, and either the American River or the Sacramento Mainstem is used as the mainstem for the Steelhead Creek tributary, depending upon percent exceedence. For low mainstem stage conditions, Steelhead Creek flows directly to the Sacramento River rather than mingling flows with the American River.

4.1 Total Probability Theorem. Instead of the Comprehensive Study concurrent flood centering methodology, a total probability approach was used to evaluate coincident flood stages on the Natomas Cross Canal and Steelhead Creek. The procedure used was an extension of the Total Probability method documented in **Reference 11**, Procedures for Developing Stage-Probability Functions for Tributary Streams, prepared by David Ford Consulting Engineers (Ford) in February 2007.

Tangible benefit of a flood management project is computed, in part, as the expected value of inundation damage reduced. This computation requires a stage-frequency function at the location of interest. If that location is on a tributary stream, development of the function must account properly for the influence of the mainstem stream into which the tributary flows. A systematic, uniform approach is required for development of the stage-frequency functions for the locations of interest. The procedure begins with an assessment of the degree to which the tributary is dependent on the mainstem. An overview flowchart for the tributary analysis procedure is shown on **Plate 16**.

If the tributary is not dependent on mainstem conditions (Case 1), then the necessary information can be developed using typical riverine analyses: estimate the discharge for a specified probability, use that as the upstream boundary condition, and use a rating curve or similar control as the downstream boundary condition for the hydraulics model.

If tributary conditions are hydraulically dependent on mainstem conditions, can the frequency of the stage at the tributary location be predicted, given the mainstem conditions? If so (Case 3), then the Comprehensive Study methodology is used to develop the tributary flow-frequency function and the mainstem stage-frequency function. A channel model is developed for the reach of interest, and a resulting stage-frequency function is derived for the tributary index location.

If tributary conditions cannot be predicted reliably from mainstem conditions (Case 2), then combinations of boundary conditions are applied to the standard watershed and channel models. Using the results from analysis of tributary stages computed with varying downstream boundary conditions, the total probability equation is used to compute the desired stage-frequency function at the tributary location. The equation is:

$$F(stage_{tributary}) = \sum_{\substack{mainstem \\ conditions}} \left(F(stage_{tributary} \mid stage_{mainstem}) \times F(stage_{mainstem}) \right)$$

If a correlation exists between the tributary and mainstem, but is not definitive (Case 4), then a conditional probability analysis needs to be done. Practical methods to accomplish this have yet to be developed and field-tested.

4.2 Application to Natomas Tributaries. The coincident-frequency procedures that Ford used to develop stage-frequency curves for the Natomas Cross Canal and Steelhead Creek channels are described in the memorandum, "NCC/SHC Coincident Frequency Study: Exposition of Analytical Procedures," dated September 10, 2008, prepared by David Ford Consulting Engineers (**Reference 12**). Primary technical tasks include assessing hydrologic dependence between tributary and mainstem channels and identifying flow regimes where hydrologic independence may be presumed. A secondary task is identifying timing differences between tributary and mainstem peak stages. Total probability methodology relies on historical rainfall and streamflow data. Stage records from the California Data Exchange Center (CDEC, **Reference 13**) were used for the analysis. Due to the lack of stage data on the Natomas Cross Canal, CDEC stage records for the Dry Creek gage at Vernon Street (VRS) were substituted to develop a cross-correlation with the Sacramento River at Verona (VON) records. Records for the Sacramento River at I Street (IST) and at Ord Ferry (ORD) gages were used to supplement/correct the VON stage records. Similarly, due to the unavailability of long-term records for Steelhead Creek, Arcade Creek (AMC) records were cross-correlated with American River at H-Street gage (HST) records. American River at Fair Oaks (AFO) records were used to fill in missing values in the HST record. **Table 18** summarizes the primary stream gages used for this study. Gaging station locations (except for ORD) are shown on **Plate 1**.

Table 18
CDEC Gage Records Used for Hydrologic Dependence Analysis

Gage Name	CDEC gage ID	Period of Record
Sacramento River at Verona	VON	01Jan1984 – Present
Sacramento River at I Street	IST	01Jan1984 – Present
Sacramento River at Ord Ferry	ORD	01Jan1984 – Present
American River at H Street	HST	01Jan1984 – Present
American River at Fair Oaks	AFO	02Nov1998 – Present
Dry Creek at Vernon Street	VRS	19Oct1996 – Present
Arcade Creek at Winding Way	AMC	29Oct1996 – Present

The memorandum, "Cross-Correlation Analysis Results for NCC/SHC Coincident-Frequency Study," dated April 17, 2008, prepared by David Ford Consulting Engineers (**Reference 14**), describes the methods Ford used to assess conditions of hydrologic dependence between (1) Steelhead Creek and the American River, (2) Natomas Cross Canal and the

Sacramento River, and (3) the American River and the Sacramento River. It also identifies peak-stage timing differences between each tributary and the downstream mainstem channel.

Table 19 shows the tributary/mainstem confluence water surface elevations used as input in the Hydraulic Design Section's hydraulic models for the Natomas Cross Canal (NCC) and Steelhead Creek (SHC) tributaries as a function of mainstem annual exceedence probability (AEP) stages. Water surface elevation (WSEL) values are referenced to the National Geodetic Vertical Datum of 1929 (NGVD29). Water surface elevations on SHC and NCC in **Table 19** correspond to stages on the American River and on the Sacramento River, respectively. For the more frequent mainsteam AEP between 0.50 and 0.04, Steelhead Creek stages are affected more by stages on the Sacramento River than by flows down the American River.

An analytical approach based on historical storm event data was used to characterize tributary/mainstem dependencies. Local event Annual Exceedence Probabilities (AEPs) were assigned to individual storm events, based on precipitation records from rainfall gages close to the SHC and NCC drainages. Rainfall frequency data was provided by Rainfall Depth-Duration Frequency Analysis for California Rain Gages (**Reference 15**), assembled by retired California State Climatologist Jim Goodridge. Historical mainstem peak flows were matched to concurrent local rainfall events on an event-by-event basis. Based on local storm magnitudes, the set of historic events was partitioned into return-frequency classes. Distributions for rarer AEP events were based on projected regional meteorologic patterns. Only rainfall and flow/stage records collected after 1980 were used for the analysis. It was assumed that n-year local flow event corresponded to the n-year local rainfall event, and that mainstem/tributary conditional distribution patterns can be extrapolated for rarer events using general knowledge of regional storm patterns and local channel hydraulics.

Table 19

Applied Stage-Frequency Functions for Mainstem AEP Events

Mainstem-event AEP	Steelhead Creek (SHC) Downstream WSEL (ft. NGVD29)	Natomas Cross Canal (NCC) Downstream WSEL (ft. NGVD29)
0.500	24.09	33.08
0.200	24.80	35.10
0.010	25.70	36.34
0.040	30.71	39.34
0.020	32.65*	40.10
0.010	35.43*	41.62
0.005	37.18*	43.00
0.002	42.62*	44.35

Notes:

AEP = Annual Exceedence Probability

WSEL = Water Surface Elevation

* WSEL is stage for American River conditions. All other WSELs are stages on the Sacramento River Mainstem.

The Hydraulic Design models were used to generate peak water surface elevations for the SHC and NCC index points for various combinations of tributary discharge and fixed mainstem stage (per **Table 19**). The tributary discharge rates were characterized by local-event AEP; similarly, the downstream confluence stages were characterized by mainstem AEP. The computed NCC and SHC index point stage values corresponded to regulated mainstem conditions.

4.3 Computational Results. Ford developed stage-frequency functions for the Natomas Cross Canal and Steelhead Creek index points. **Table 20** presents the stage-frequency functions for the NCC and SHC index points based on Ford's coincident-frequency evaluation. The stage values were computed under regulated mainstem conditions. Water surface elevation (WSEL) values are referenced to the National Geodetic Vertical Datum of 1929 (NGVD29).

Table 20

Computed Stage-Frequency Functions for Local AEP Events

Local-event AEP	Steelhead Creek (SHC) Index Point WSEL (ft. NGVD29)	Natomas Cross Canal (NCC) Index Point WSEL (ft. NGVD29)
0.500	26.3	33.9
0.200	28.6	34.5
0.010	29.9	34.8
0.040	31.4	36.6
0.020	33.4	37.8
0.010	35.5	38.6
0.005	37.4	40.1
0.002	40.1	42.4

Notes:
AEP = Annual Exceedence Probability
WSEL = Water Surface Elevation
SHC index point is located at RM 3.713
NCC index point is located at RM 4.323

Stages listed in **Table 20** are based on UNET modeling, not on the latest HEC-RAS model. The above stages may change when the HEC-RAS model is used for the analyses. The memorandum, "NCC/SHC Coincident Frequency Study: Computational Results," dated September 10, 2008 prepared by Ford (**Reference 16**), provides additional details regarding the results in **Table 20** from the analyses - the special factors considered, the hydraulic profiles and probabilistic relations used in the computations, and the coincident stage-frequency functions.

Table 21 shows the combination of which mainstem flood hydrographs are being used in combination with which Natomas tributary flood hydrographs in the HEC- RAS hydraulic model. These flood hydrograph combinations are being used in preparation for the F3 Conference Milestone. Different combinations of floods may be tested for later analysis.

Preliminary analysis determined that, for the mouth of the Natomas Cross Canal, the flood stages for the Sacramento Mainstem and Shanghai-Yuba centerings were similar. So the Shanghai-Yuba flood series hydrographs are not being used in the current phase (pre-F3 Milestone) of the analysis, but will be tested later.

Table 21
Flood Hydrograph Combinations used in HEC-RAS Hydraulic Model
for Current Phase of Analysis

Sacramento Mainstem Flood-event AEP	Steelhead Creek Flood-event AEP	Natomas Cross Canal Flood-event AEP
0.500	0.500	0.500
0.200	0.500	0.500
0.010	0.200	0.200
0.040	0.010	0.010
0.020	0.040	0.040
0.010	0.020	0.020
0.005	0.010	0.010
0.002	0.005	0.005
American River Flood-event AEP	Steelhead Creek Flood-event AEP	Natomas Cross Canal Flood-event AEP
0.500	0.500	0.500
0.200	0.500	0.500
0.010	0.200	0.200
0.040	0.010	0.010
0.020	0.040	0.040
0.010	0.020	0.020
0.005	0.010	0.010
0.002	0.005	0.005

Notes: AEP = Annual Exceedence Probability

5.0 List of References

1. Sacramento and San Joaquin River Basins Comprehensive Study, Technical Studies Documentation. Appendix B: Synthetic Hydrology Technical Documentation. Appendix C: Reservoir Operations Modeling, Existing Design Operations and Reoperation Analyses. State of California Reclamation Board. U.S. Army Corps of Engineers, Sacramento District. December 2002.

2. Yuba River Basin Project General Reevaluation Report, Appendix A, Synthetic Hydrology & Reservoir Operations Technical Documentation. U.S. Army Corps of Engineers, Sacramento District. Aug 2004, corrected June 2008.

3. Folsom Dam and Lake Revised PMF Study, American River Basin, California, Hydrology Office Report. U.S. Army Corps of Engineers, Sacramento District. October 2001.

4. Hydrometeorological Report No. 59, Probable Maximum Precipitation for California. U.S. Department of Commerce, National Oceanic and Stmospheric administration. U.S. Army Corps of Engineers. February 1999.

5. Rain Flood Flow Frequency Analysis, American River, California, Office Report. U.S. Army Corps of Engineers, Sacramento District. August 2004.

6. Natomas General Reevaluation Report, Natomas Cross Canal and Steelhead Creek Watersheds, Placer, Sacramento, and Sutter Counties, California, Hydrology Appendix. U.S. Army Corps of Engineers, Sacramento District. October 2006.

7. Dry Creek, Placer and Sacramento Counties, California, Hydrology Office Report. U.S. Army Corps of Engineers, Sacramento District. July 1984, revised April 1988

8. "Use of Radar-Rainfall Estimates to Model the January 9-10, 1995 Floods in Sacramento, CA," by David C. Curtis, Ph.D., NEXRAIN Corporation, and John H. Humphrey, Ph.D., P.E., C.C.M., HYDMET, Inc. Paper presented October 1995

9. Sacramento County ALERT System gage map: <http://www.sacflood.org/alrtloc1.htm>.

10. City of Roseville Public Works Web page with access to maps with local gaging station locations: <http://www.roseville.ca.us/pw/engineering/floodplain_management/roseville_current_stream_levels/default.asp>.

11. Procedures for Developing Stage-Probability Functions for Tributary Streams. Prepared by David Ford Consulting Engineers, Inc. February 26, 2007.

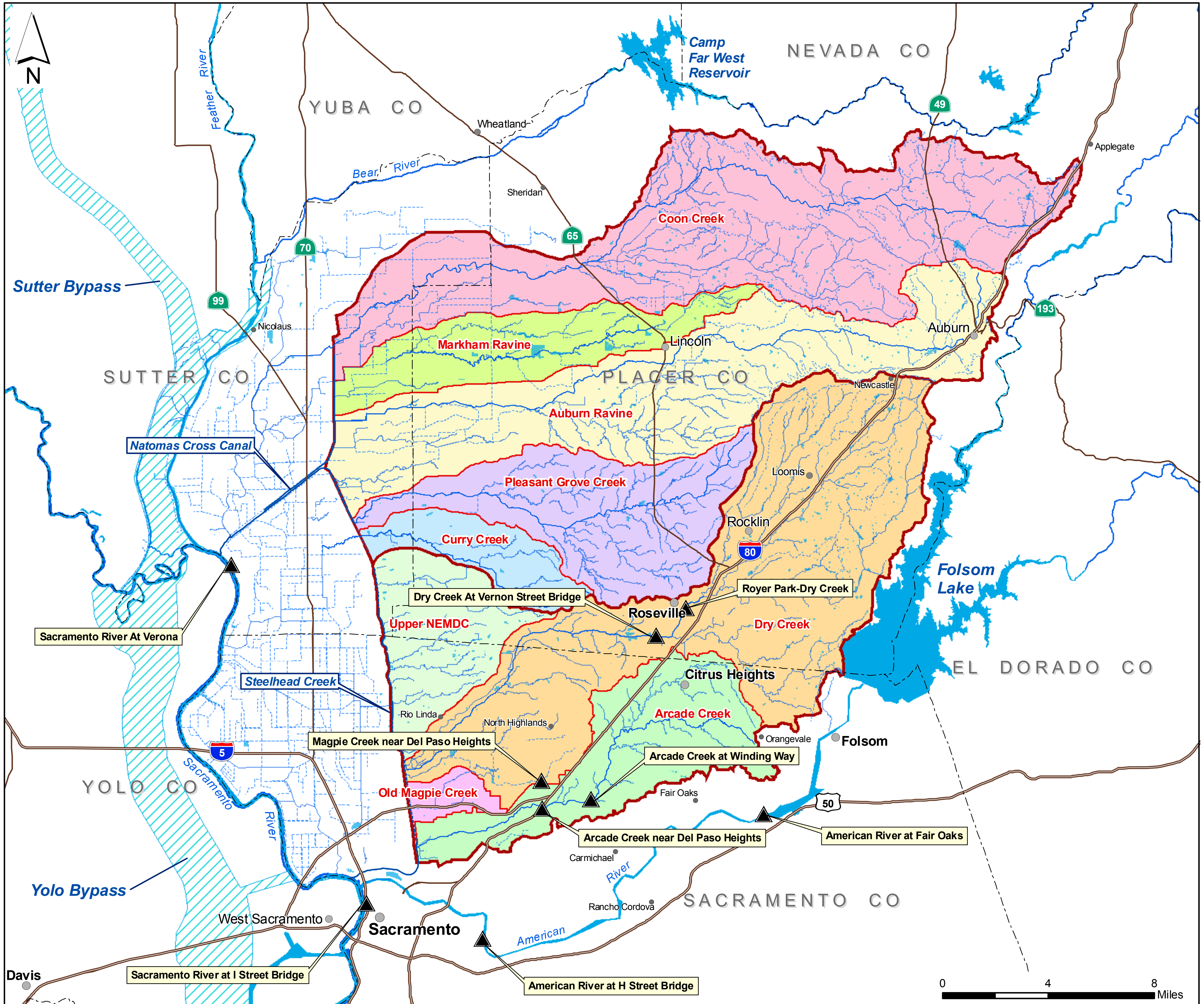
12. Memorandum: NCC/SHC Coincident-Frequency Study: Exposition of Analytical Procedures. Prepared by David Ford Consulting Engineers, Inc. September 10, 2008.

13. California Data Exchange Center (CDEC), the access point to the California Department of Water Resources' operation hydrologic data at: <http://cdec.water.ca.gov>.

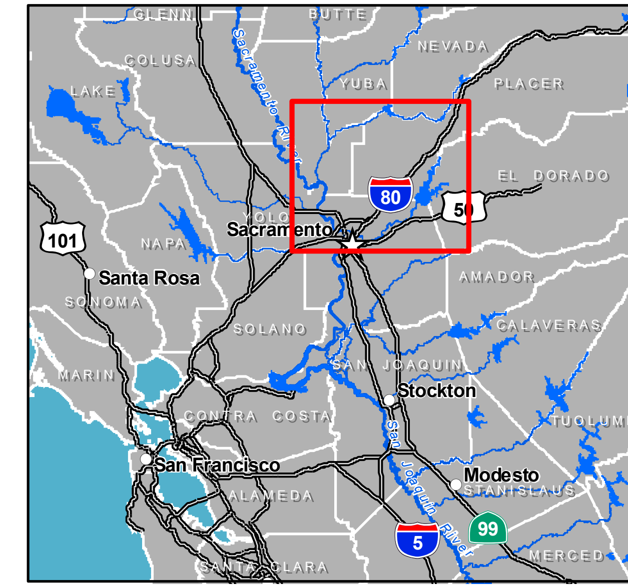
14. Memorandum: Cross-Correlation Analysis Results for NCC/SHC Coincident-Frequency Study. Prepared by David Ford Consulting Engineers, Inc. April 17, 2008.

15. Rainfall Depth-Duration Frequency Analysis for California Rain Gages. Mr. James Goodridge, retired State of California climatologist. Revised 2005.

16. Memorandum: NCC/SCH Coincident Frequency Study: Computational Results. Prepared by David Ford Consulting Engineers, Inc. September 10, 2008.



Prepared by J.S.M



Legend

- Stream Gage
- Watershed
- Sub-Watershed
- Lake or Major River
- River or Stream
- Canal or Ditch
- Freeway
- Highway
- County Boundary
- City or Town

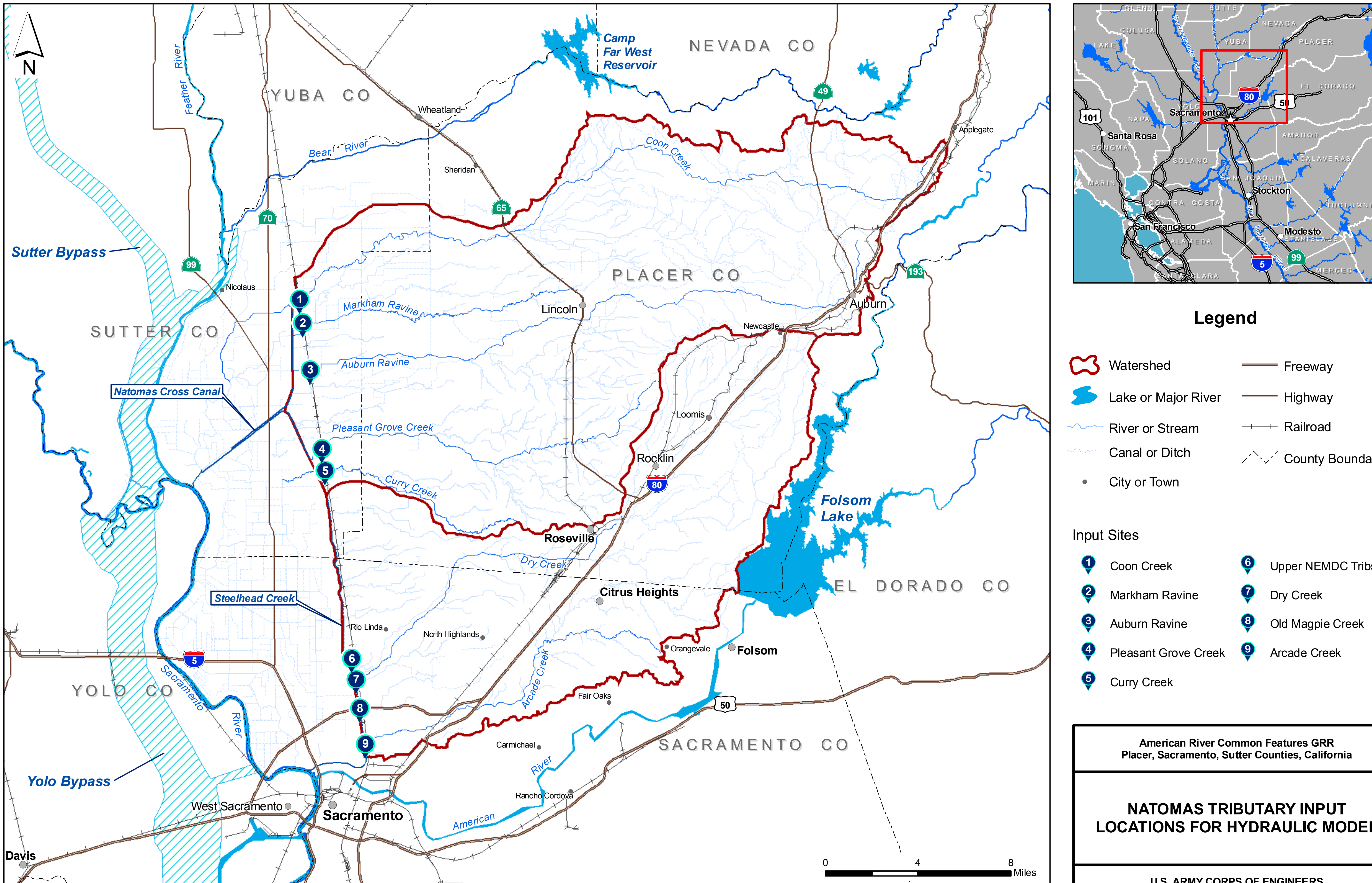
American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

GENERAL MAP

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008

PLATE 1



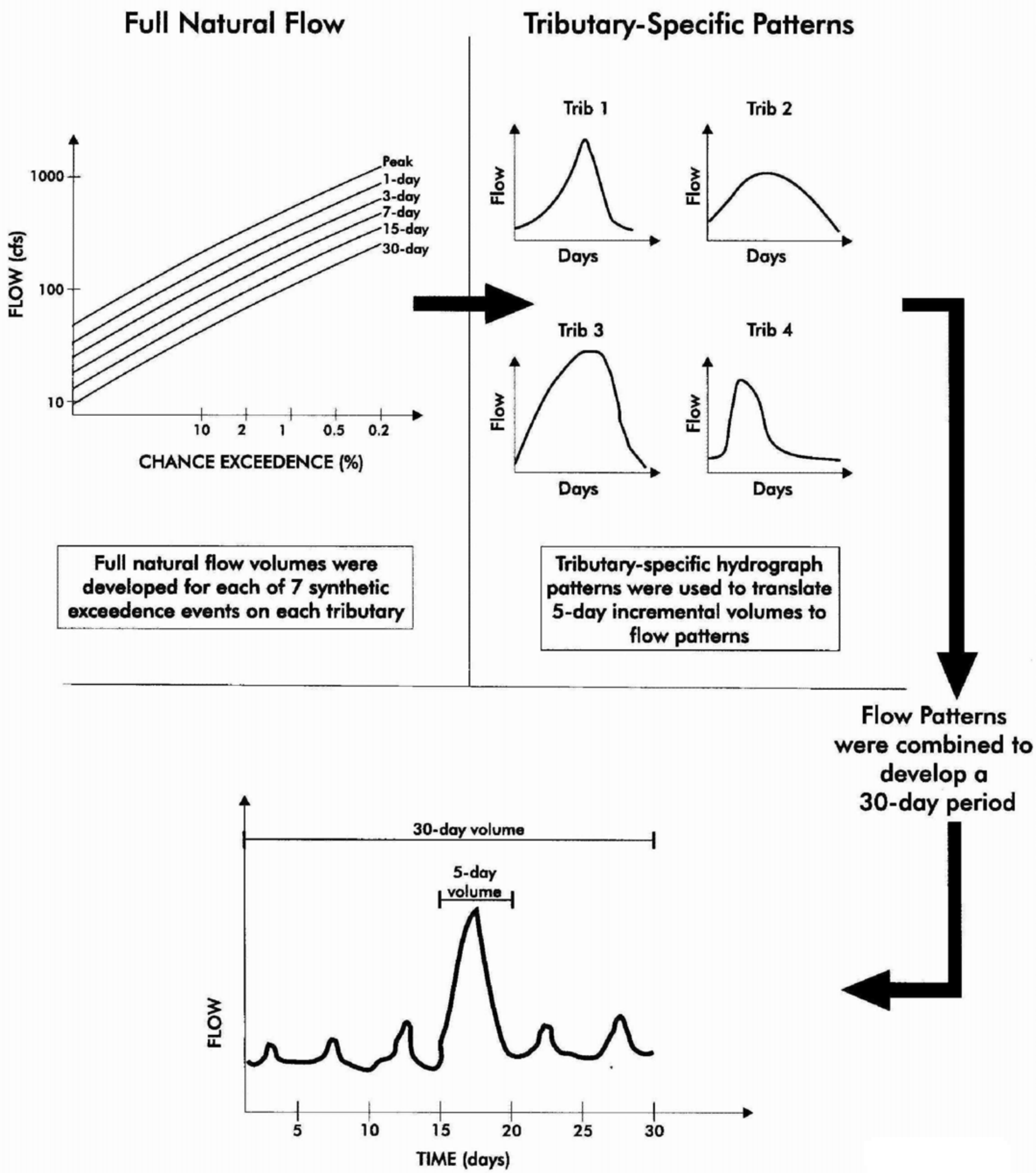
American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

NATOMAS TRIBUTARY INPUT LOCATIONS FOR HYDRAULIC MODEL

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008

PLATE 2



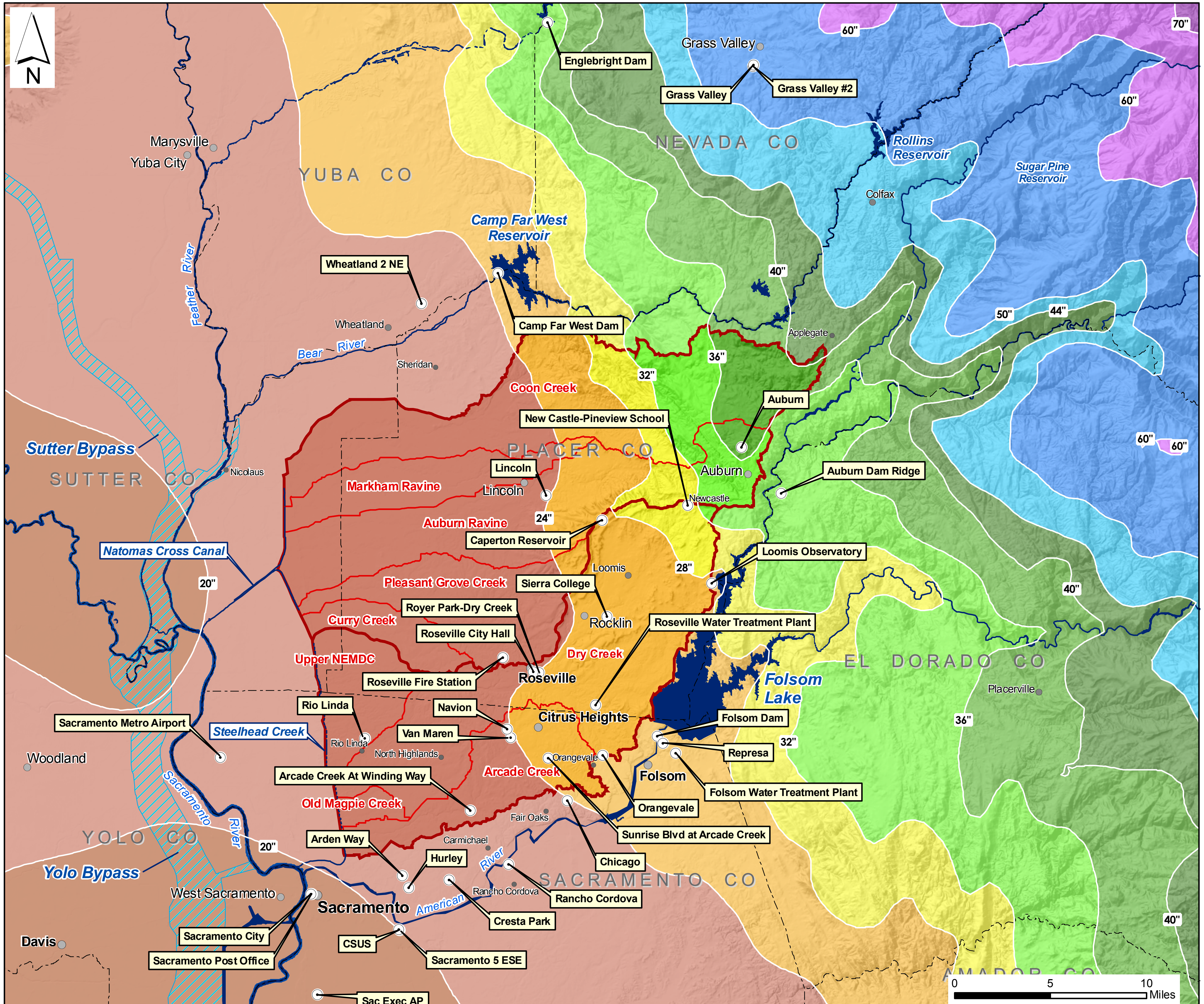
Resulting 30-day hydrographs for each of the 7 exceedence events on each tributary

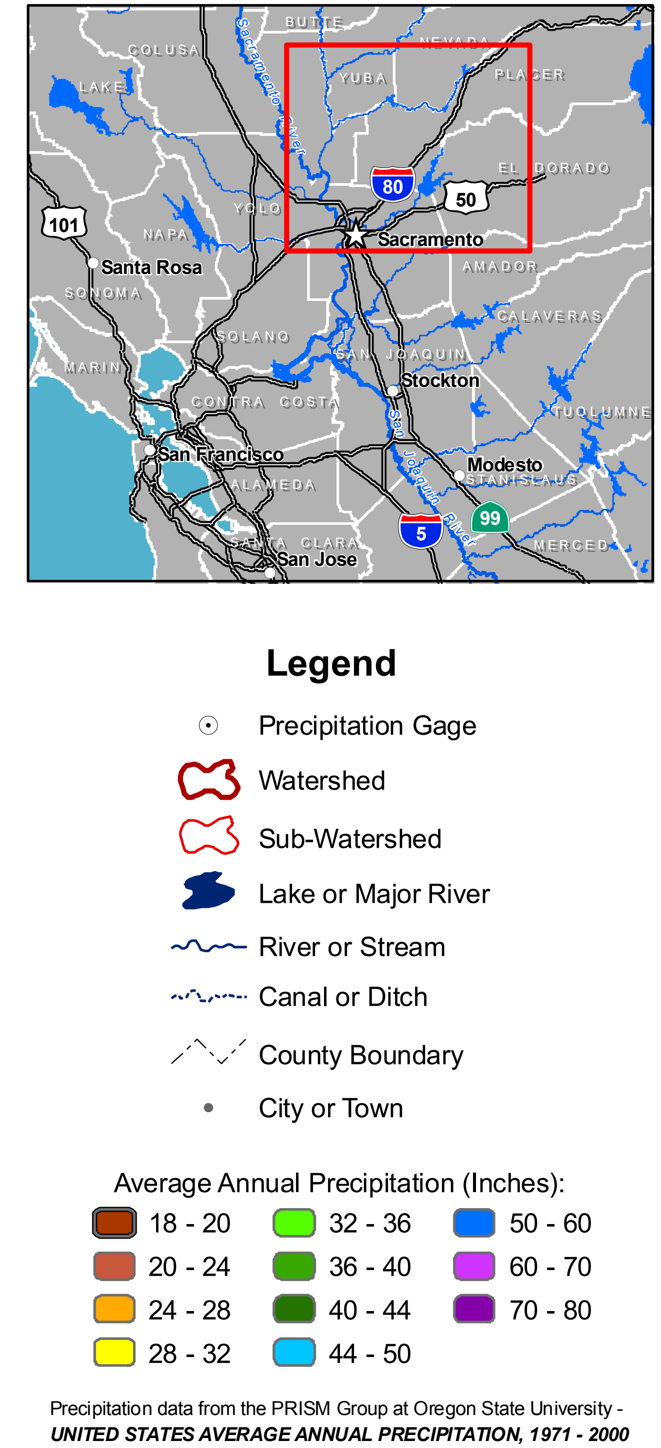
Sacramento & San Joaquin River Basins Comprehensive Study

HYDROGRAPH CONSTRUCTION

U.S. Army Corps of Engineers
Reclamation Board, State of California

July 2002





Prepared by J.S.M

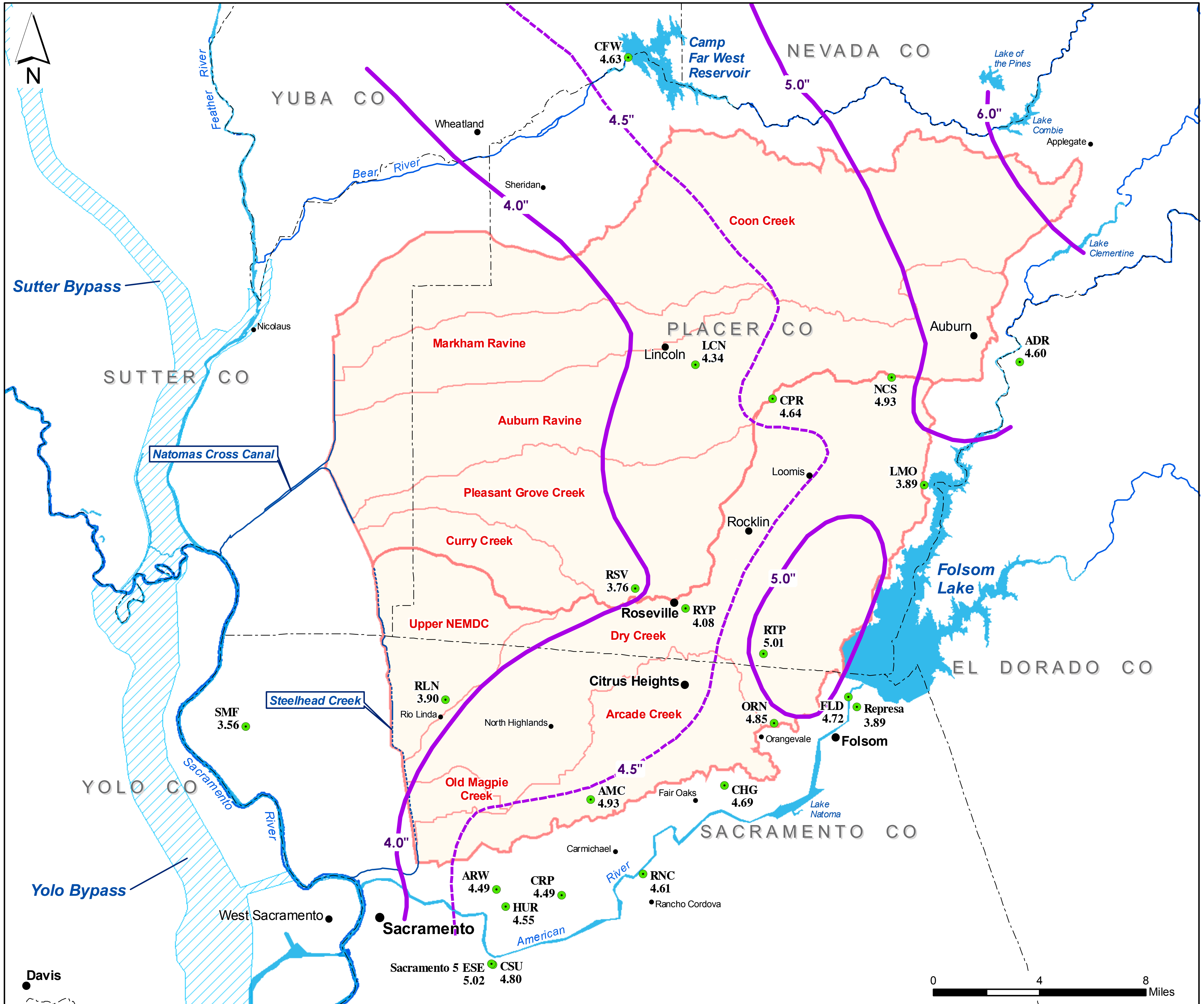
American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

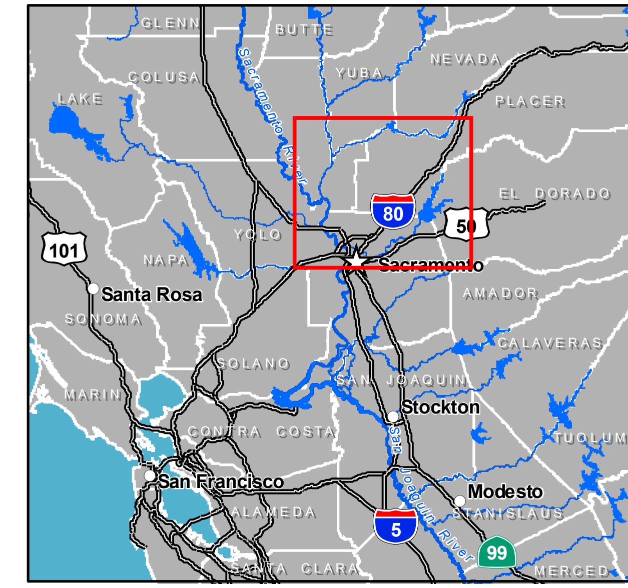
AVERAGE ANNUAL PRECIPITATION

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008

PLATE 4





Legend

- Precip Gage (Event Total in inches)
- Isohyet
- Watershed
- Sub-Watershed
- Lake or Major River
- River or Stream
- County Boundary
- City or Town

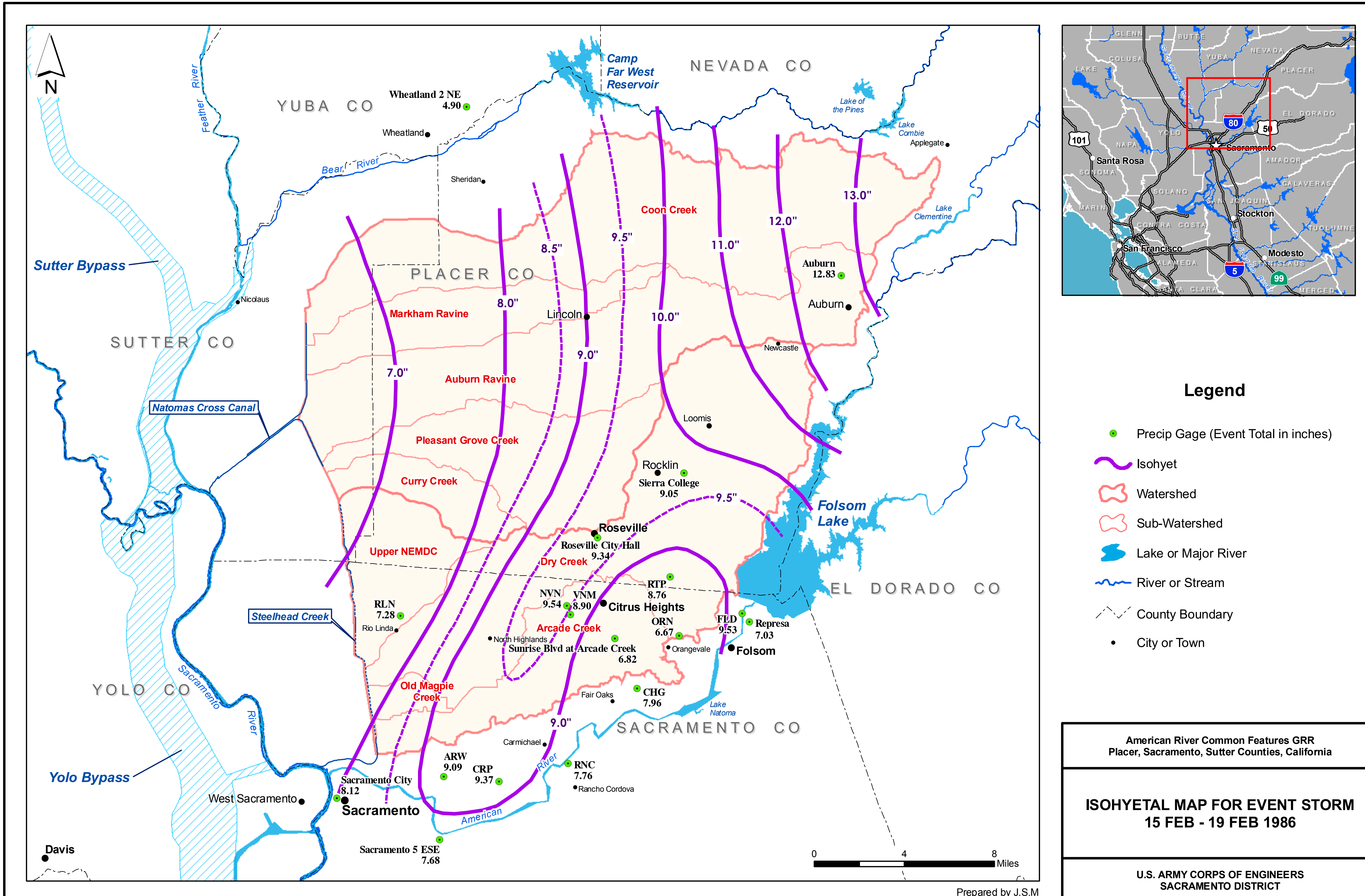
American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

ISOHYETAL MAP FOR EVENT STORM 30 DEC 2005 - 2 JAN 2006

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008

PLATE 5



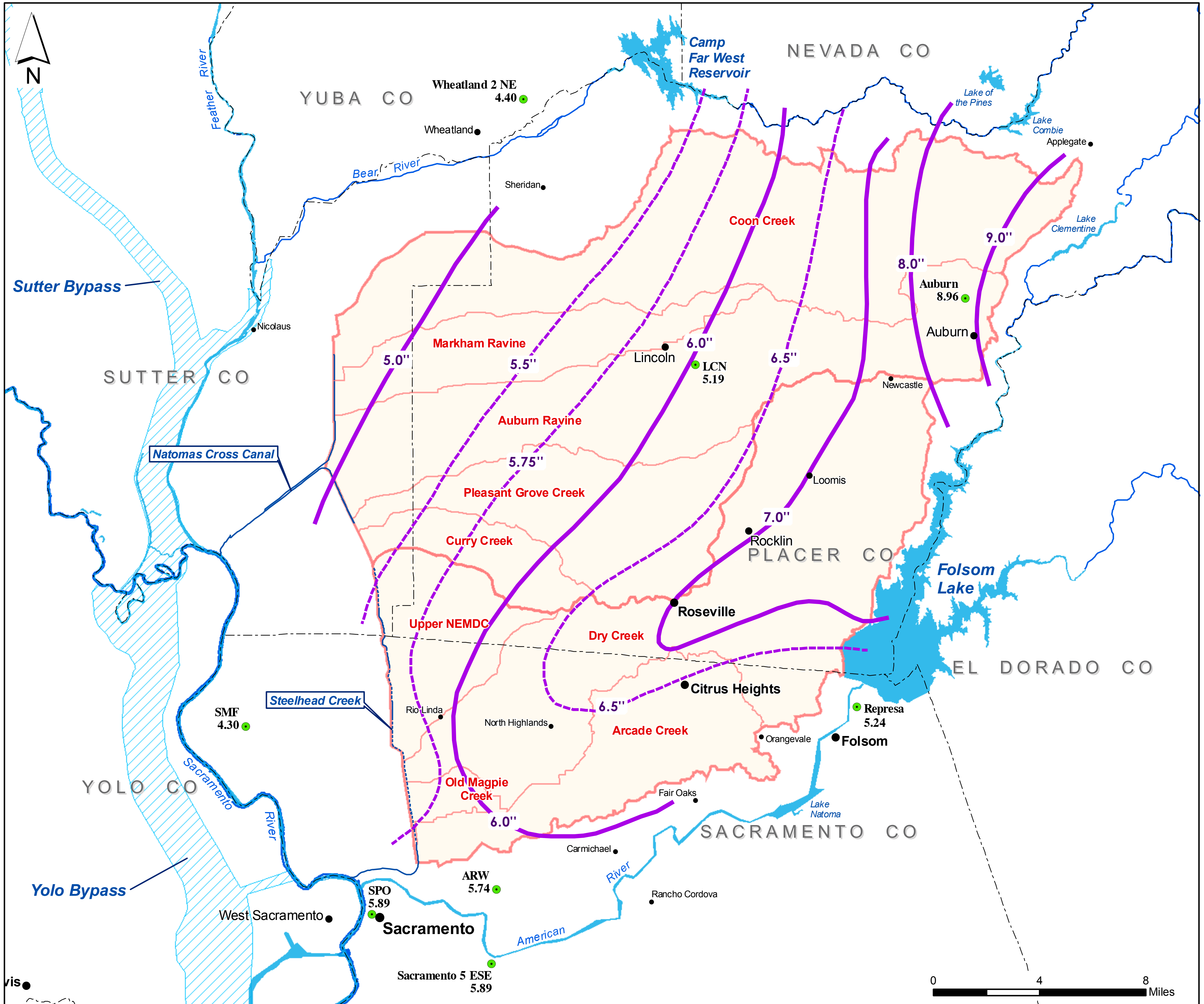
American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

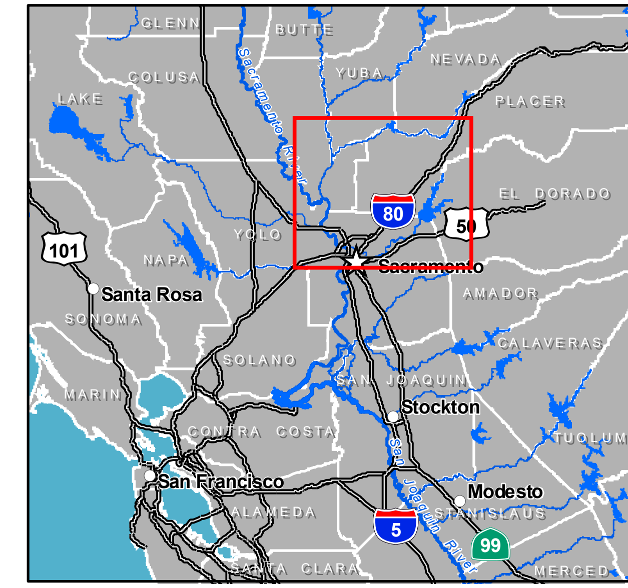
ISOHYETAL MAP FOR EVENT STORM
15 FEB - 19 FEB 1986

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008

PLATE 6



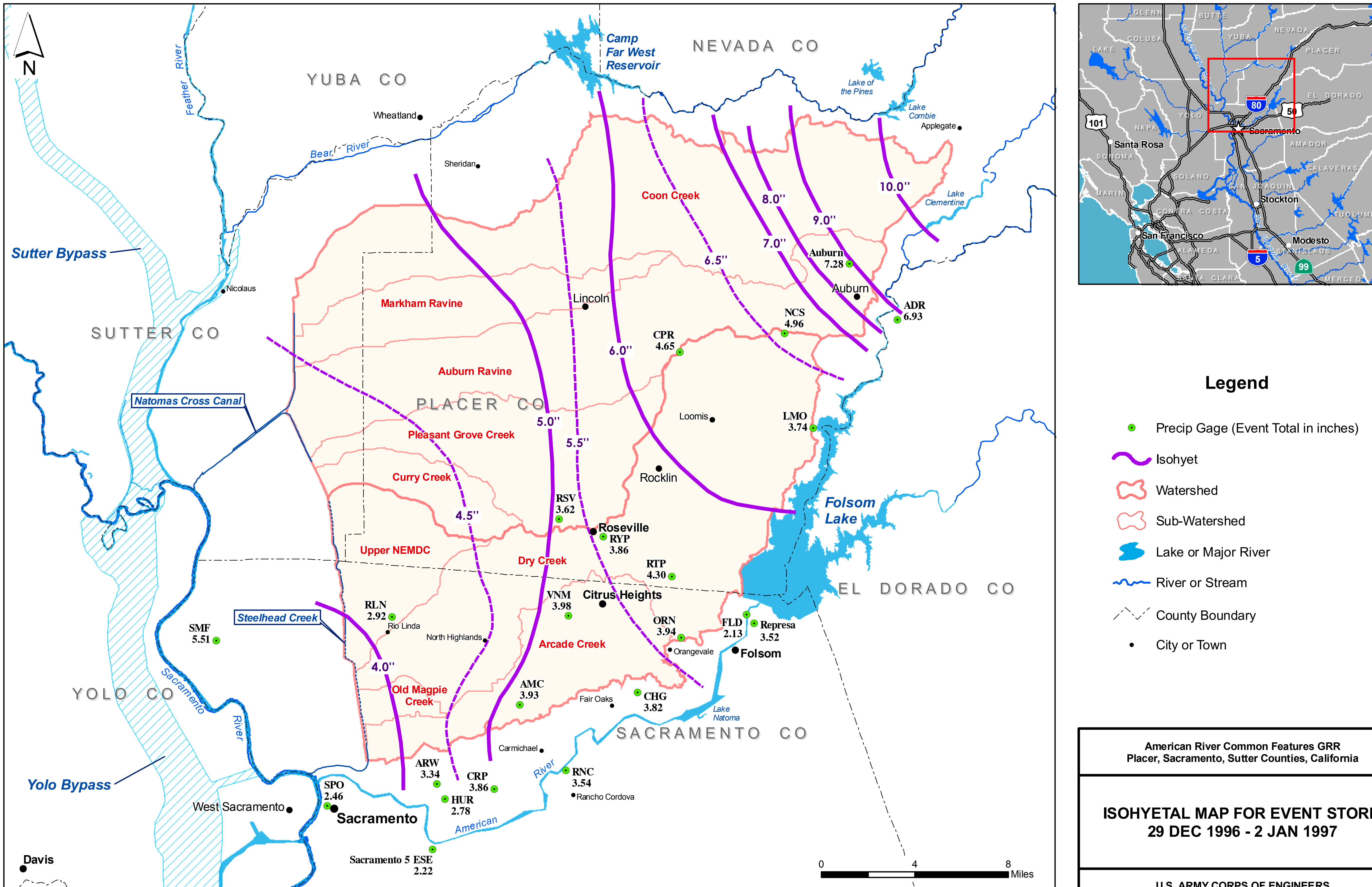


American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

ISOHYETAL MAP FOR EVENT STORM
8 JAN - 12 JAN 1995

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008



American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

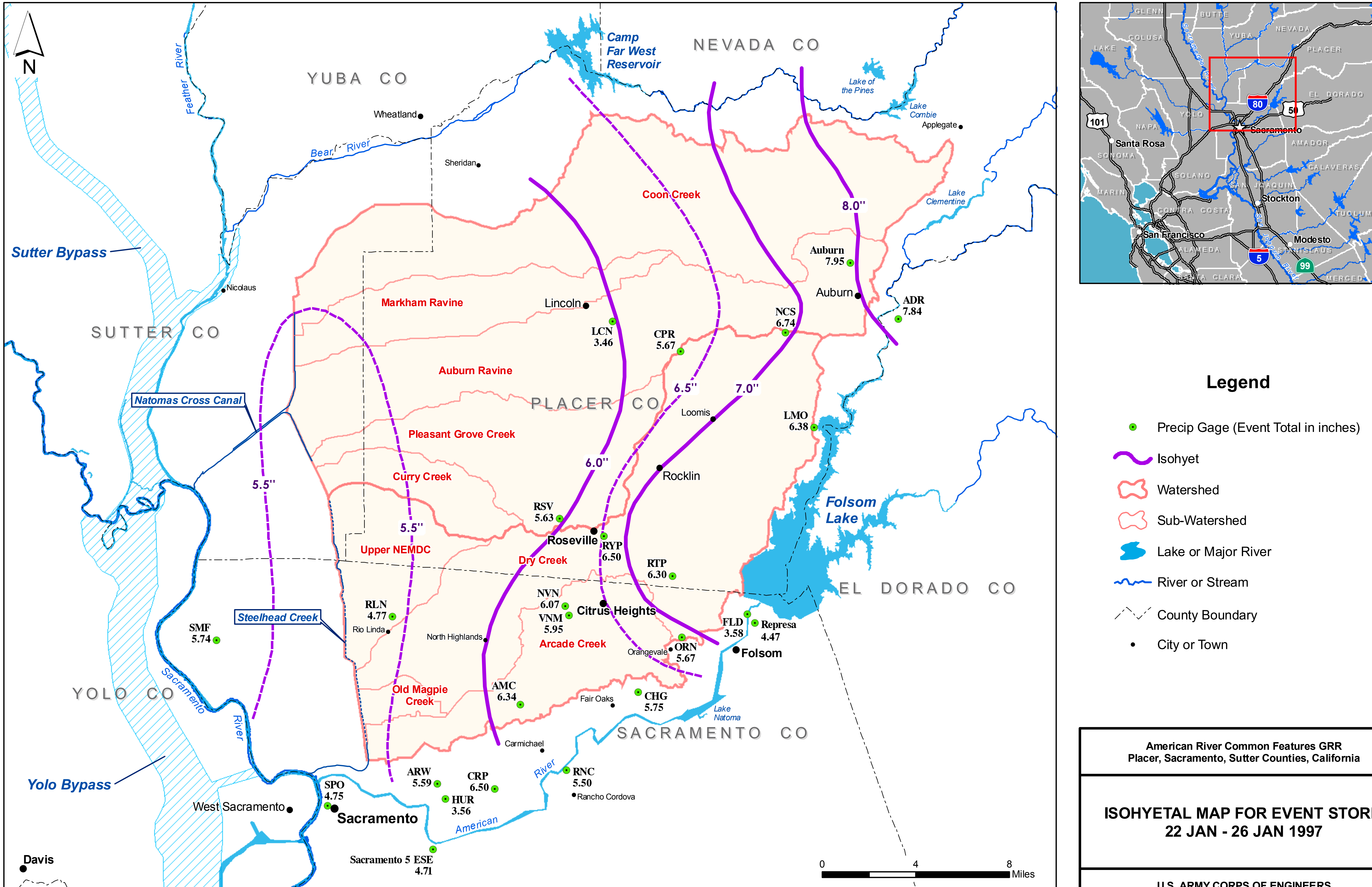
ISOHYETAL MAP FOR EVENT STORM
29 DEC 1996 - 2 JAN 1997

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Prepared by J.S.M

SEP 2008

PLATE 8

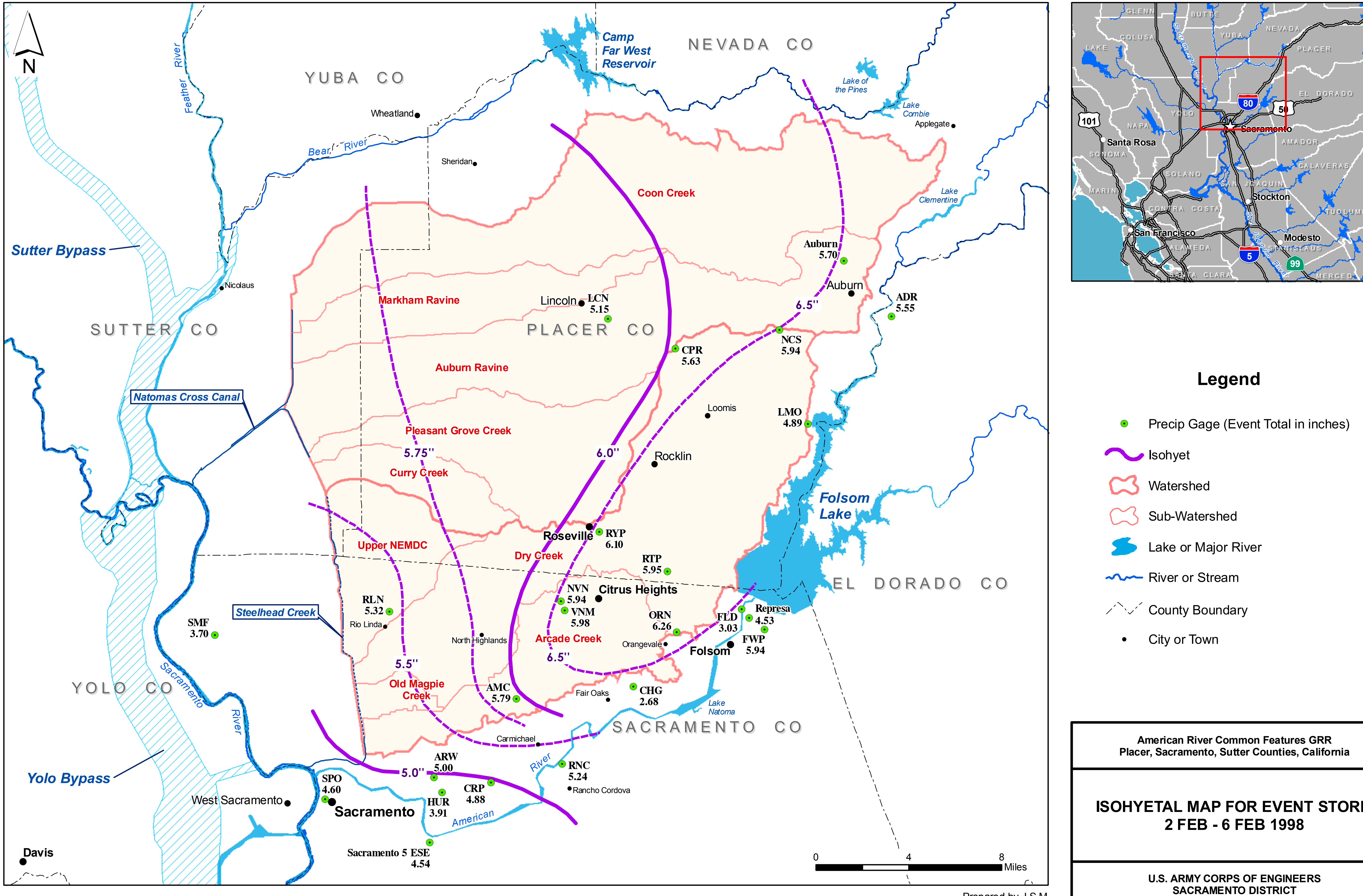


American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

ISOHYETAL MAP FOR EVENT STORM
22 JAN - 26 JAN 1997

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008 PLATE 9

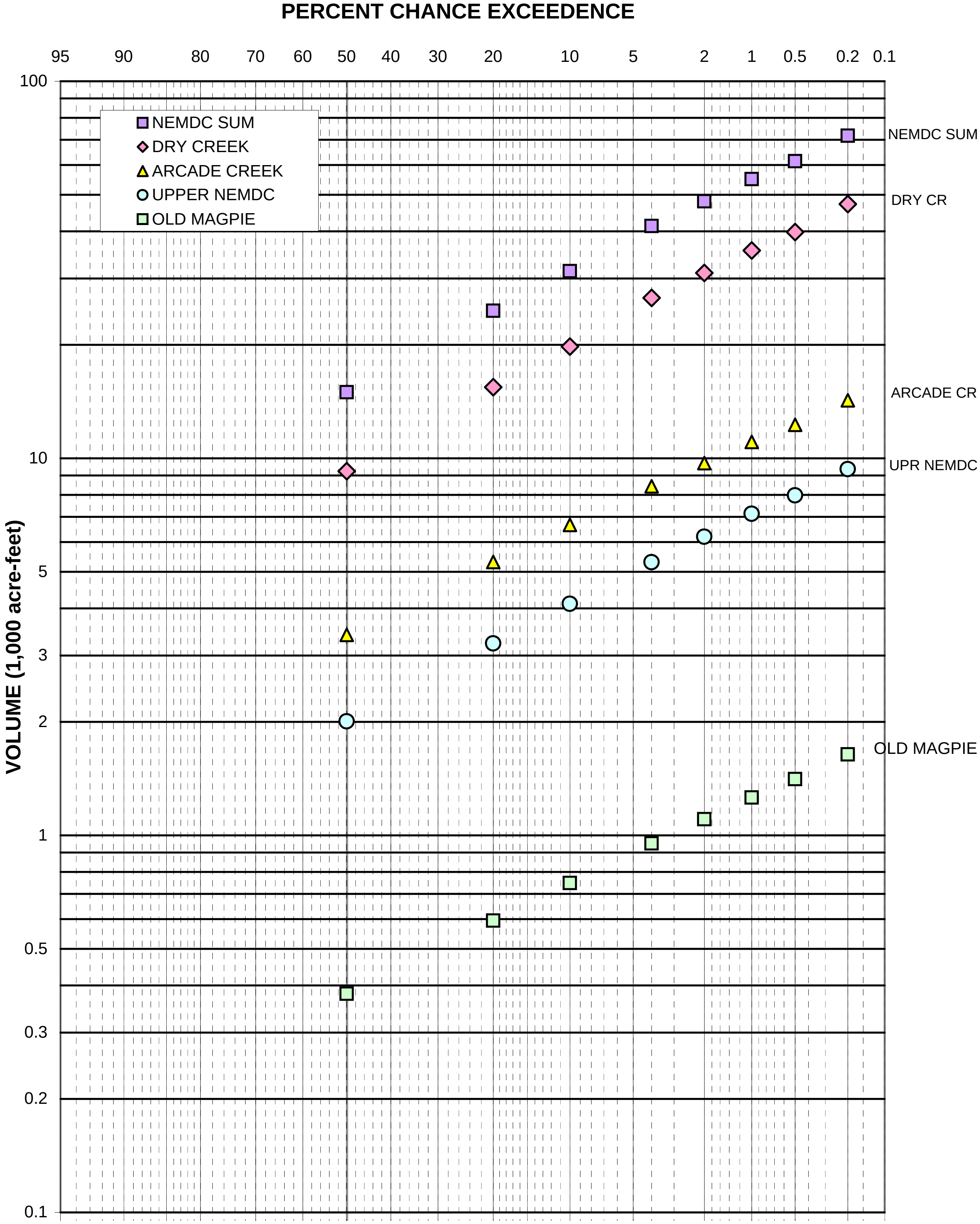


American River Common Features GRR
Placer, Sacramento, Sutter Counties, California

ISOHYETAL MAP FOR EVENT STORM
2 FEB - 6 FEB 1998

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008 PLATE 10



NOTES:

Dry Creek 116.48 sq. mi.

Arcade Creek 40.14 sq. mi.

Upper NEMDC 27.13 sq. mi.

Old Magpie Sum 4.57 sq. mi.

NEMDC Sum 188.32 sq. mi.

AMERICAN RIVER COMMON FEATURES GRR

PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

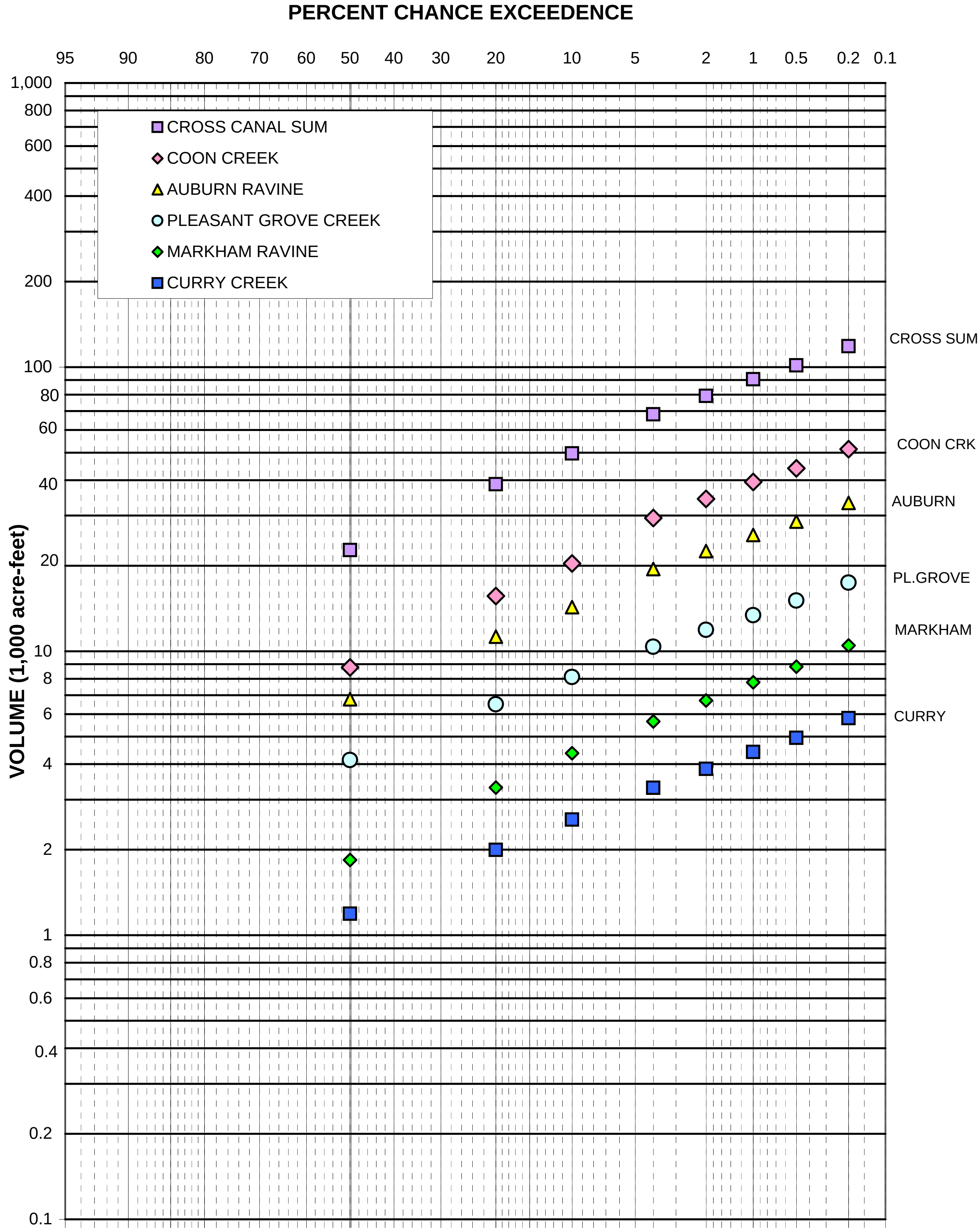
5-DAY VOLUME FREQUENCY CURVES
STEELHEAD CREEK DRAINAGE

U.S. ARMY CORPS OF ENGINEERS

SACRAMENTO DISTRICT

Developed by LLW and prepared by JLB

SEP 2008



NOTES:

Coon Creek 112.61 sq. mi.

Auburn Ravine 79.97 sq. mi.

Pleasant Grove Creek 46.69 sq. mi.

Markham Ravine 32.36 sq. mi.

Curry Creek 16.59 sq. mi.

Cross Canal Sum 288.22 sq. mi.

Developed by LLW and prepared by JLB

AMERICAN RIVER COMMON FEATURES GRR

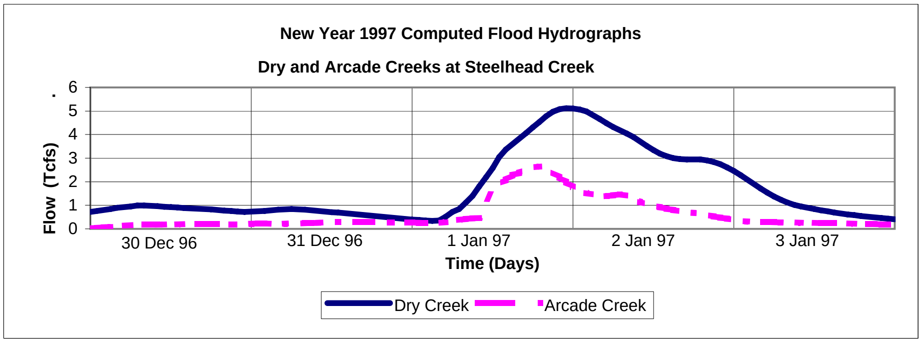
PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

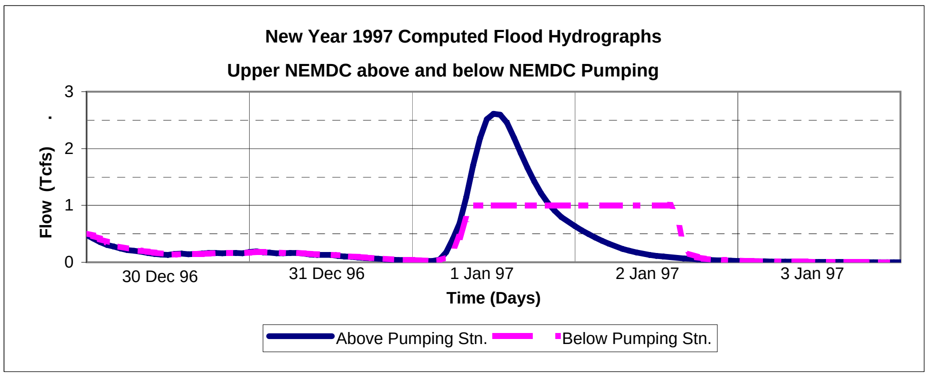
5-DAY VOLUME FREQUENCY CURVES
CROSS CANAL TRIBUTARIES

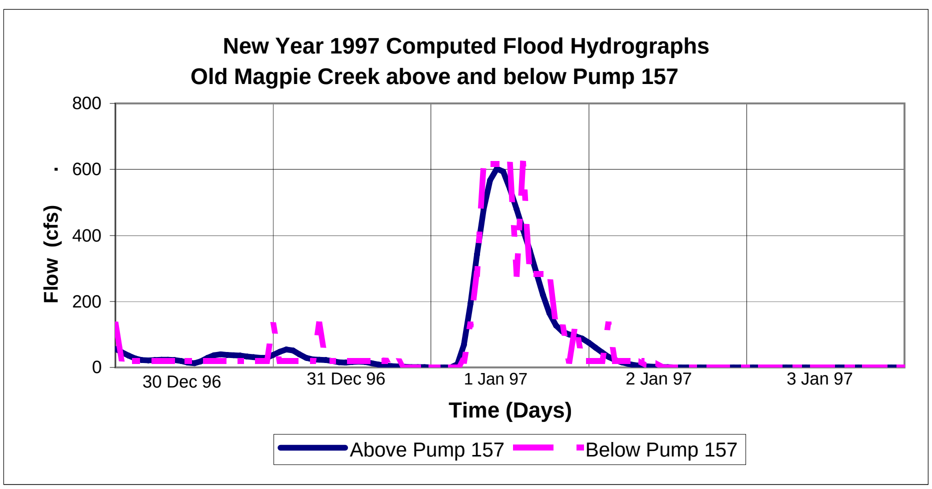
U.S. ARMY CORPS OF ENGINEERS

SACRAMENTO DISTRICT

SEP 2008







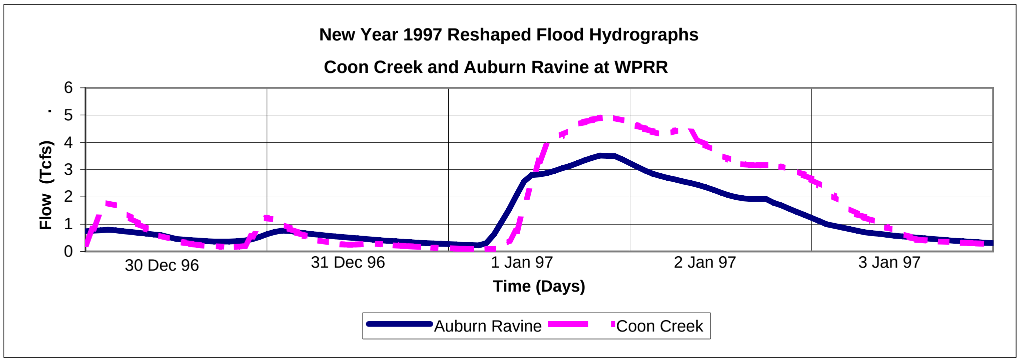
AMERICAN RIVER COMMON FEATURES GRR
PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

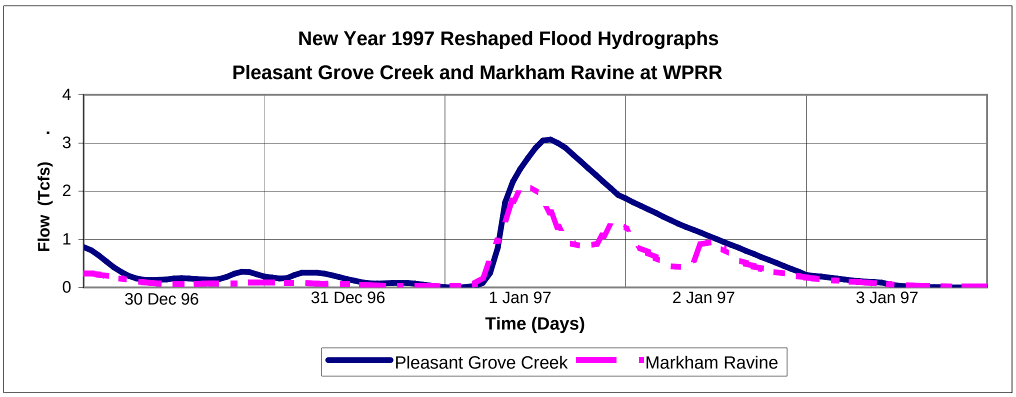
COMPUTED AND RESHAPED HYDROGRAPHS FOR FOR NEW YEAR 1997 FLOOD EVENT STEELHEAD CREEK TRIBUTARIES

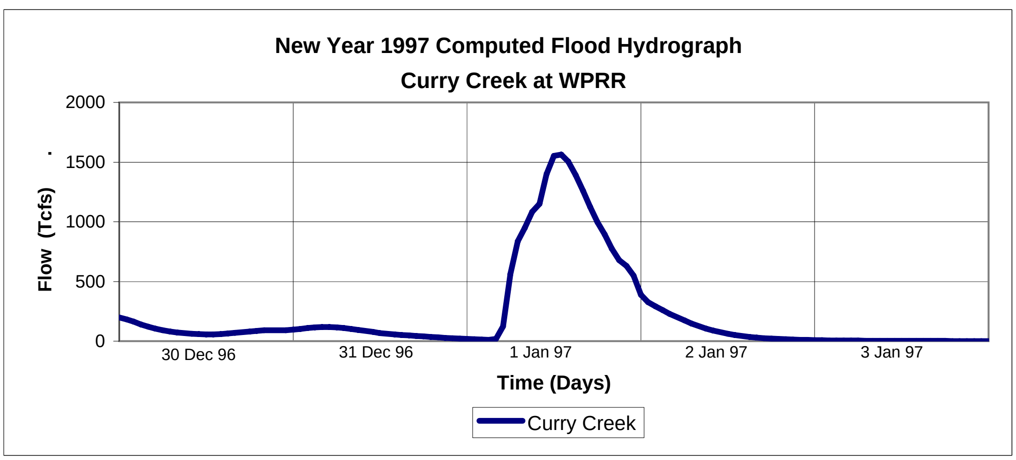
U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008

PLATE 13-A





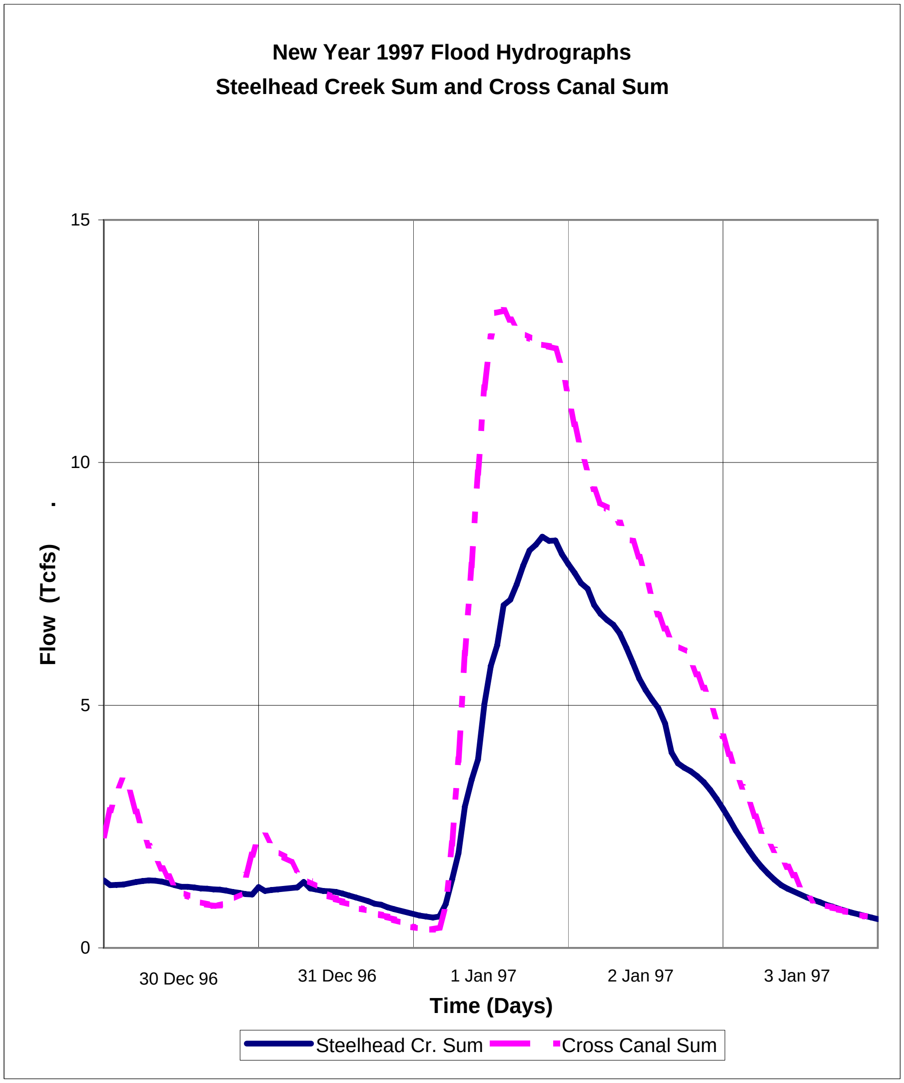


AMERICAN RIVER COMMON FEATURES GRR
PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

COMPUTED AND RESHAPED HYDROGRAPHS FOR FOR NEW YEAR 1997 FLOOD EVENT NATOMAS CROSS CANAL TRIBUTARIES

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008 PLATE 13-B



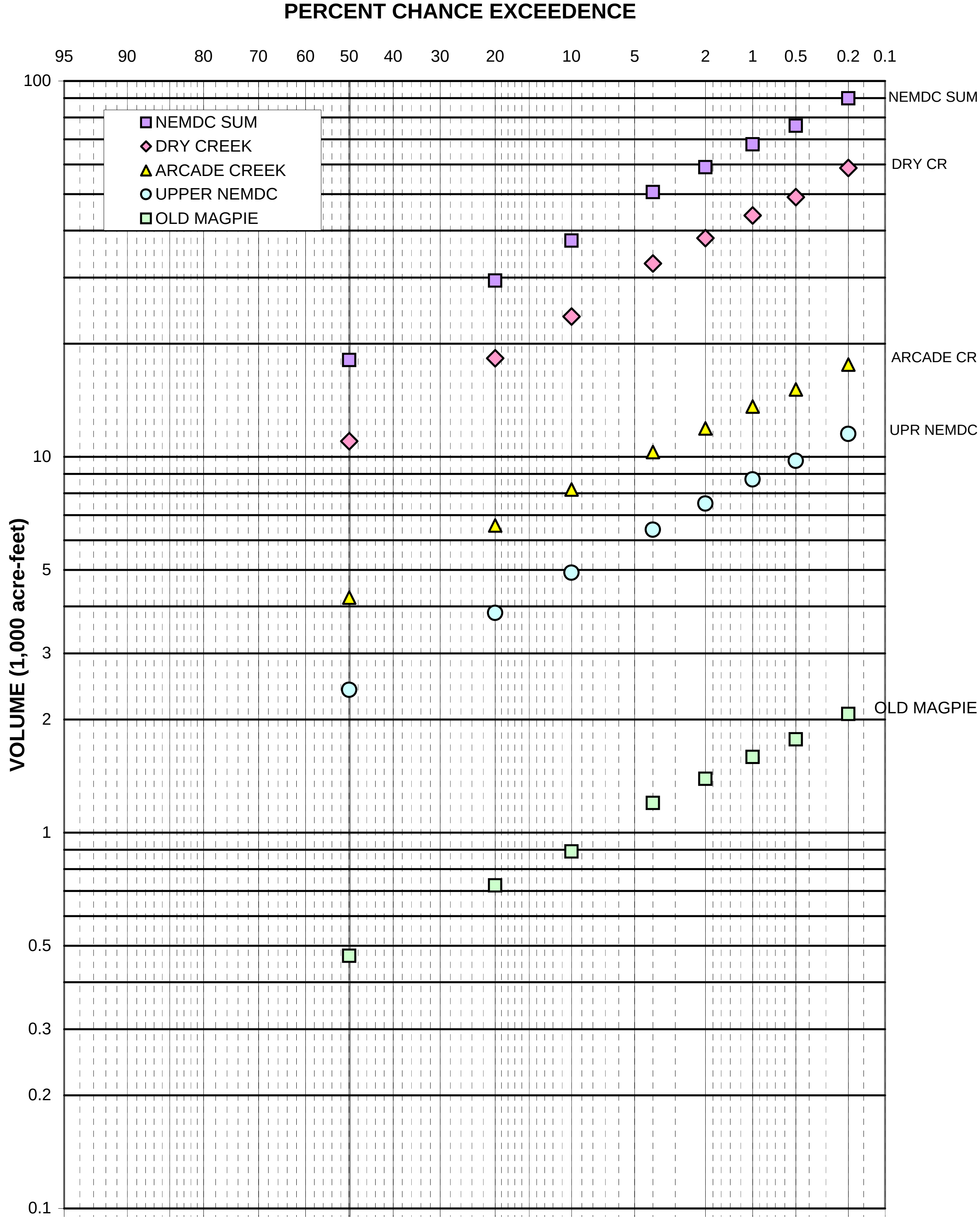
AMERICAN RIVER COMMON FEATURES GRR
PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

COMPUTED AND RESHAPED HYDROGRAPHS FOR FOR NEW YEAR 1997 FLOOD EVENT STEELHEAD CREEK SUM NATOMAS CROSS CANAL SUM

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008

PLATE 13-C



NOTES:

Dry Creek 116.48 sq. mi.

Arcade Creek 40.14 sq. mi.

Upper NEMDC 27.13 sq. mi.

Old Magpie Sum 4.57 sq. mi.

NEMDC Sum 188.32 sq. mi.

AMERICAN RIVER COMMON FEATURES GRR

PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

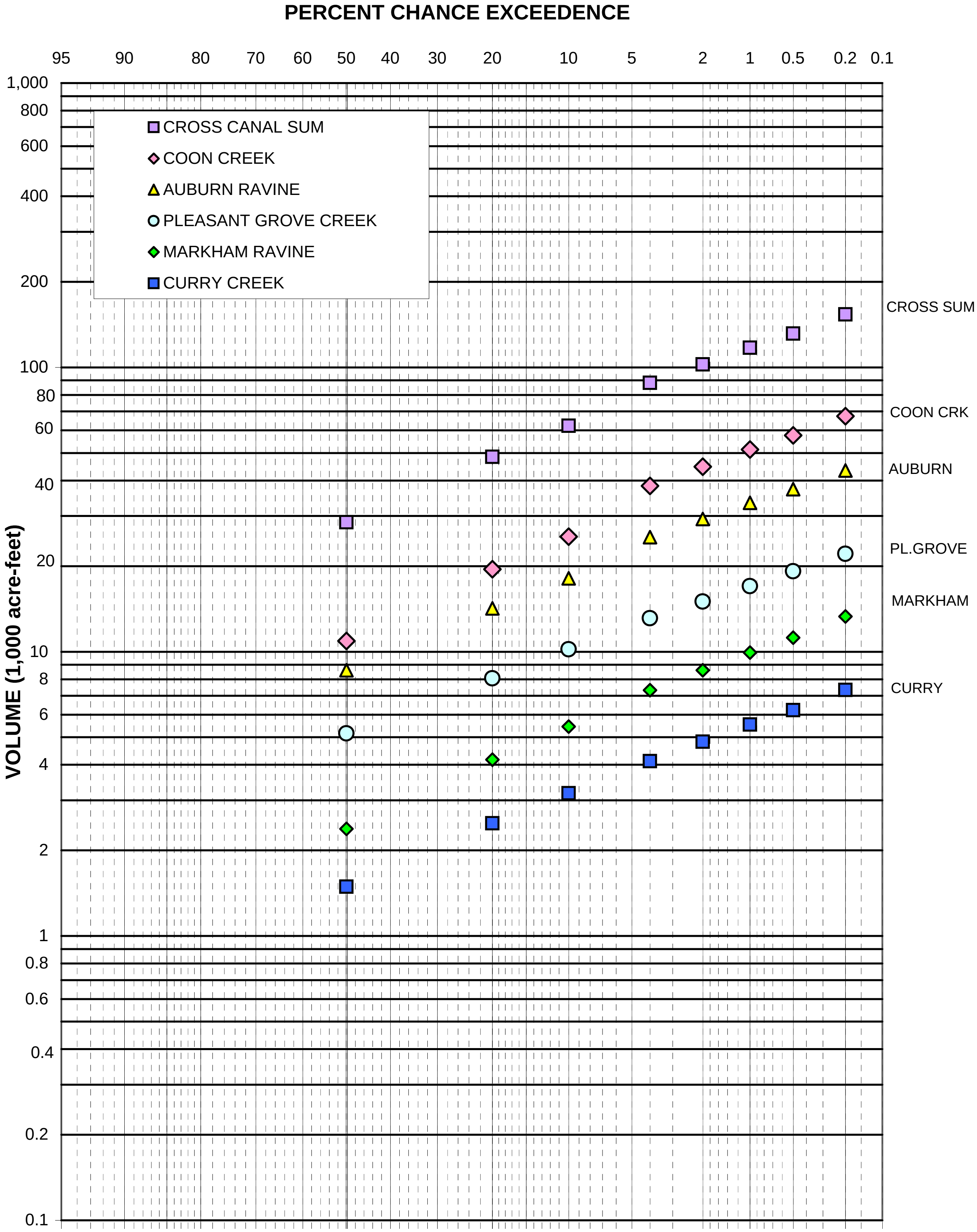
10-DAY VOLUME FREQUENCY CURVES STEELHEAD CREEK DRAINAGE

U.S. ARMY CORPS OF ENGINEERS

SACRAMENTO DISTRICT

Developed by LLW and prepared by JLB

SEP 2008



NOTES:

Coon Creek 112.61 sq. mi.
Auburn Ravine 79.97 sq. mi.
Pleasant Grove Creek 46.69 sq. mi.
Markham Ravine 32.36 sq. mi.
Curry Creek 16.59 sq. mi.
Cross Canal Sum 288.22 sq. mi.

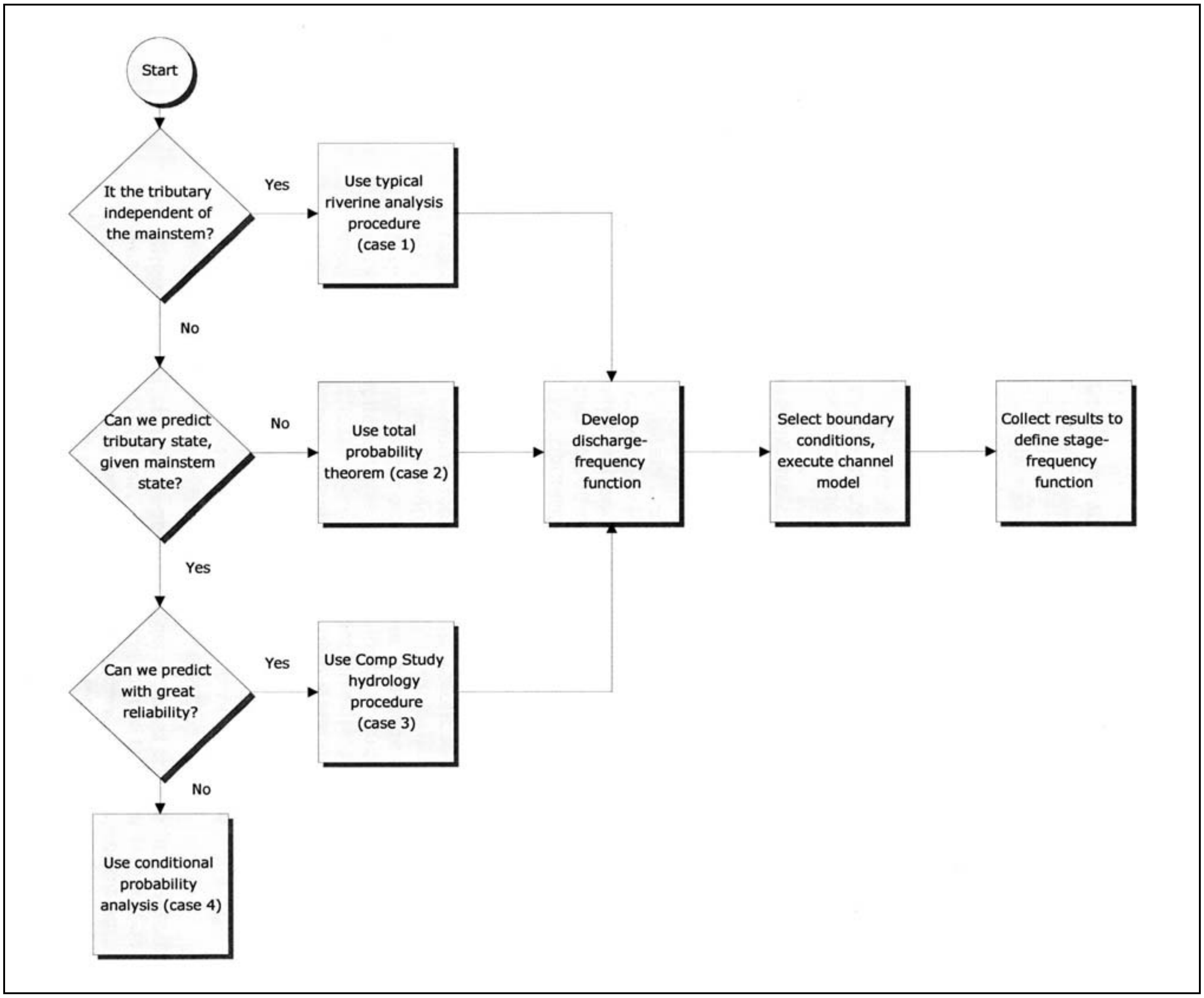
Developed by LLW and prepared by JLB

AMERICAN RIVER COMMON FEATURES GRR
PLACER, SACRAMENTO, SUTTER COUNTIES, CALIFORNIA

10-DAY VOLUME FREQUENCY CURVES
CROSS CANAL TRIBUTARIES

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

SEP 2008



Overview Flowchart for Tributary Analysis Procedure

American River Watershed
Common Features Project
Natomas Post-Authorization Change Report

American River Hydrology
&
Folsom Dam Reservoir Operations

APPENDIX B2

DRAFT

This page left blank intentionally.

DRAFT
TABLE OF CONTENTS

FIGURES

TABLES

DRAFT

AMERICAN RIVER HYDROLOGY & FOLSOM DAM RESERVOIR OPERATIONS

A-1 Purpose

The scope of this General Reevaluation Report (GRR) covers the greater Sacramento area, which includes the Lower American River and the Natomas Basin. Hydraulic and geotechnical studies of the area have been on-going and have already identified many issues (e.g. seepage, erosion, vegetation, etc) which could lead to levee failure. The latest findings indicate that the Sacramento area is still highly susceptible to flooding due to levee failure even with all the authorized repairs and improvements. The economic analyses will evaluate the flood risk and cost benefit of fixing the identified problems. This write-up covers the development of the Folsom Dam discharge hydrographs provided to Hydraulic Design for the floodplain delineation efforts and the development of the hydrologic data inputs provided to Economics for the HEC-FDA model. The economic analysis will evaluate the extent of the damage caused by levee failures within the basin. Two scenarios were evaluated for the existing condition: the without-project (WO) condition and the future without-project condition, which is labeled as the no-action (NA) condition. These scenarios provide the information needed to perform an incremental analysis of the state of the levees at various levels of improvement (objective release 115,000 cfs, 145,000 cfs, or 160,000 cfs) and of the affect of the levee state when combined with the other authorized project components. Generally, these scenarios are hypothetical and would not be built or implemented as stand-alone projects. The reservoir routings covered herein were developed for planning purposes, only. All reservoir elevations provided herein use the NGVD29 vertical datum.

A-2 Background

As an interim means of reducing flood risk, Congress authorized the American River Common Features Project under Section 101(a) (1) of the Water Resources Development Act (WRDA) 1996. The features that were common to three candidate plans identified by the Corps, SAFCA, and the State of California Reclamation Board (State Reclamation Board) in the 1996 Supplemental Information Report (SIR) were covered in the authorization. The levee repairs and improvements included:

- 24 miles of slurry wall in the levees along the lower American River
- 12 miles of levee modifications along the east bank of the Sacramento River downstream from the Natomas Cross Canal
- Installation of three telemeter streamflow gages upstream from the Folsom Reservoir
- Modification to the flood warning system along the lower American River
- Raising the left bank of the non-Federal levee upstream of Mayhew Drain for a distance of 4,500 feet by an average of 2.5 feet
- Raising the right bank of the American River levee from 1,500 feet upstream to 4,000 feet downstream of the Howe Avenue Bridge by an average of 1 foot
- Modifying the south levee of the Natomas Cross Canal for a distance of 5 miles to ensure that the south levee is consistent in level with the level of protection provided by the authorized levee along the east bank of the Sacramento River
- Modifying the north levee of the Natomas Cross Canal for a distance of 5 miles to ensure the height of the levee is equivalent to the height of the south levee as authorized (above)
- Installing gates to the existing Mayhew Drain culvert and pumps to prevent backup of floodwater on the Folsom Boulevard side of the gates
- Installing a slurry wall in the north levee of the American River from the east levee of the Natomas east Main Drain upstream for a distance of approximately 1.2 miles
- Installing a slurry wall in the north levee of the American River from 300 feet west of Jacob Lane north for a distance of approximately 1 mile to the end of the existing levee

Section 366 of WRDA 1999 authorized more improvements which included the raising and strengthening of the levees along the American River and additional work in Natomas.

The Common Features GRR was initiated because the economic basis for the original authorization has changed. The Common Features Project has been subject to significant cost increases due to major design modifications and to additional work proposals. Further investigations into additional modes of levee failure (i.e. slope stability, seepage, underground utilities and vegetative growth and long term degradation effects that include erosion) have revealed that in order to ensure the integrity of the levee system, while sustaining 160,000 cfs, much more work is required than was originally identified under WRDA 96 and WRDA 99. According to *Appendix D – Hydraulic Technical Documentation of the F3 Document*, the hydraulic modeling and geotechnical studies have identified potential seepage issues on both the Sacramento and American Rivers and erosion issues on the American River. In order to better describe the potential impact of flooding within the entire Sacramento area, the scope of the Common Features project must be expanded to consider the risk of levee failure along the Sacramento River, American River and the Natomas Basin. This system-wide approach provides a more comprehensive view of the flood risk to the Sacramento metropolitan area.

Congress also authorized the “Folsom Modifications Project” under Section 101 of WRDA 1999 and the “Folsom Dam Raise Project” in 2003. Although these projects were authorized independently, the project performances are intertwined based on when the projects are assumed completed. Due to constructability issues with the “Folsom Modifications Project”, both the “Folsom Modifications Project” and the “Folsom Dam Raise Project” required reexamination. The Corps sought to combine the objectives of these two authorized projects with Reclamation’s dam safety project. This resulted in the Joint Federal Project (JFP), which met the flood damage reduction and dam safety objectives of the USACE, Reclamation, and the local sponsor. The ability of the downstream levees to handle 160,000 cfs is a key factor in achieving the following goals: 1) control the 1-in-200 year event by holding the release at 160,000 cfs (or less) and 2) control the PMF event while maintaining at least 3 ft of freeboard.

A-3 American River Hydrology

The Comprehensive Study data provides the majority of the input to the Hydraulic Design HEC-RAS model. The one exception is the data for the American River. Both the hydrology and routing tool for American River flows differ. Although the HEC-ResSim model built for the Comprehensive Study simulates system-wide operation for multiple reservoirs on the Sacramento River along with those on its major tributaries, the Folsom Dam Excel-based reservoir routing model provides the means necessary to examine Folsom Dam project features in more detail. For consistency, the same hydrology used in other American River studies was utilized for the Common Features GRR. See *Appendix A – Synthetic Hydrology Technical Documentation* for a discussion on the differences between the Comprehensive Study and the American River studies unregulated hydrographs for the American River.

A series of hypothetical inflow hydrographs (i.e. 50%-, 10%-, 4%-, 2%-, 1%-, 0.5%-, 0.2%-annual chance flood events) were developed for the flood risk management analyses. See **Figure A-1**. Design flood hydrographs can be patterned after historical or hypothetical events. In this instance, the flood hydrographs are patterned after the synthetic 2001 PMF event. Each hydrograph consists of multiple waves -- as would occur if a series of storms moved through the region. The sequencing of waves is an important aspect to consider when developing synthetic flood hydrographs. Antecedent waves could induce encroachment into the flood pool prior to the arrival of the main wave. This situation is most likely to occur when a project has limited release capability as under the existing project condition.

The selected hydrograph pattern is proportioned to match the annual maximum 3-day volume and peak for designated exceedance probabilities. The 3-day duration is considered the most critical within the American River basin. Past analyses has shown that the 3-day duration has the greatest impact on operation of the existing flood control system (Folsom Dam and the downstream levees), as well as plan formulation for the American River Basin and most other Sacramento Basin tributaries.

The flood volumes are obtained from a family of unregulated inflow frequency curves. The statistics used to generate these curves were last updated in 2004 using the statistical procedures and methodologies outlined in *Bulletin 17B, Guidelines for Determining Flood Flow Frequency* (United States Geologic Survey [USGS], 1982). *Rain Flood Flow Frequency Analysis, American River, California* (Corps, 2004) documents this process from start to finish beginning with preparation of the data and ending with development of the Log Pearson III statistics presented in **Table A-1.** The mean daily flow at the Fair Oaks gage downstream was used to develop the unregulated inflow for Folsom Dam. The drainage area between Fair Oaks and Folsom Dam does not generate a significant amount of local flow.

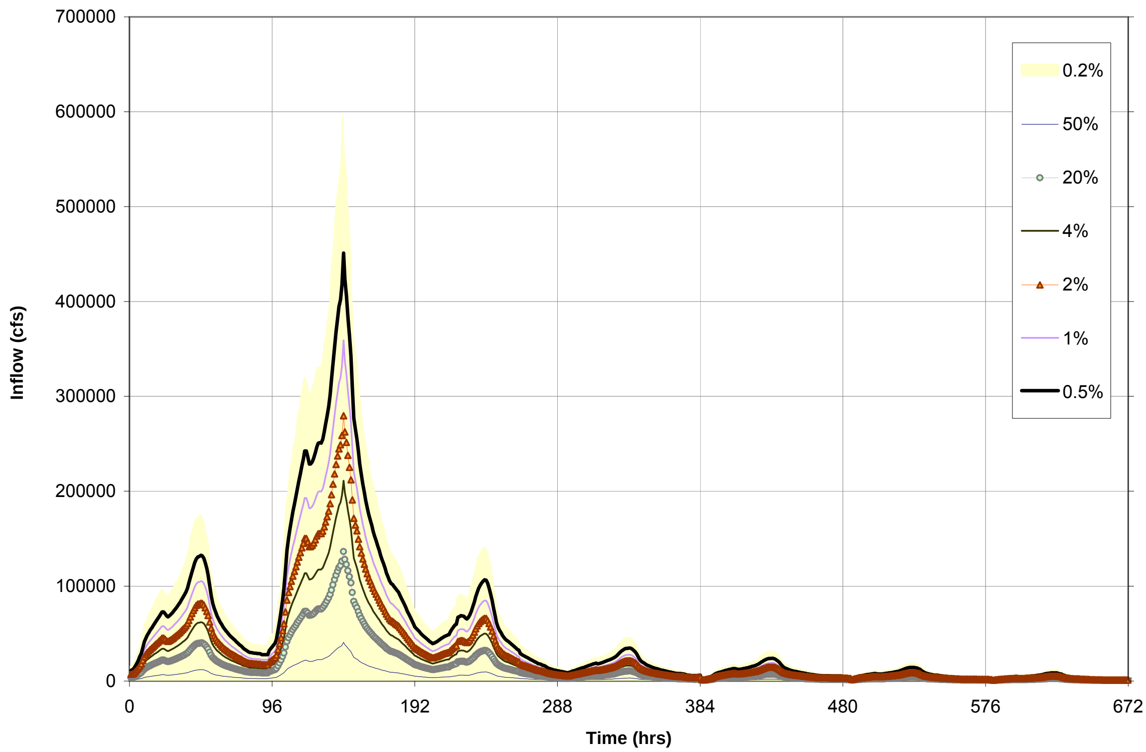


FIGURE A-1
FLOOD HYDROGRAPHS

The flood hydrographs above are based on a storm centered over the American River basin. Other storm centerings (i.e. Shanghai Bend, the mainstem of the Sacramento River) were considered to identify the conditions that would put the most stress on levee locations susceptible to failure. *Appendix A – Synthetic Hydrology Technical Documentation* contains a discussion regarding the development of the Comprehensive Study hydrographs based on the different storm centerings. The Comprehensive Study results were used to identify the coincident frequencies on the American River given a 50%-, 10%-, 4%-, 2%-, 1%-, 0.5%-, or 0.2%-annual chance flood event occurring elsewhere outside the American River basin. These coincident frequencies were used to develop two additional sets of flood hydrographs, one for the Shanghai Bend centering and another for the Sacramento River mainstem centering.

TABLE A-1: American River at Fair Oaks (1905-2004) – Unregulated Inflow Statistics			
Duration	**Log Mean (cfs)**	**Log Standard Deviation (cfs)**	**Skew**
Peak	4.581	0.430	-0.08
1 Day	4.453	0.425	-0.05
3 Day	4.326	0.414	-0.05
7 Day	4.162	0.398	-0.13
15 Day	4.015	0.373	-0.26
30 Day	3.897	0.360	-0.42

The family of unregulated rain flood frequency curves generated from these statistics is presented in **Figure A-2**. Exceedance frequencies can be read off of the mean 3-day rain flood frequency curve (**Figure A-3**). For the 0.01 probability event, the mean 3-day volume is 188,400 cfs.

A-4 Reservoir Model and Operating Assumptions

The Folsom Dam Operations and Planning Model was updated to include the latest storage capacity table developed in 2005, the auxiliary spillway rating curves derived from the Folsom Dam Auxiliary Spillway physical model study results from Nov 2007, and the dam safety assumptions coordinated with Reclamation.

a. Water Control Plan

The Water Control Diagram (WCD) provides the guidelines and limitations defining the release and storage of water within the flood control space. Around 1995, an interim WCD was implemented for Folsom Dam. This interim WCD is the product of an operational agreement between Reclamation and the Sacramento Area Flood Control Agency (SAFCA). The Folsom Dam WCD maintains a minimum allowable flood control reservation of 400,000 acre-feet. With an additional 270,000 acre-feet of variable flood space based on creditable storage available in upstream reservoirs, a maximum flood control reservation of 670,000 acre-feet is possible. This WCD will be referred to as the 400/670 WCD **(Figure A-4)**. The 400/670 diagram is more conservative than the WCD contained in the 1986 Folsom Dam Water Control Manual so there is no conflict in operation.

Under WRDA 1999, Congress directed the reduction of the variable flood control space from the current operating range of 400,000-670,000 acre-feet to 400,000-600,000 acre-feet upon the completion of improvements to Folsom Dam. The modifications to the project will include the construction of an auxiliary spillway under the JFP project, which will be followed by a 3.5 ft dam raise. The hypothetical future WCD for Folsom Dam is herein referred to as the 400/600 WCD **(Figure A-5)**.

Operation within the surcharge pool is prescribed by the applicable Emergency Spillway Release Diagram (ESRD). The diagram is constructed following procedures in EM 1110-2-3600, "Engineering and Design – Management of Water Control Systems". The ESRD smoothes the transition from releases made under normal flood operation releases to those required for dam safety. The diagram indicates the minimum permissible release that can be made without endangering the structure and without releasing quantities in excess of natural runoff. The ESRD attenuates Folsom Dam flood outflows to a level less than the inflow to the dam. The release specified is made immediately in order to reduce the magnitude of later releases. The objective of the ESRD is to avoid creating a worse situation than already exists and to provide a set of rules to increase flows above the downstream channel capacity in order to protect the dam from overtopping. The ESRD instructs the operators on how and when to make this key operating decisions when the only information known is reservoir elevation and the current release.

b. Operational Limitations

1) Surcharge Storage (Flood Pool) Limitation

Per Code of Federal Regulations (CFR) 33.208.11, the project owner (Reclamation) has full responsibility for the safety of the dam/appurtenant facilities and for regulation of the project during surcharge utilization. In 2007, the Corps and Reclamation reached an agreement that Reclamation practices and standards should take precedence in defining dam safety operation and criteria. The maximum surcharge space requirement is greatly affected by the inflow design flood volume, the total discharge capacity of the project, and the plan of operation. Folsom Dam spillway was originally sized to handle a much smaller inflow design event (the probable maximum flood – aka PMF). The maximum surcharge pool level of 475.5 ft and the accompanying 5 feet of freeboard are no longer sufficient under current conditions. According to the report *American River Basin, California, Folsom Dam and Lake Revised PMF Study* (Corps, 2001), Folsom Dam can only pass 70 percent of the PMF -- assuming full operation of the outlets and spillway gates and no dam failure; The amount of overtopping is estimated to be 3.5 feet above all earthen structures.

Under the Joint Federal Project, the maximum surcharge storage space requirement would increase from elevation 475.5 to elevation 477.5. This increase is accompanied by a decrease in the freeboard requirement per Reclamation's freeboard analyses. Freeboard space above the maximum allowable surcharge storage is needed to prevent overtopping mainly by wind or wave action. The authorized storage space would remain constant and independent of any modifications to the project. The dam safety operation for the Folsom Dam project is constrained by downstream safety considerations which limit or delay increases above what the levees can handle until the reservoir water surface exceeds the designated Flood Pool. The release is held to the emergency objective release while the pool is less than or equal to the designated Flood Pool. Under the existing operation, the Flood Pool is set at elevation 470.0 ft. The 1986 ESRD allows usage of about 45,000 acre-feet of surcharge storage between elevation 466 ft (normal full pool) and elevation 470.0 ft. Once the Flood Pool is exceeded, any delays in meeting the dam safety release requirement may put the dam and downstream inhabitants at greater risk.

2) Discharge Rate of Increase Limitation

Corps guidance EM 1110-2-1420, "Engineering and Design - Hydrologic Engineering Requirements for Reservoirs" states that project operation plans should ensure that release rates-of-change be gradual and not exceed the historical maximum rates of increase. The current Folsom Dam rate-of-increase is 15,000 cfs per 2-hour period. This requirement was applied to all the Scenarios while the discharge remained at or below the emergency objective release. Thereafter, the rate of increase is unlimited for the WO conditions -- similar to the existing operation. For the NA conditions, the rate-of-increase changes to 100,000 cfs/hr while the discharge remains at or below 360,000 cfs. This criterion was coordinated with Reclamation as a requirement for their dam safety operation under the JFP project and the recommended plan (JFP project plus 3.5 ft Dam Raise) as described in the 2007 PAC document.

3) Downstream Channel Limitations

The objective release for normal flood control operation is specified by the WCD. Prior to the authorized Common Features levee improvements, the normal objective release was thought to be 115,000 cfs. Given the information available today, the actual “safe” target for an indefinitely sustained release is 90,000 cfs. The 90,000 cfs offers a zero percent chance of levee failure for the WO condition. The authorized levee improvements enable the levee system to handle 115,000 cfs under normal flood operations. The 115,000 cfs offers a zero percent chance of levee failure for the NA condition. The objective release changes once the emergency flood control operation begins. For the WO condition, the emergency objective release increases to 115,000 cfs. For the NA-145 Scenario, the emergency objective release is increased to 145,000 cfs. For the W-160 Scenario, the emergency objective release is increased to 160,000 cfs. The ability of the downstream channel to sustain 160,000 cfs is a critical assumption for the Joint Federal Project.

A-5 Scenario Description

The Common Features GRR study covers two different Folsom Dam flood routing scenarios for the existing condition: the without-project condition and the no-action future without-project) condition. The without-project (WO) represents the period prior to any work on the levees. The objective release is limited to 115,000 cfs. The no-action condition represents the current state of the levee system after all the authorized repairs and improvements are complete. Under the NA condition, the downstream levees can sustain 145,000 cfs Altogether, there are six routings under the existing condition: WO1, WO2, WO3, NA1-145, NA2-145, and NA3-145. There are three routings under the “with-project” condition: W1-160, W2-160, and W3-160. Refer to **Table A-2** for key information associated with the various scenarios. The following describes the assumptions for each alternative. Given study time constraints, a standard ESRD was assembled for each alternative. No effort was made to “optimize” or tailor the ESRDs beyond establishing the total spillway capacity available, the “Flood Pool” elevation, the emergency objective release limit, and placement of the minimum induced surcharge curve.

a. WO Scenarios

This represents the levee condition existing prior to WRDA 1996 & 1999. The emergency objective release is 115,000 cfs. Prior to the authorized repairs/improvements, the American River levees were thought capable of handling 115,000 cfs under normal flood operations and 160,000 cfs for a short duration to facilitate downstream evacuation. Current studies estimate that the capacity of the levee system under the "without-project condition" was actually closer to 90,000 cfs as a “safe” release for normal flood control operation and no more than 115,000 cfs for emergency releases.

1) WO1 – This represents the levee condition existing prior to WRDA 1996 & 1999. The emergency objective release is 115,000 cfs. The dam safety release is restricted to 115,000 cfs until the water surface reaches 470.0 ft to facilitate evacuation of the downstream. The water control plan consists of the 400/670 water control diagram used in conjunction with a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam cannot pass the PMF without maintaining adequate freeboard. For dam safety purposes, outflow is made to match inflow once the water surface reaches pool elevation 475.5 feet.

2) WO2 – This represents the levee condition existing prior to WRDA 1996 & 1999. The emergency objective release is 115,000 cfs. The dam safety release is restricted to 115,000 cfs until the water surface reaches 470.0 ft to facilitate evacuation of the downstream. This scenario reflects improvements to Folsom Dam -- the construction of the Joint Federal Project (auxiliary spillway). The water control plan consists of the 400/600 water control diagram along with a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam cannot pass the PMF without overtopping the dam. For dam safety purposes, outflow is made to match inflow once the water surface reaches pool elevation 475.5 feet.

3) WO3 – This reflects additional improvements to Folsom Dam, the construction of the Joint Federal Project (auxiliary spillway) followed by a 3.5 ft dam raise. The emergency objective downstream release is 115,000 cfs. The dam safety release is not allowed to exceed 115,000 cfs until the water surface reaches 470.0 ft in order to facilitate evacuation of the downstream. The water control plan consists of both a 400/600 water control diagram and a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam cannot pass the PMF without overtopping the dam. For dam safety purposes, outflow is made to match inflow once the water surface reaches pool elevation 475.5 feet.

b. NA Scenarios

The NA scenarios represent the levee condition following the completion of WRDA 1996 & 1999. The downstream levees are capable of sustaining 145,000 cfs. Only, NA2 and NA3 operations are designed to pass the PMF -- meaning these scenarios can contain the resultant maximum surcharge volume within the maximum surcharge pool as specified in **Table A-2**. The resultant freeboard meets the freeboard requirement set by Reclamation for dam safety purposes. This also satisfies the Corps minimum freeboard requirement per regulation *ER 1110-8-2 (FR), "Engineering and Design - Inflow Design Floods for Dams and Reservoirs"*. No other goals or performance criteria were targeted in the NA2-145 and NA3-145 routings. The operation for the NA scenarios is intended to show increased performance as modifications are made to the project. NA3-145 outperforms NA2-145 which in turn must be better than NA1. Except for the downstream emergency objective release constraint of 145,000 cfs, NA2-145 and NA3-145 have operational criteria similar to the future with-project described in the next section.

1) NA1 – This scenario reflects no improvements to Folsom Dam. The emergency objective release is 145,000 cfs. The dam safety release is restricted to 145,000 cfs until the water surface exceeds 470.0 ft to facilitate evacuation of the downstream. The water control plan is comprised of the 400/670 water control diagram and a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam cannot pass the PMF without maintaining adequate freeboard. For dam safety purposes, outflow is made to match inflow once the water surface reaches pool elevation 475.5 feet.

2) NA2 – This scenario reflects an improvement made to Folsom Dam -- the construction of the Joint Federal Project (auxiliary spillway). The dam safety release is restricted to 145,000 cfs until the water surface reaches 466.0 ft to facilitate evacuation of the downstream. Downstream considerations no longer trump the dam safety operation within the surcharge space above pool elevation 466.0 ft. The water control plan consists of the 400/600 water control diagram along with a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam can pass the PMF without overtopping the dam.

3) NA3 -- This reflects additional improvements to Folsom Dam, the construction of the Joint Federal Project (auxiliary spillway) followed by the 3.5 ft dam raise. The height of the emergency gates will be increased to enable the three emergency spillway gates to remain in the closed position for a longer period, if necessary. The emergency objective downstream release is 145,000 cfs. The dam safety release is not allowed to exceed 145,000 cfs until the water surface exceeds 471.5 ft. The water control plan consists of both a 400/600 water control diagram and a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam can pass the PMF without overtopping the dam.

c. W Scenarios

The W scenarios are the future with-project condition. The W2 and W3 scenarios can pass the PMF while still satisfying the minimum 3 ft freeboard requirement for the top of dam. These scenarios are intended to show the increased performance gained by fixing the problems identified post WRDA 1996/1999 authorization. W2-160 and W3-160 have strong similarities to the 2007 PAC Report alternatives. W2-160 and W3-160 have the goal of passing the single 1-in-200 yr design event while maintaining a release of 160,000 cfs. Per coordination with Reclamation on the JFP, their preference is that this design event be maintained within the authorized normal full pool (elevation 466 feet). For the raise project, Reclamation prefers that the maximum water surface for the design event be confined at or below Flood Pool .5 feet.

1) W1 – This scenario reflects no improvements to Folsom Dam. The emergency objective release is 160,000 cfs. The dam safety release is restricted to 160,000 cfs until the water surface exceeds 466.0 ft. The water control plan is comprised of the 400/670 water control diagram and a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam cannot pass the PMF without maintaining adequate freeboard. For dam safety purposes, outflow is made to match inflow once the water surface reaches pool elevation 475.5 feet.

3) W2 – This scenario reflects an improvement made to Folsom Dam -- the construction of the Joint Federal Project (auxiliary spillway). The dam safety release is restricted to 160,000 cfs until the water surface exceeds 466.0 ft. Downstream considerations no longer trump the dam safety operation within the surcharge space above pool elevation 466.0 ft. The water control plan consists of the 400/600 water control diagram along with a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam can pass the PMF without overtopping the dam.

3) W3 -- This reflects additional improvements to Folsom Dam, the construction of the Joint Federal Project (auxiliary spillway) followed by the 3.5 ft dam raise. The height of the emergency gates will be increased to enable the three emergency spillway gates to remain in the closed position for a longer period, if necessary. The emergency objective downstream release is 160,000 cfs. The dam safety release is not allowed to exceed 160,000 cfs until the water surface reaches 471.5 ft. The water control plan consists of both a 400/600 water control diagram and a hypothetical emergency spillway release diagram. Under this scenario, Folsom Dam can pass the PMF without overtopping the dam.

TABLE A-2: DESCRIPTION OF SCENARIOS

Alternative	Top of Dam	Maximum Surcharge Flood Pool[1]	Freeboard [3]	Flood Pool [4]	Emergency Objective Release	Normal Flood Control Reservation Range [5]
	El, ft	El, ft	El, ft	El, ft	Cfs	El, ft (acre-feet)
WO1 Pre-Common Features	480.5	475.5[2]	5	470.0	90,000 (< 35% encroachment) 115,000 (> 35% encroachment)	425.8 to 388.3 (400,000 – 670,000)
WO2 Pre-Common Features Auxiliary Spillway	480.5	475.5[2]	5	470.0	90,000 (< 35% encroachment) 115,000 (> 35% encroachment)	425.8 to 399.7 (400,000 – 600,000)
WO3 Pre-Common Features Auxiliary Spillway Folsom Dam Raise 3.5 ft	484.0	479.0	5	470.0	90,000 (< 35% encroachment) 115,000 (> 35% encroachment)	425.8 to 399.7 (400,000 – 600,000)
NA1-145 Common Features	480.5	475.5	5	470.0	145,000	(425.8 to 388.3 400,000 – 670,000)
NA2-145 Common Features Auxiliary Spillway	480.5	477.5	3	466.0	145,000	425.8 to 399.7 (400,000 – 600,000)
NA3-145 Common Features Auxiliary Spillway Folsom Dam Raise 3.5 ft	484.0	481.0	3	471.5	145,000	425.8 to 399.7 (400,000 – 600,000)
W1-160 Common Features	480.5	475.5	5	470.0	160,000	(425.8 to 388.3 400,000 – 670,000)
W2-160 Common Features Auxiliary Spillway	480.5	477.5	3	466.0	160,000	425.8 to 399.7 (400,000 – 600,000)
W3-160 Common Features Auxiliary Spillway Folsom Dam Raise 3.5 ft	484.0	481.0	3	471.5	160,000	425.8 to 399.7 (400,000 – 600,000)

KEY
El, ft – Elevation in feet

Notes:
1. *These values reflect the highest allowable pool elevation given both freeboard and top of dam height requirements. The maximum surcharge flood pool is established by routing a PMF through the reservoir. The PMF has been updated or revised periodically (e.g. 1946, 1980, 1991, and 2001).*
2. *The existing project requires more surcharge storage than is available under the original project design. Under existing conditions with no modifications to Folsom Dam, the 2001 PMF event would overtop Folsom Dam.*
3. *Reclamation has determined that 3 feet provides sufficient freeboard for the with-project scenarios (no action).*
4. *The FDR flood pool elevations are associated with the JFP and 3.5 Ft Dam Raise projects described in the PAC document. The release from Folsom Dam will not exceed 160,000 cfs as long as the water surface remains at or below the FDR flood pool.*
5. *The authorized storage space allocation for flood control differs with the scenarios. The flood space requirement itself varies seasonally. The maximum space would be needed only during the most critical flood period (December through February)*

A-6 Summary of Routing Output Analyses

a. WO Scenarios (pre-dates improvements authorized under WRDA 1996 & 1999)

With the addition of an auxiliary spillway in WO2, the main benefit gained is the ability to accelerate evacuation of the flood space. Although the downstream channel was originally designed to sustain an objective release of 115,000 cfs under normal flood operations, the current findings is that the potential for levee failure was greater than thought possible at that time. Under today's standards, the downstream channel was never maintained well enough to sustain safe releases of 115,000 cfs. To ensure zero percent chance of failing the downstream levees, the normal objective release requirement should have been reduced to 90,000 cfs. According to the attached **Figure A-8**, WO1 is able to limit the release to 90,000 cfs up to a 1-in-25 yr chance event. WO2 and WO3 must not utilize the extra capacity made available by the addition of the auxiliary spillway beyond this "safe" level except for events larger than a 1-in-25 yr chance event. Reservoir encroachment is the unit of measurement selected to identify event size. The encroachment volume for a 1-in-25 yr chance event never exceeded 35% in the WO1 routing. Therefore, larger events would be characterized by their larger encroachment percentages. Thus, the model was adjusted to limit the release to 90,000 cfs as long as the encroachment level remained at or below 35%. Thereafter, the release restriction would be lifted and the discharge would be allowed to ramp up to 115,000 cfs.

The operation for the WO scenarios is intended to show increased performance as modifications are made to the Common Features project and improvements are made to Folsom Dam. WO3 outperforms WO2 which in turn is better than WO1. The WO scenarios were not intended to pass the PMF. Operation for the WO scenarios was not constrained by any measurable criteria (i.e. passing a certain percentage of the PMF or limiting the magnitude of any dam overtopping to a certain amount). These scenarios cannot contain the resultant maximum surcharge volume within the confines of the maximum surcharge pool specified in **Table A-2**. The resultant freeboard is also less than the required freeboard amount. For these scenarios, the operation postpones making releases greater than 115,000 cfs due to downstream considerations by using up to 4 ft of surcharge storage space. The dam safety release is restricted to 115,000 cfs until the water surface reaches 470.0 ft to facilitate evacuation of the downstream.

b. NA Scenarios

The ESRDs created for the various scenarios may be considered much too efficient. The NA3-145 alternative is an example of this. According to the attached **Figure A-9**, the routing results indicate that Folsom Dam operations can hold the release at 145,000 cfs for a 1-in-200 yr event. Note, however, significant use of the surcharge space is required to achieve this result. The "Flood Pool" is being greatly exceeded. The release is appropriate given the circumstances in the routing with rapidly falling inflow and insignificant rate of rise in the reservoir pool elevation. The only way to make the consequences of exceeding the "Flood Pool" fully apparent in the routing is to use "simplified" ESRDs -- ones in which the pool elevation would be the only factor used to determine the discharge requirement. The "simplified" ESRD would remove any flexibility in surcharge space usage by automatically forcing the discharge to increase beyond the target flow anytime the pool elevation exceeded the designated "Flood Pool". Under this scenario, at 471.5 ft the discharge would be held to 145,000 cfs but at 471.51 the release would be greater than 145,000 cfs. The "soft" enforcement makes more sense than the "hard" enforcement approach when it comes to reservoir operations. **Table A-3** offers a comparison of maximum water surface versus "Flood Pool" specification for the various scenarios.

c. W Scenarios

TABLE A-3: FLOOD POOL ROUTING SUMMARY Ŧ

1-in-N chance per year event	WO1 (Flood Pool 470.0 ft)		WO2 (Flood Pool 470.0 ft)		WO3 (Flood Pool 470.0 ft)		NA1-145 (Flood Pool 470.0 ft)		NA2-145 (Flood Pool 466.0 ft)		NA3-145 (Flood Pool 471.5 ft)		W1-160 (Flood Pool 470.0 ft)		W2-160 (Flood Pool 466.0 ft)		W3-160 (Flood Pool 471.5 ft)	
	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)	Max WS (El, ft)	Peak Outflow (cfs)
2	403.93	30295	403.53	37708	403.53	37708	402.43	30183	403.18	25215	403.18	25215	403.08	25891	401.91	37708	403.18	25215
10	429.80	43692	408.97	90000	408.97	90000	429.13	43127	421.65	71655	421.65	71655	431.09	43519	421.65	71655	421.65	71655
25	442.53	98760	427.80	90000	427.80	90000	442.69	99738	431.43	115000	431.43	115000	444.54	104311	432.02	115000	432.02	115000
50	457.34	115000	443.02	115000	443.02	115000	457.01	115000	442.97	115000	442.97	115000	459.13	115000	444.04	115000	444.04	115000
100	476.35	123107	461.00	115000	461.00	115000	470.81	145000	460.46	115000	460.46	115000	472.32	145000	461.31	115000	461.31	115000
200	476.33	444310	476.65	169173	478.67	138359	476.40	320142	470.02	210332	474.92	145000	476.37	321017	470.02	196633	472.47	160000
250	476.65	476319	475.23	331691	477.27	232803	476.67	412114	470.65	309673	477.90	197562	476.64	408551	470.44	296022	477.15	193667
500	479.62	554268	480.97	627077	481.31	510279	479.01	512982	472.08	594159	478.32	558062	479.04	513195	471.57	594159	478.03	534386

Notes:
Ŧ *The gray shaded area depicts encroachment into the remaining surcharge storage space above the "Flood Pool" mark; Dam Safety operation takes the highest priority above the "Flood Pool" mark.*

This page intentionally left blank.

A-7 Risk Analysis (HEC- FDA Inputs)

Corps engineering guidance (EM 1110-2-1619, "Risk-Based Analysis for Flood Damage Reduction Studies") and planning guidance (ER 1105-2-100, "Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures" and ER 1105-2-101, "Risk Analysis for Flood Damage Reduction Studies") require that risk analyses be used to quantify the project performance of the various scenarios. The hydrologic data provided to Economics as input for the HEC-FDA program includes the unregulated inflow exceedance probability function and the curves defining the relationship between unregulated inflow and reservoir discharge. The uncertainty in the hydrology is defined by the confidence limits, derived via statistics. The uncertainty in reservoir discharge is derived by changing the parameters used in the reservoir routings. The risk analysis scenarios reflect the operating conditions ranging from the most likely to occur (BASE) to the most extreme operating conditions likely to produce the largest (MAXIMUM) or smallest (MINIMUM) expected release. The BASE condition assumptions and results are previously described for the W01, W02, W03, NA1, NA2, and NA3 scenarios. Generally, the operational criteria are developed based on actual flood operations, the analysis of historical data, and discussion between representatives of the Corps, SAFCA, and Reclamation. **Table A-4** presents selected assumptions used to create the different scenarios.

TABLE A-4: RISK ANALYSIS OPERATIONAL ASSUMPTIONS [1, 2]

Uncertainty Parameters	Alternative	Discharge Scenario		
		BASE (Normal)	MAXIMUM (Upper Limit)	MINIMUM (Lower Limit)
Initial Encroachment [3] (acre-feet)	WO & NA	0	50,000	0
Extra Space in Folsom Lake (acre-feet)	WO & NA	0	0	100,000
Available Upstream Reservoir Space (acre-feet)	WO & NA	0	0	150,000
Starting Storage (acre-feet)	WO & NA	367,000	417,000	429,000
Response Time Delay [4] (hours)	WO	8	8	8
	NA	4	8	0
Main Dam River Outlets Operation During Concurrent Spillway Operation (percent gate opening)	WO & NA	60	0	60
KEY Cfs – cubic feet per second				

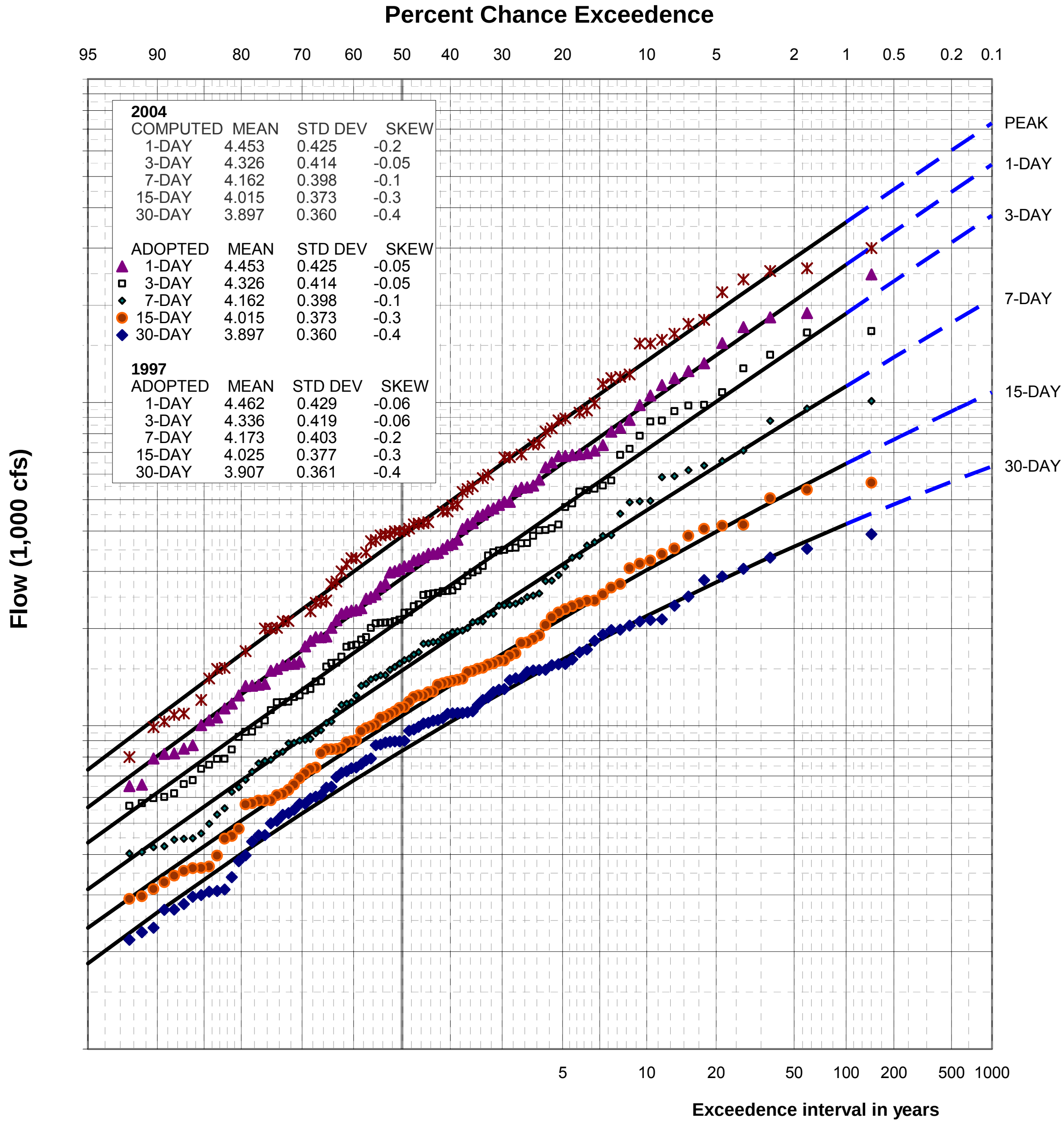
Notes:

1. *Discharge is presumed through only one power penstock due to maintenance work during the flood season (per Reclamation).*
2. *Application of the uncertainty parameters may sometimes result in anomalies for the smaller or more frequent events. The settings meant to induce the largest or smallest discharge may actually result in the reverse. This issue appears intermittently.*
3. *Encroachment is relative to the allowable storage as determined from the water control diagram (dependent on upstream storage space).*
4. *Lag in matching Release to previous hour Inflow – while discharge is less than the normal objective release target.*

A-8 Conclusion

Water Management produced routings for two different scenarios. The without-project (WO) condition reflects the American River levee system prior to any improvements or repair work. The no-action (NA) condition reflects the existing state of the American River levees with the improvements made as authorized by WRDA 1996 and 1999. The NA condition will result in the ability of the downstream channel to sustain 145,000 cfs (or 160,000 cfs as reported in the 2007 PAC Report). The 50%-, 20%-, 4%-, 2%-, 1%-, 0.5%, 0.2%-annual chance flood events were routed through Folsom Dam for the various WO and NA scenarios. The routing results were given to Hydraulic Design for the floodplains development and to Economics for the economic benefit analyses. The hydrographs provided to Hydraulic Design are shown in **Figures A-4 through A-6**.

Figure A-10 through A-23 provides a snapshot of the data provided to Economics in a variety of ways. **Figure A-10 through A-13** presents the set of WO, NA, and W results (BASE condition only) as regulated frequency curves. This allows one to view the increase in project performance as improvements are made to Folsom Dam. **Figure A-14** consolidates the results of all the routings (BASE condition only) as “inflow versus outflow curves” to allow comparisons across the different set of routings. **Figure A-15 through A-23** presents the uncertainty band around the discharge for any given event. Note that the uncertainty range required some adjustment around the more frequent event where the points crossed. Generally, the anomalies (MAX < BASE < MIN) where the points cross occur for events with less than 1-in-5 yr chance exceedance. In these instances, the MAX discharge is lower than BASE due to the inability to match inflow quickly (8 hour lag). This handicap is a benefit or plus for the smaller flood events. The MIN discharge is large than BASE due to the ability to match inflow quickly (1 hour lag). This advantage (rapid response) is a detriment or negative for the smaller, more frequent events. The initial starting storage also is a factor in this aspect. A full summary of the routings can be found in **Tables A-5** through **A-31**. The reservoir routings covered herein were developed for planning purposes only. These scenarios are hypothetical and would not be built or implemented as stand-alone projects. All reservoir elevations provided herein use the NGVD29 vertical datum.



NOTES:

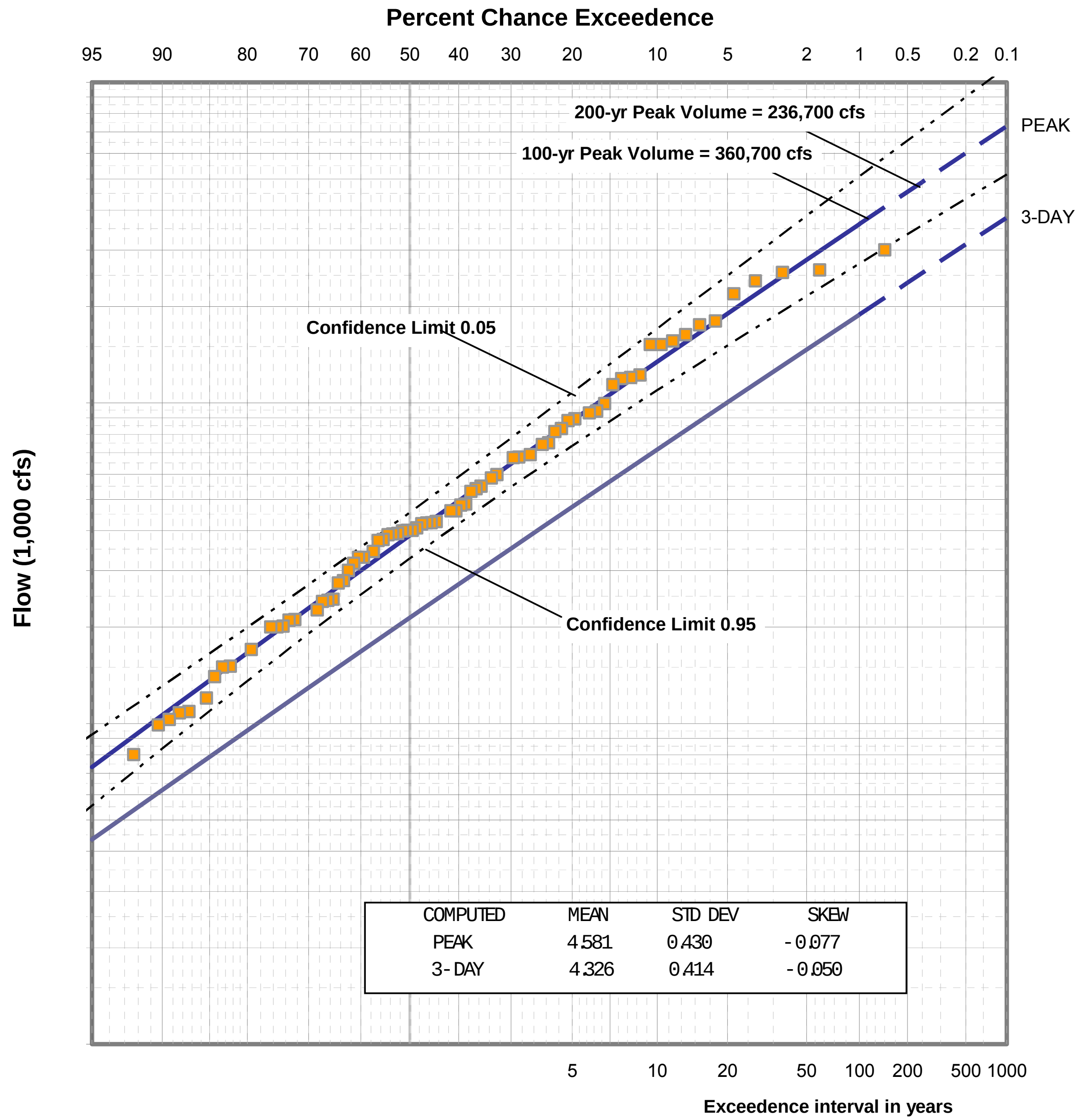
1. Adjusted USGS gage 11446500 to account for daily change in storage at upstream reservoirs (potential channel, out-of-channel, or storage losses neglected).
2. Median plotting positions.
3. Computed Probability
4. No adjustments for outliers.
4. Drainage area: 1,888 sq. mi.
5. Period of record: 1905-2004.

AMERICAN RIVER WATERSHED PROJECT
COMMON FEATURES
GENERAL REEVALUATION REPORT

RAIN FLOOD FREQUENCY CURVES
AMERICAN RIVER AT FAIR OAKS
UNREGULATED CONDITIONS

FIGURE A-2

This page intentionally left blank.



NOTES:

1. Adjusted USGS gage 11446500 to account for daily change in storage at upstream reservoirs (potential channel, out-of-channel, or storage losses neglected).
2. Median plotting positions.
3. Computed Probability
4. No adjustments for outliers.
5. Confidence limits based on station statistics
6. Drainage area: 1,888 sq. mi.
7 Period of record: 1905-2004.

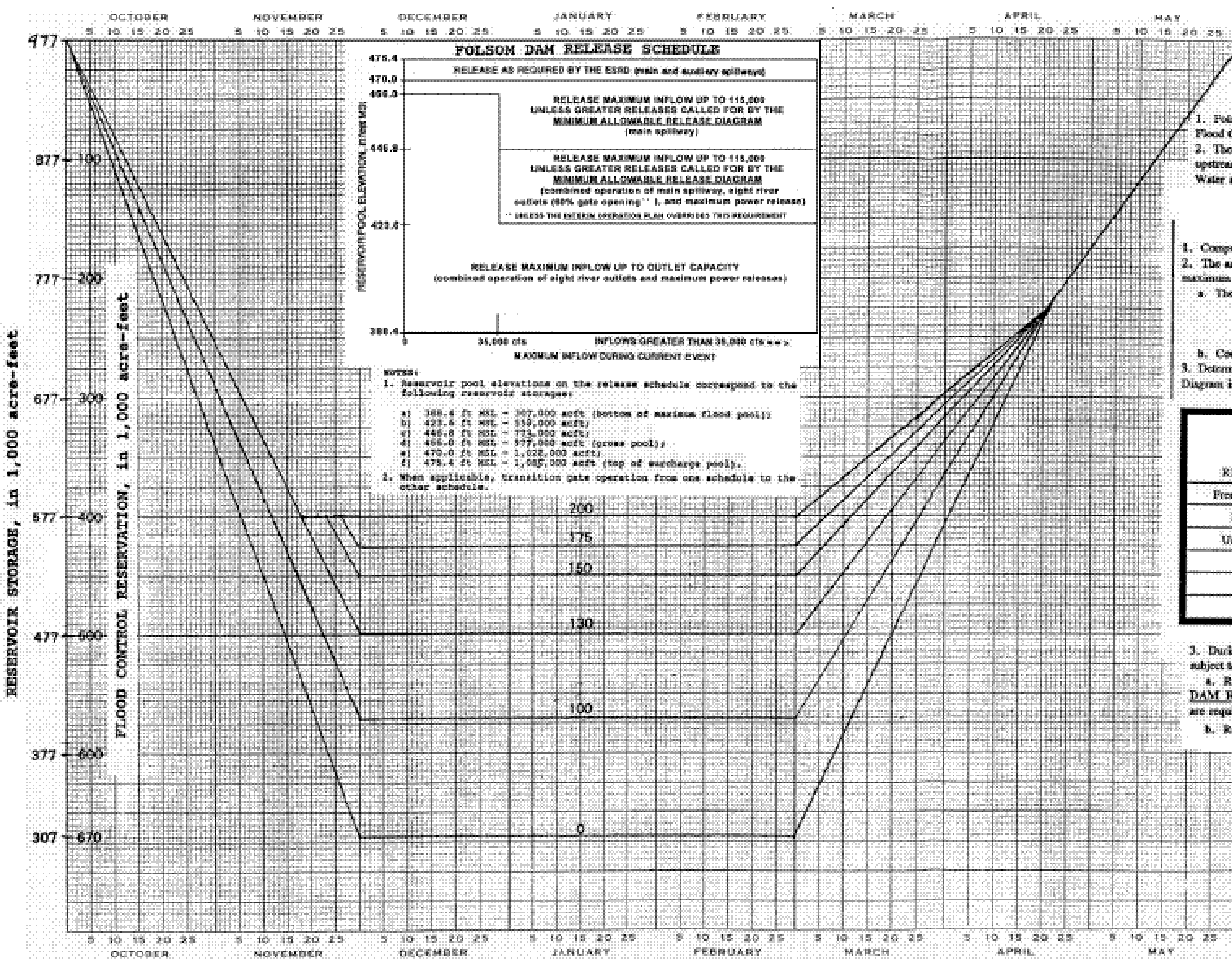
AMERICAN RIVER WATERSHED PROJECT

COMMON FEATURES

UNREGULATED PEAK AND MEAN 3-DAY RAIN FLOOD FREQUENCY CURVES AMERICAN RIVER AT FAIR OAKS

FIGURE A-3

This page intentionally left blank.



FLOOD CONTROL DIAGRAM FOR FOLSOM DAM AND LAKE

USE OF DIAGRAM

1. Folsom Dam and Lake shall be operated for flood control in accordance with the Flood Control Diagram. When water is stored within the Flood Control Reservation, reservoir releases must be in accordance with requirements of this diagram.
2. The parameters on the flood control diagram define the required Flood Control Reservation, on any given day, based on available space in the upstream reservoirs. Once the required Flood Control Reservation is computed, the Required Reservoir Storage for flood control can be determined. Water stored in excess of the Required Reservoir Storage must be evacuated. Computation of the parameter is discussed below:

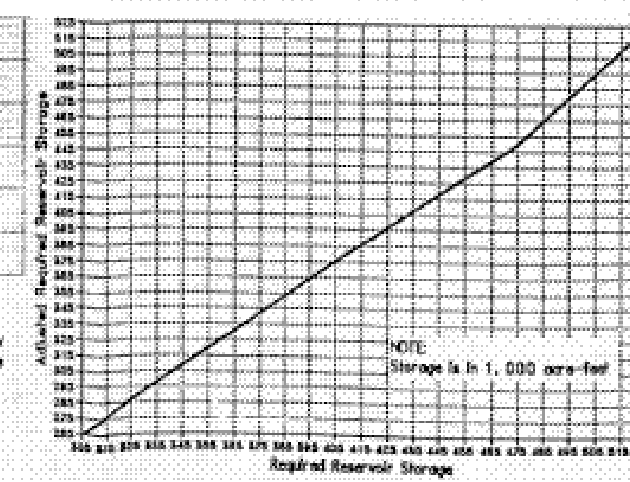
FOLSOM DAM AND LAKE COMPUTATION OF REQUIRED RESERVOIR STORAGE

1. Compute space available below spillway crest, in acre-feet, for the following reservoirs: French Meadows, Hell Hole and Union Valley.
2. The amount of creditable flood control transfer space in each reservoir is then computed by taking the smaller of the space available or the maximum creditable space for that reservoir.
   a. The maximum creditable space by reservoir is as follows:
      French Meadows - 45,000 acre-feet
      Hell Hole - 80,000 acre-feet
      Union Valley - 75,000 acre-feet
   b. Combine the creditable flood control transfer space for each reservoir to compute the total creditable space.
3. Determine the Flood Control Reservation at Folsom Lake by applying the creditable flood control transfer space (parameter on the Flood Control Diagram in 1,000 acre-feet).

SAMPLE COMPUTATION OF REQUIRED RESERVOIR STORAGE

RESERVOIR	STORAGE ON JAN 1 (TAF)	STORAGE AT SPILLWAY CREST (TAF)	SPACE AVAILABLE (TAF)	MAXIMUM CREDITABLE SPACE (TAF)	CREDITABLE FLOOD CONTROL TRANSFER SPACE (TAF)
French Meadows	75.7	110.7	35	45	35
Hell Hole	87.6	207.6	120	80	80
Union Valley	170.1	235.1	65	75	65
TOTAL CREDITABLE FLOOD CONTROL TRANSFER SPACE (TAF)					180
FLOOD CONTROL RESERVATION AT FOLSOM LAKE (TAF)					420
REQUIRED RESERVOIR STORAGE AT FOLSOM LAKE (TAF)					557

3. During a potential flood situation, water stored within the Flood Control Reservation, defined hereon, shall be released as rapidly as possible subject to the following schedule:
   a. Required Flood Control Release - Promptly release inflow up to 115,000 cfs, while inflows are increasing, as discussed in the FOLSOM DAM RELEASE SCHEDULE. Control flows in the American River below the dam to not more than 115,000 cfs, except when larger releases are required by the accompanying **INTERIM OPERATION PLAN**.
   b. Releases will not be increased more than 15,000 cfs or decreased more than 10,000 cfs during any 2 hour period.

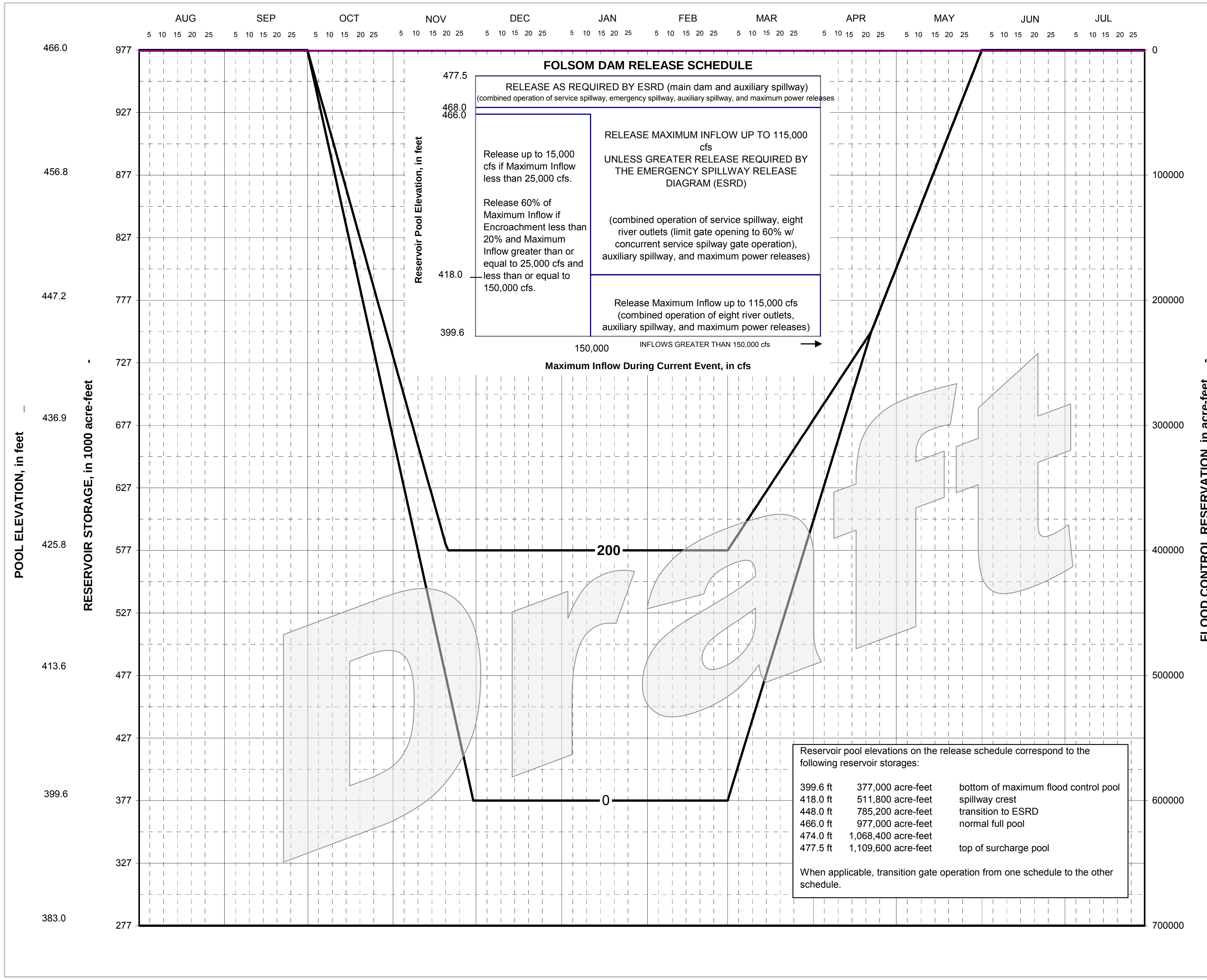


4. Maintenance or repair of any one power generating unit at Folsom Dam which results in loss of the power penstock for more than one day may require additional Flood Control Reservation. The amount of additional reservation is defined by the Adjusted Required Reservoir Storage, as identified on the graph at right, depending on the following conditions:
   a. The Required Reservoir Storage in Folsom Lake, as computed in paragraph 2 above;
   b. The existing and forecasted meteorological conditions in the basin; and,
   c. The duration the generating unit is expected to be off-line.

Required Reservoir Storage greater than or equal to 525,000 acre-feet requires no additional Flood Control Reservation.

FIGURE A-4
WATER CONTROL DIAGRAM -- HISTORICAL
EXISTING CONDITION 400/670

This page intentionally left blank.



FLOOD CONTROL DIAGRAM

USE OF DIAGRAM

1. Folsom Dam and Lake shall be operated for flood control in accordance with the Flood Control Diagram. When water is stored within the Flood Control Reservation, reservoir releases must be in accordance with the requirements of this diagram.

2. The parameters on the flood control diagram define the required Flood Control Reservation, on any given day, based on available space in the upstream reservoirs. Once the required Flood Control Reservation is computed, the Required Reservoir Storage for flood control can be determined. Water stored in excess of the Required Reservoir Storage must be evacuated. Computation of the parameter is discussed below:

COMPUTATION OF REQUIRED FLOOD RESERVATION STORAGE

1. Compute space available below spillway crest, in acre-feet, for the following reservoirs: French Meadows, Hell Hole and Union Valley.

2. The amount of creditable flood control transfer space in each reservoir is then computed by taking the smaller of the space available or the maximum creditable space for that reservoir.

   a. The maximum creditable space by reservoir is as follows:
      French Meadows 45,000 acre-feet
      Hell Hole 80,000 acre-feet
      Union Valley 75,000 acre-feet

   b. Combine the creditable flood control transfer space for each reservoir to compute the total creditable space.

   c. Determine the Flood Control Reservation at Folsom Lake by applying the creditable flood control transfer space (parameter on the Flood Control Diagram in 1,000 acre-feet).

SAMPLE COMPUTATION OF REQUIRED RESERVOIR STORAGE

RESERVOIR	STORAGE ON JAN 1 (TAF)	STORAGE @ SPILLWAY CREST (TAF)	SPACE AVAILABLE (TAF)	MAXIMUM CREDITABLE SPACE (TAF)	CREDITABLE FLOOD CONTROL TRANSFER SPACE (TAF)
FRENCH MEADOWS	65.7	110.7	45	45	45
HELL HOLE	87.6	207.6	120	80	80
UNION VALLEY	160.1	235.1	75	75	75
TOTAL CREDITABLE FLOOD CONTROL TRANSFER SPACE (TAF)					200
FLOOD CONTROL RESERVATION AT FOLSOM LAKE (TAF)					577
REQUIRED RESERVOIR STORAGE AT FOLSOM LAKE (TAF)					577

RELEASE SCHEDULE

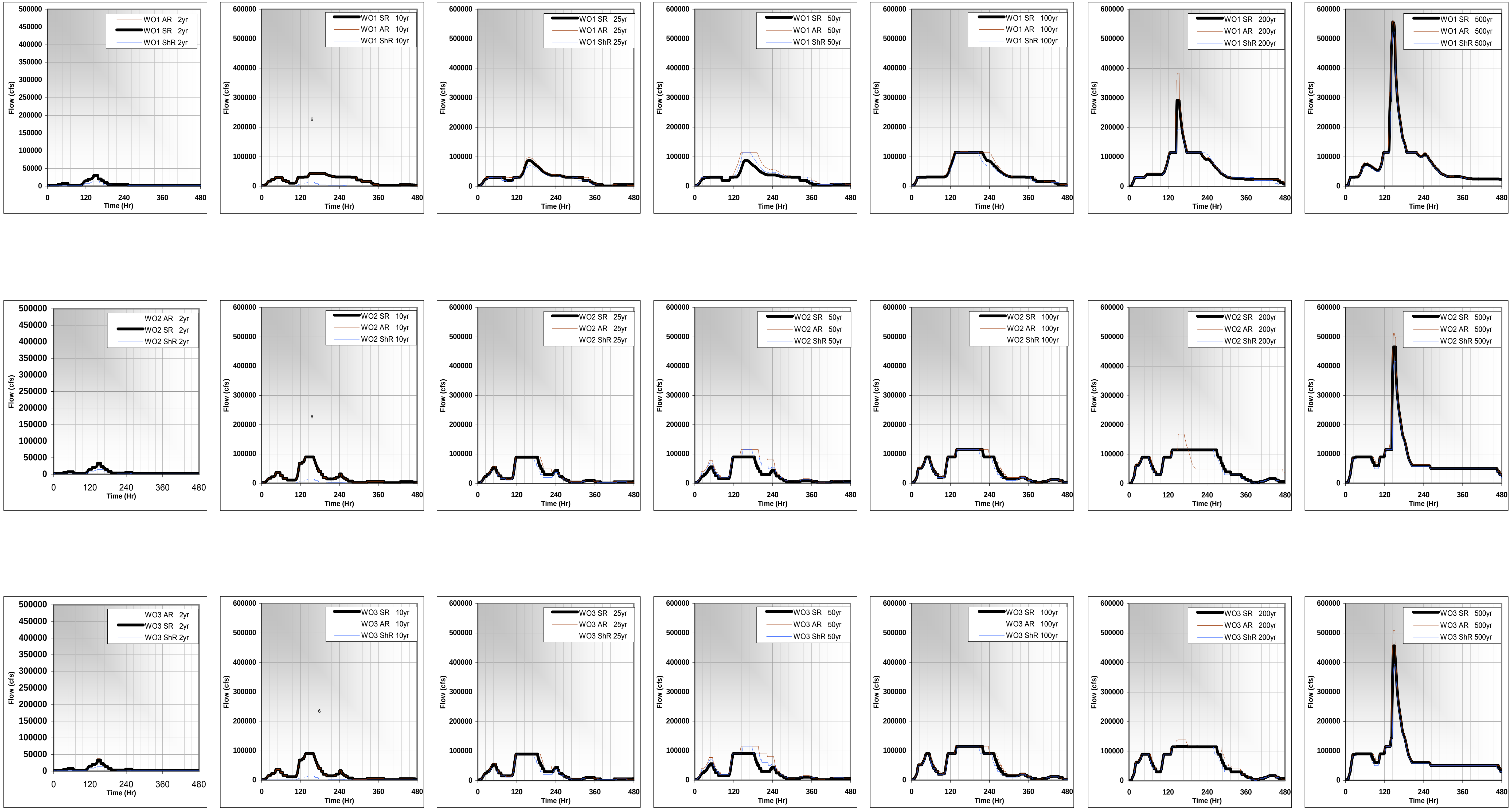
3. During a potential flood situation, water stored within the Flood Control Reservation, defined herein, shall be released as rapidly as possible subject to the following schedule:

   a. Required flood Control Release - Promptly release inflow up to 115,000 cfs while inflows are increasing, as discussed in the FOLSOM DAM RELEASE SCHEDULE. Control flows in the American River below the dam to not more than 115,000 cfs, except when larger releases are required by the accompanying EMERGENCY SPILLWAY RELEASE DIAGRAM (ESRD). Once the reservoir pool begins falling, maintain releases in excess of inflow until water stored in the Flood Control Reservation is evacuated.

   b. Releases will not be increased more than 30,000 cfs or decreased more than 10,000 cfs during any 2-hour period.

FIGURE A-5
WATER CONTROL DIAGRAM -- HYPOTHETICAL
FUTURE CONDITION 400/600

This page intentionally left blank.

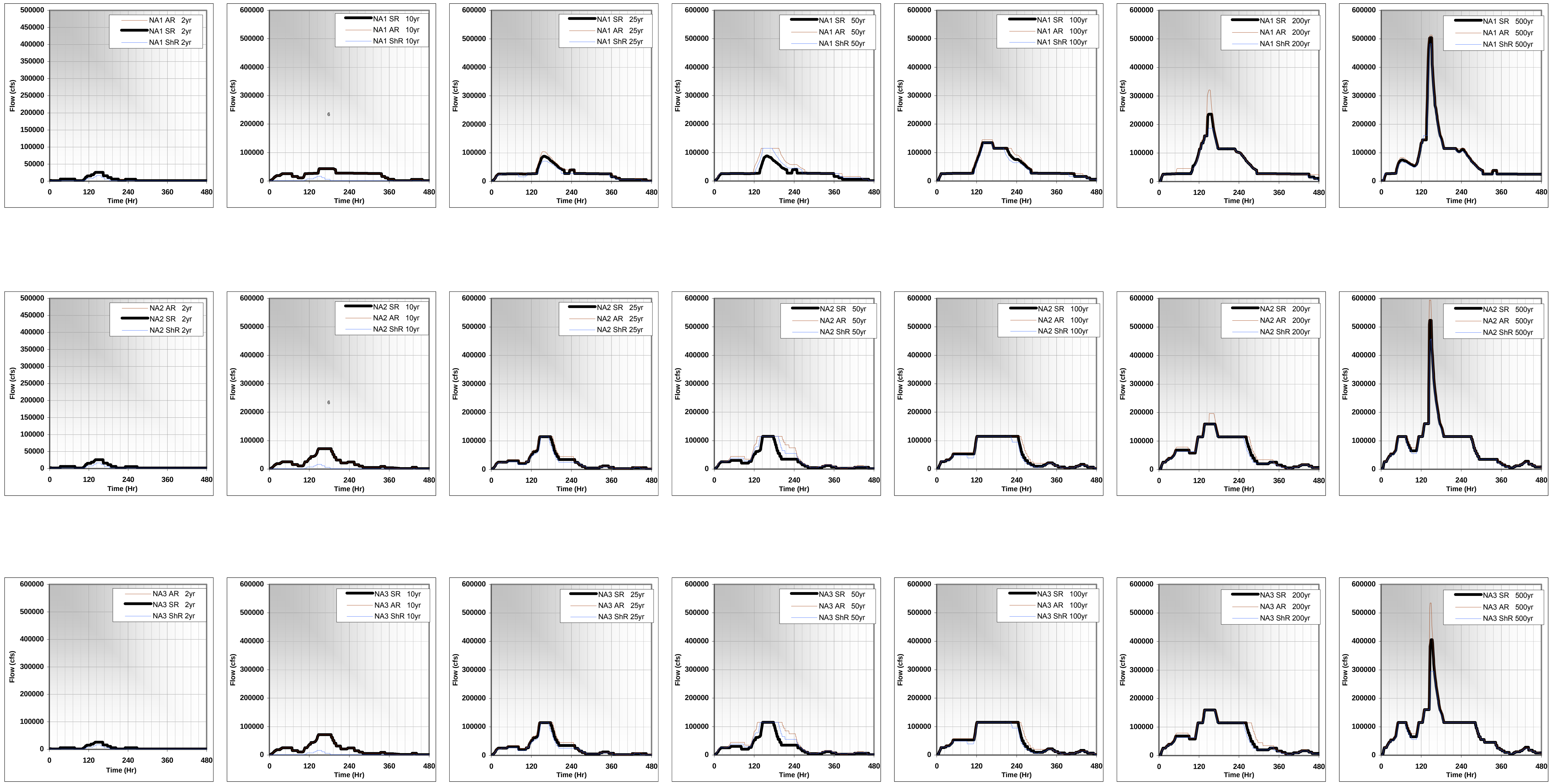


Key

SR Sacramento River
AR American River
ShR Shanghai River

FIGURE A-8: DISCHARGE HYDROGRAPHS – BASE – EXISTING CONDITION

This page intentionally left blank.

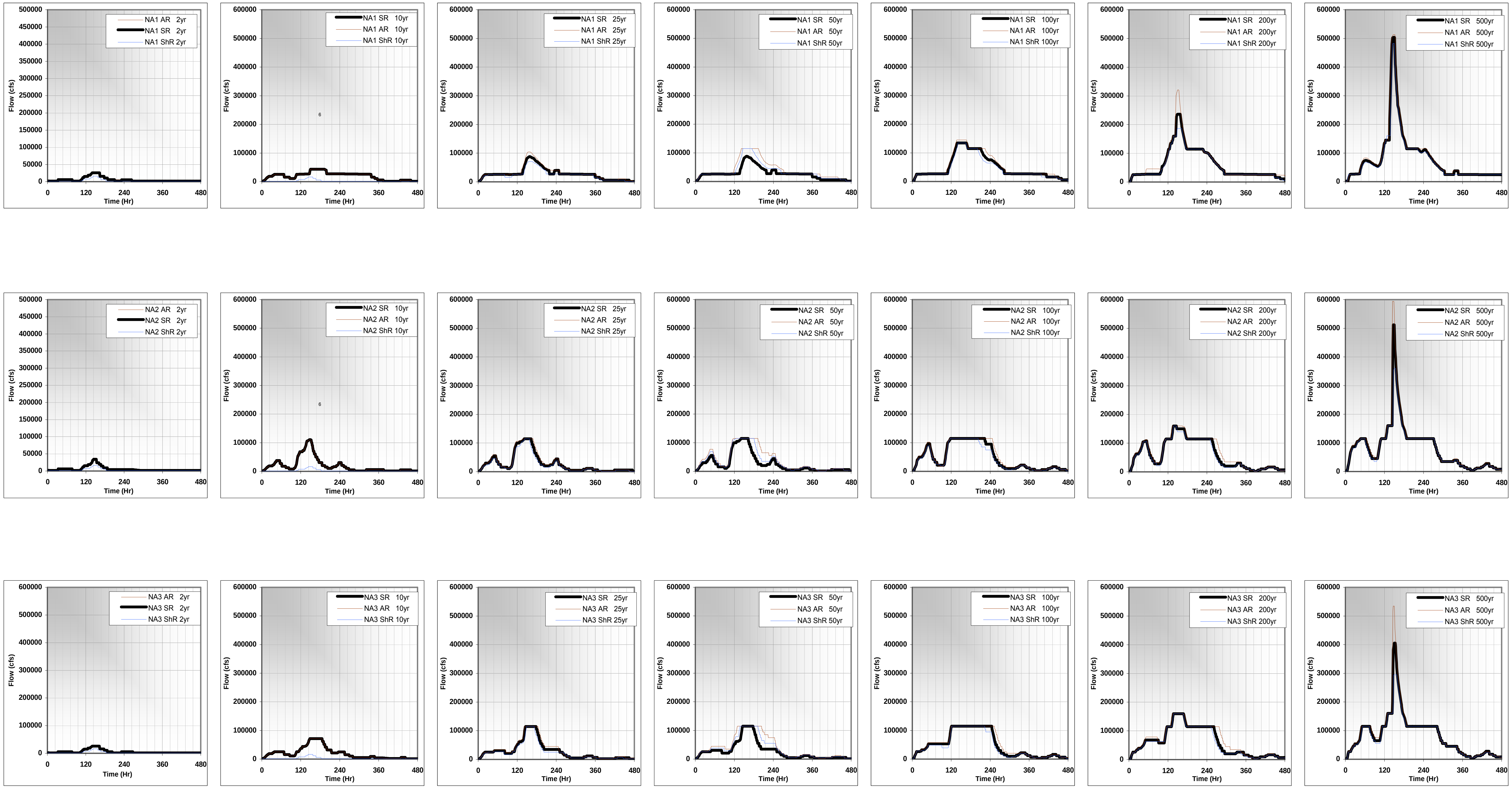


Key

SR Sacramento River
AR American River
ShR Shanghai River

FIGURE A-9: DISCHARGE HYDROGRAPHS – BASE – FUTURE WITHOUT-PROJECT (NO ACTION)

This page intentionally left blank.

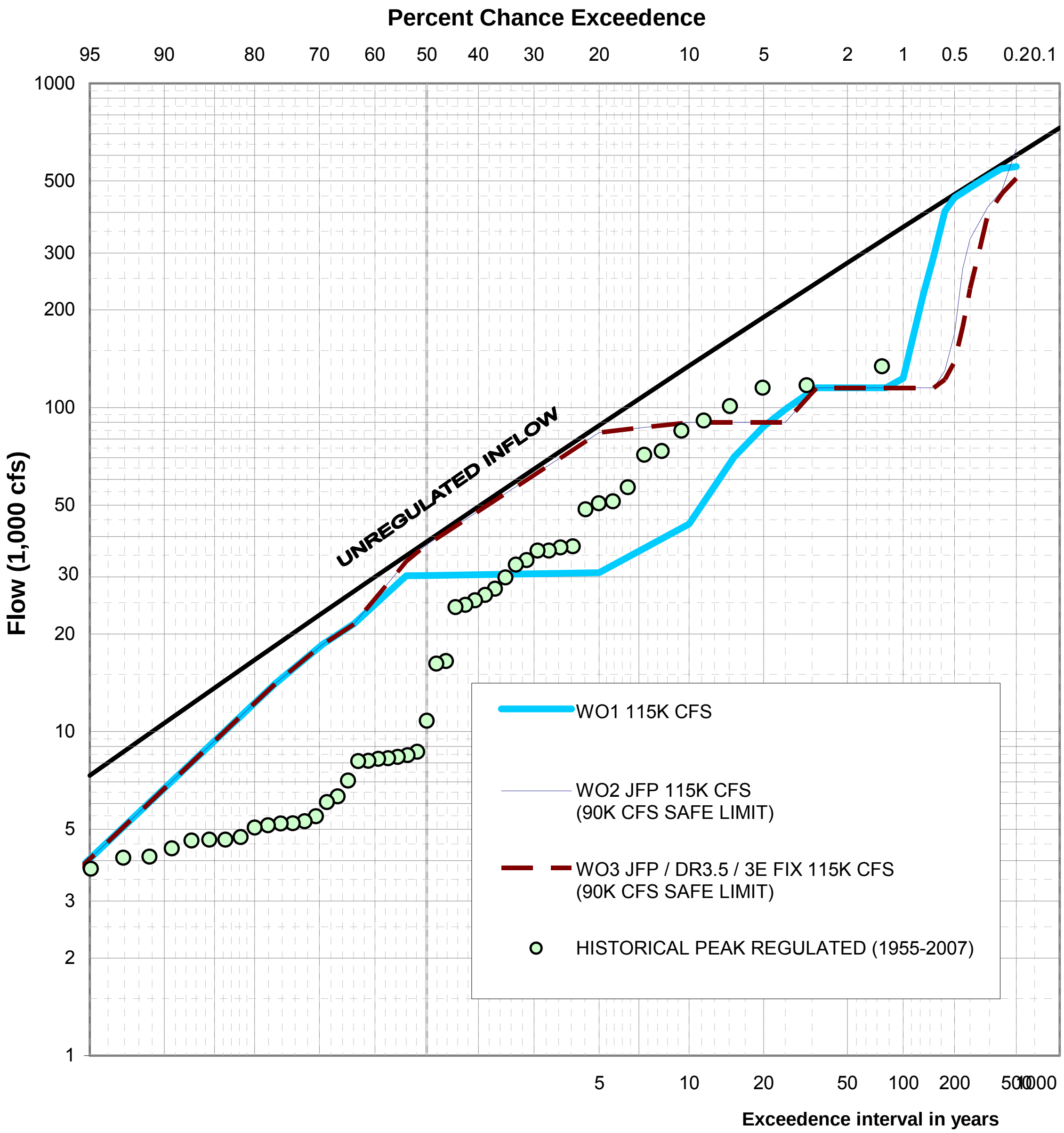


Key

SR Sacramento River
AR American River
ShR Shanghai River

FIGURE A-10: DISCHARGE HYDROGRAPHS – BASE – WITH-PROJECT

This page intentionally left blank



NOTES:

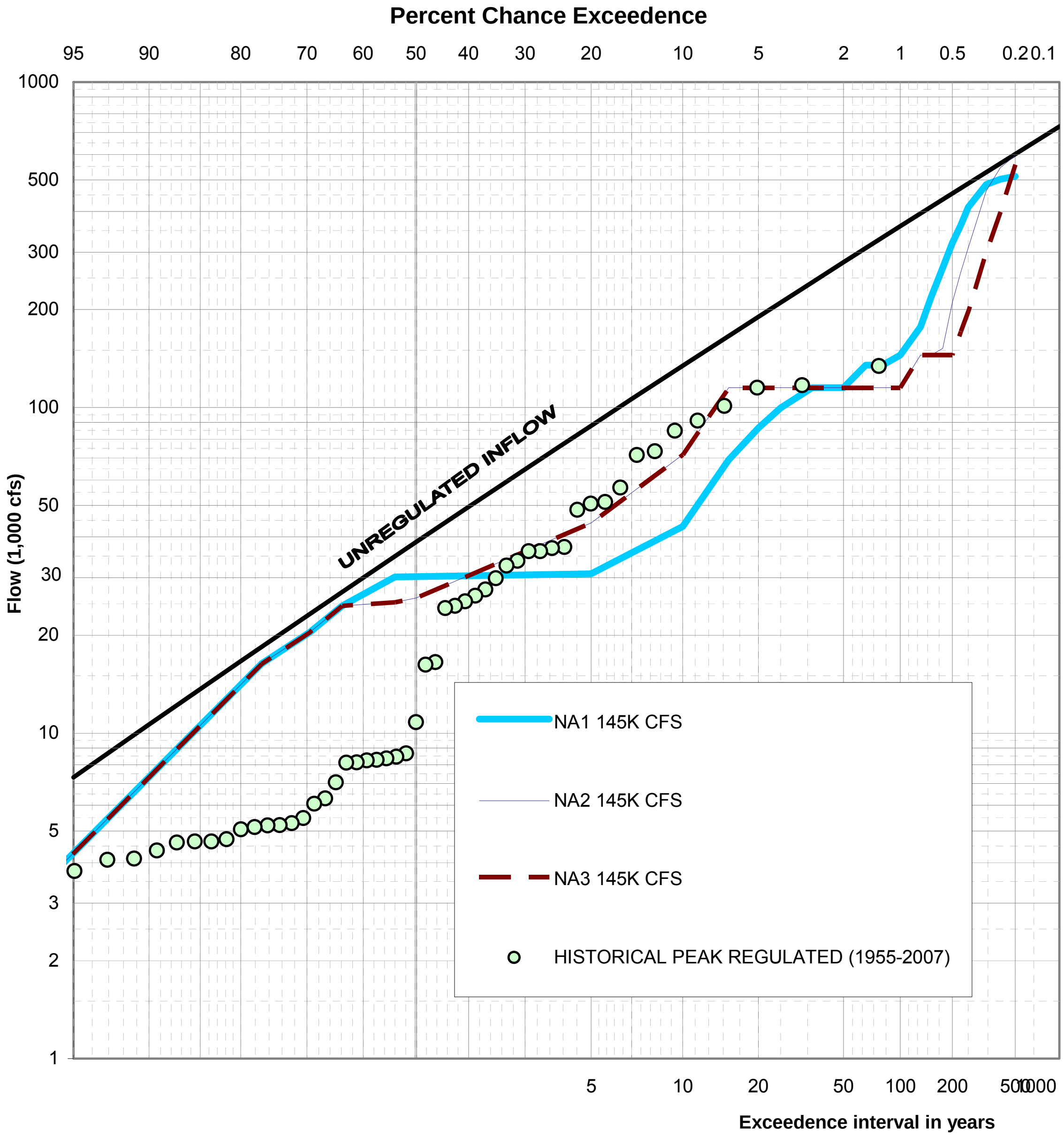
1. Adjusted USGS gage 11446500 to accoiunt for daily change in storage at upstream reservoirs (potential channel, out-of-channel, or storage losses neglected).
2. Median plotting positions
3. No adjustments for outliers.
4. Drainage area: 1,888 sq. mi.
5. Period of record: 1905-2004 (Unregulated).

AMERICAN RIVER WATERSHED PROJECT
COMMON FEATURES GRR

REGULATED FREQUENCY CURVES
FOLSOM DAM
EXISTING CONDITION
(WITHOUT-PROJECT)

FIGURE A-11

This page intentionally left blank.



NOTES:

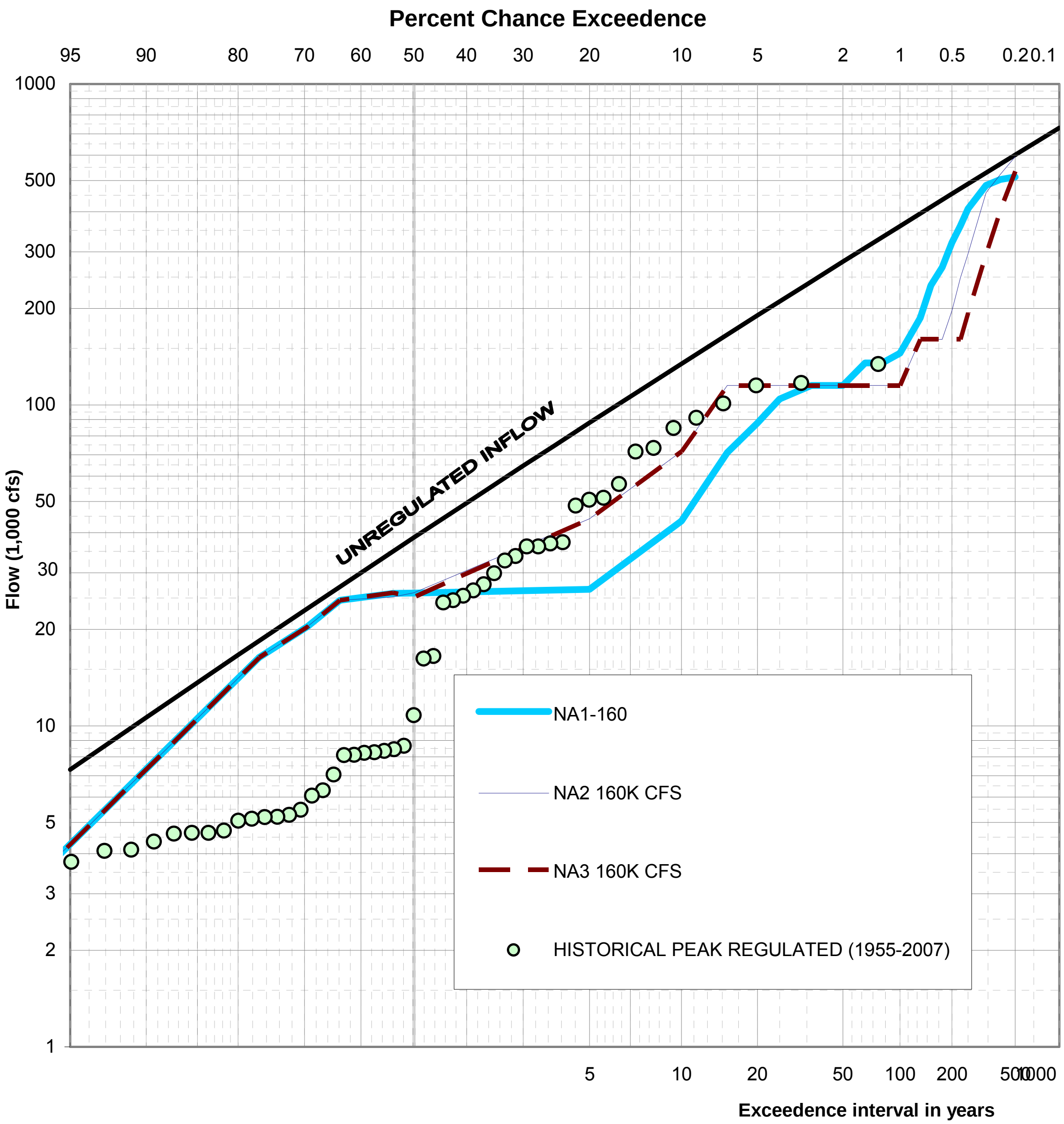
1. Adjusted USGS gage 11446500 to accoiunt for daily change in storage at upstream reservoirs (potential channel, out-of-channel, or storage losses neglected).
2. Median plotting positions
3. No adjustments for outliers.
4. Drainage area: 1,888 sq. mi.
5. Period of record: 1905-2004 (Unregulated).

AMERICAN RIVER WATERSHED PROJECT
COMMON FEATURES GRR

REGULATED FREQUENCY CURVES
FOLSOM DAM
FUTURE WITHOUT-PROJECT
(NO ACTION)

FIGURE A-12

This page intentionally left blank.



NOTES:

1. Adjusted USGS gage 11446500 to accoiunt for daily change in storage at upstream reservoirs (potential channel, out-of-channel, or storage losses neglected).
2. Median plotting positions
3. No adjustments for outliers.
4. Drainage area: 1,888 sq. mi.
5. Period of record: 1905-2004 (Unregulated).

AMERICAN RIVER WATERSHED PROJECT
COMMON FEATURES GRR

REGULATED FREQUENCY CURVES
FOLSOM DAM
FUTURE PROJECT

FIGURE A-13

This page intentionally left blank.

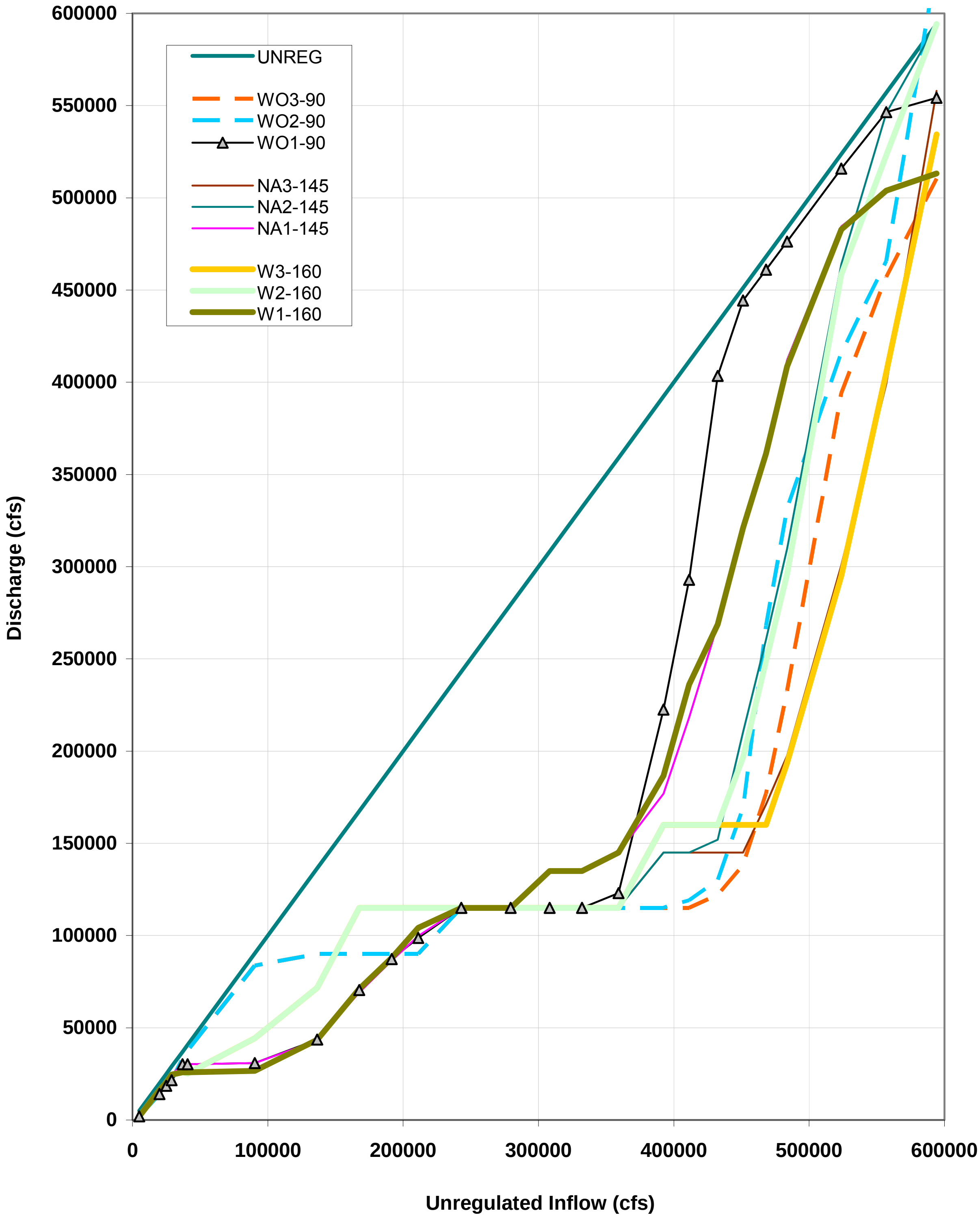


FIGURE A-14: INFLOW-OUTFLOW TRANSFORM – BASE – COMPARISON

This page intentionally left blank.

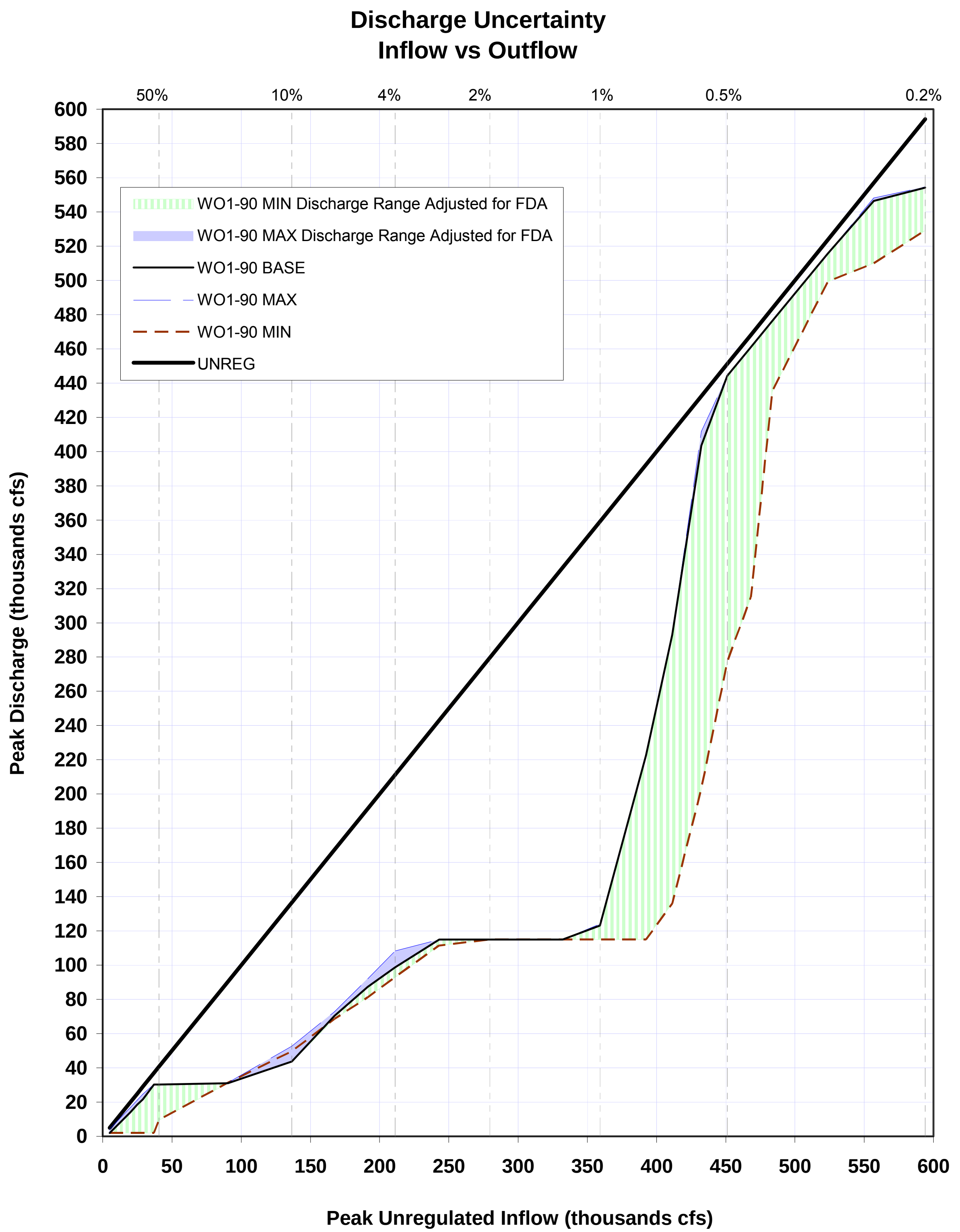


FIGURE A-15: DISCHARGE UNCERTAINTY – WO1 WITHOUT-PROJECT – 115,000 CFS

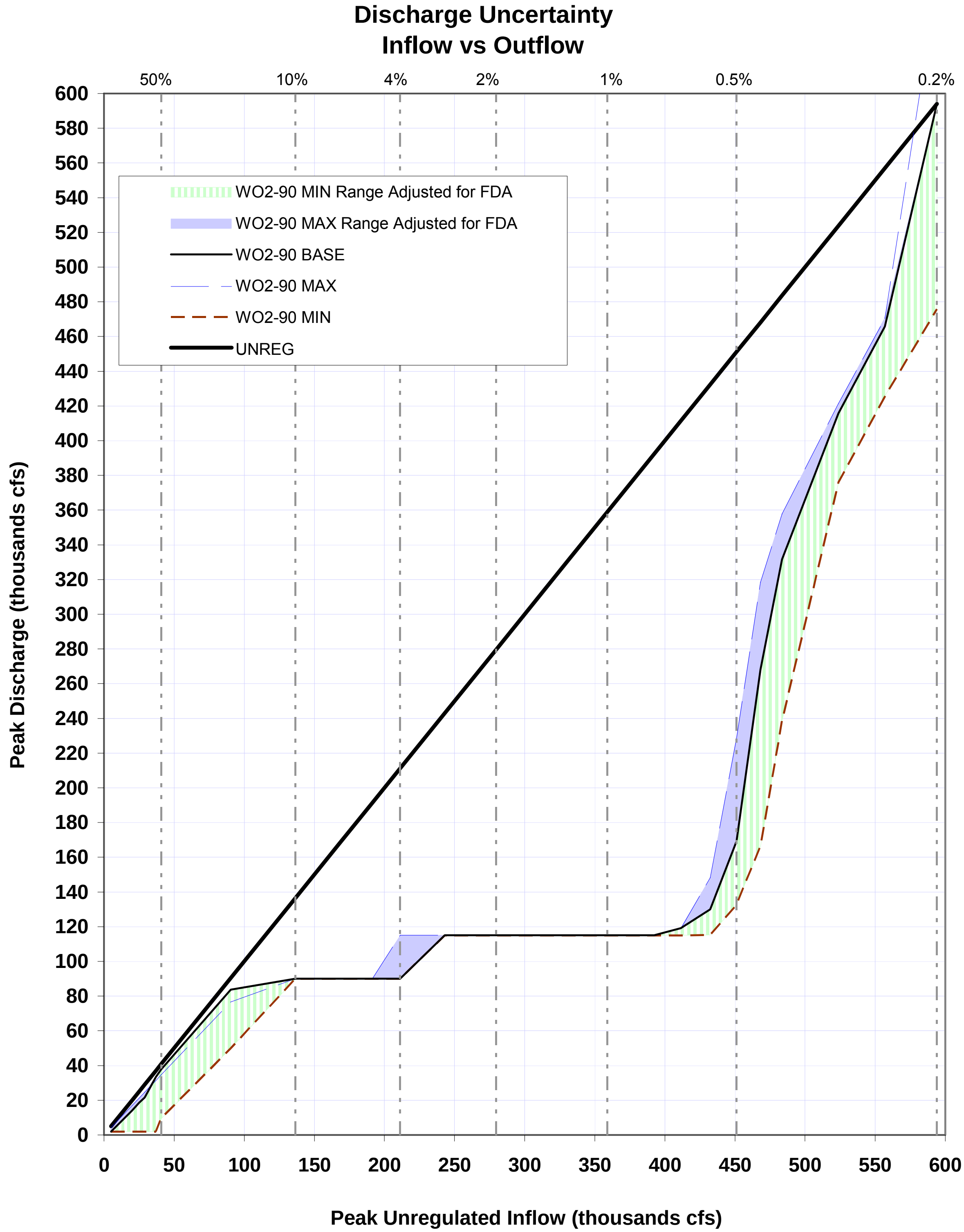


FIGURE A-16: DISCHARGE UNCERTAINTY – WO2 WITHOUT-PROJECT – 115,000 CFS

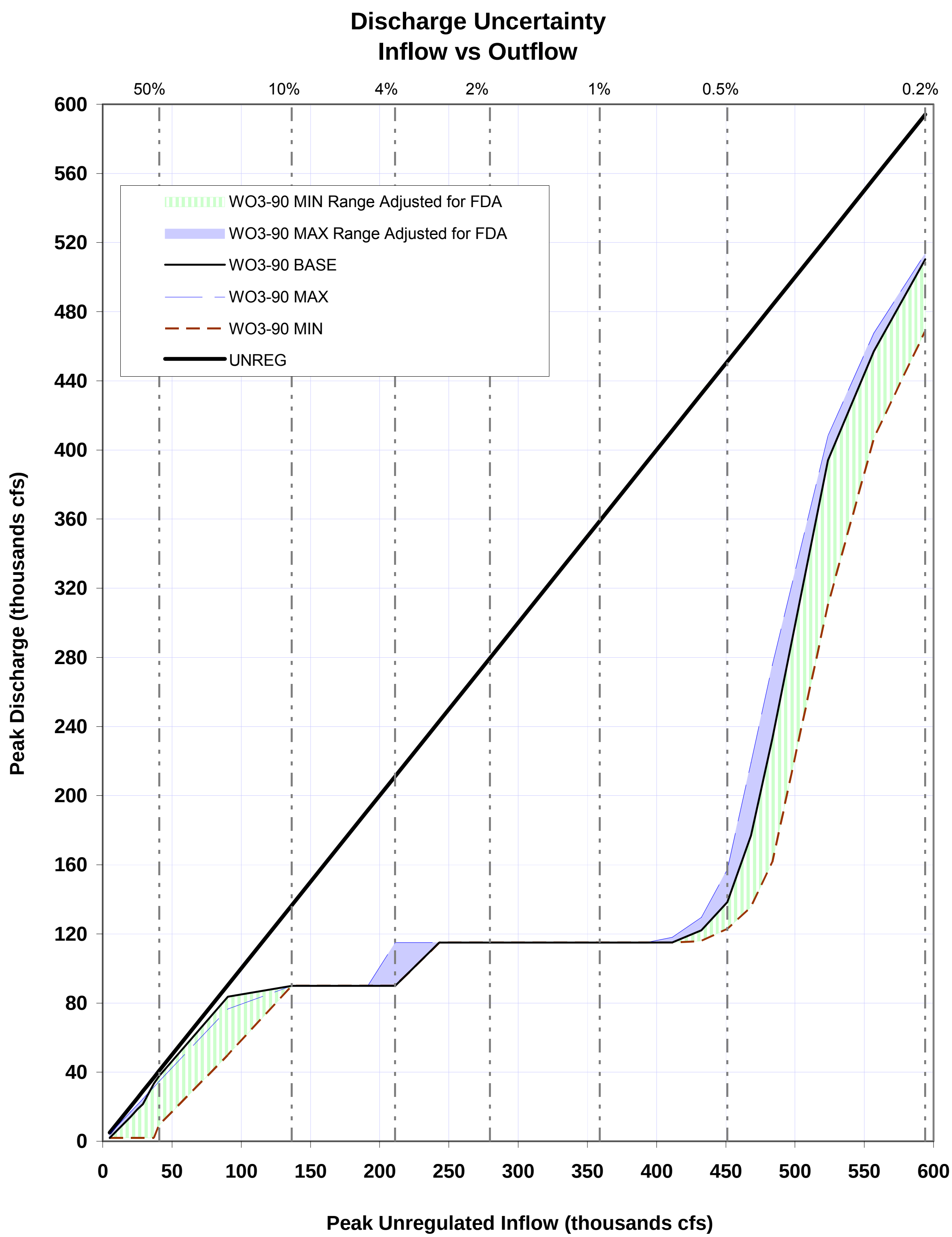


FIGURE A-17: DISCHARGE UNCERTAINTY – WO3 WITHOUT-PROJECT – 115,000 CFS

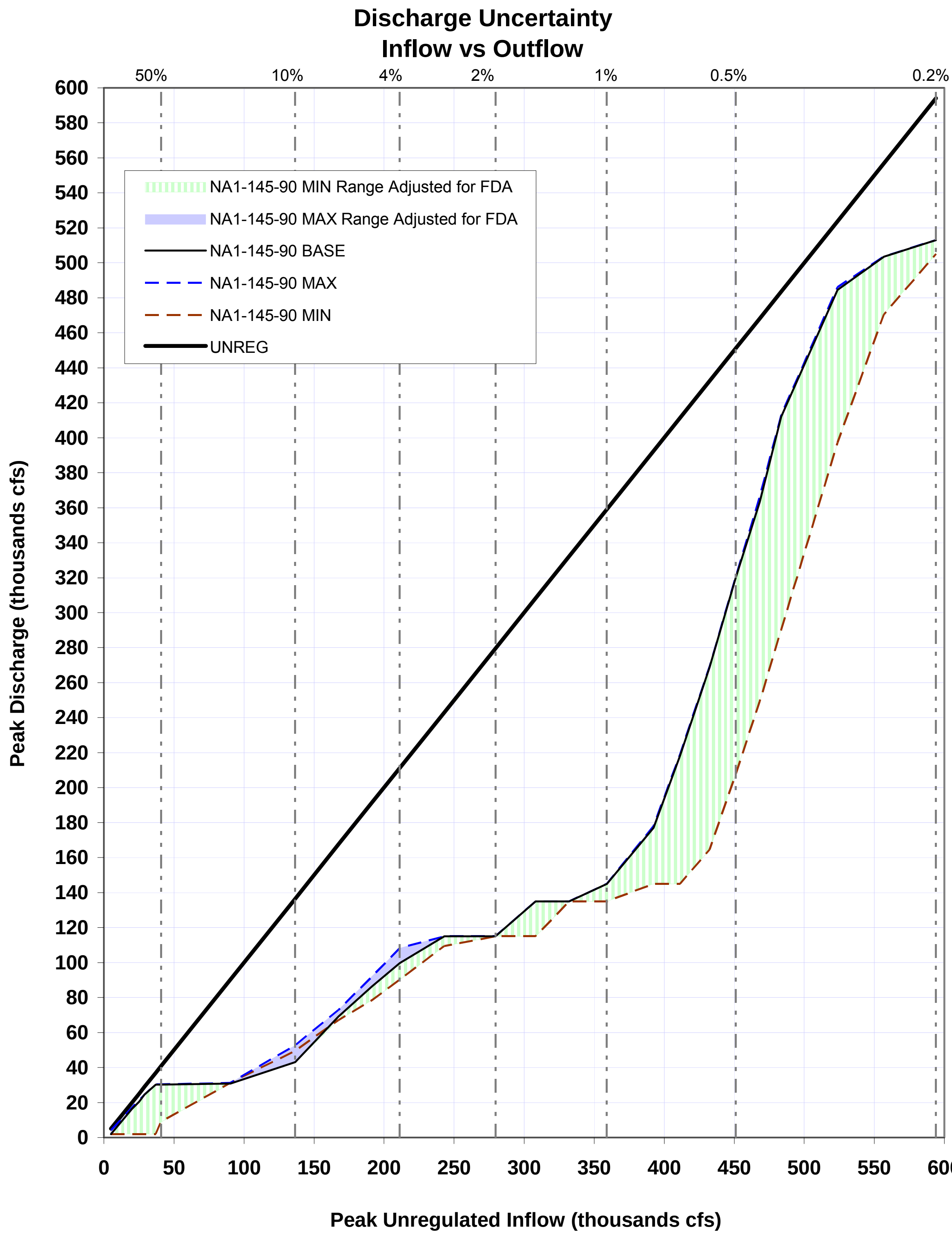


FIGURE A-18: DISCHARGE UNCERTAINTY – NA1 NO ACTION (FUTURE WITHOUT-PROJECT) – 145,000 CFS

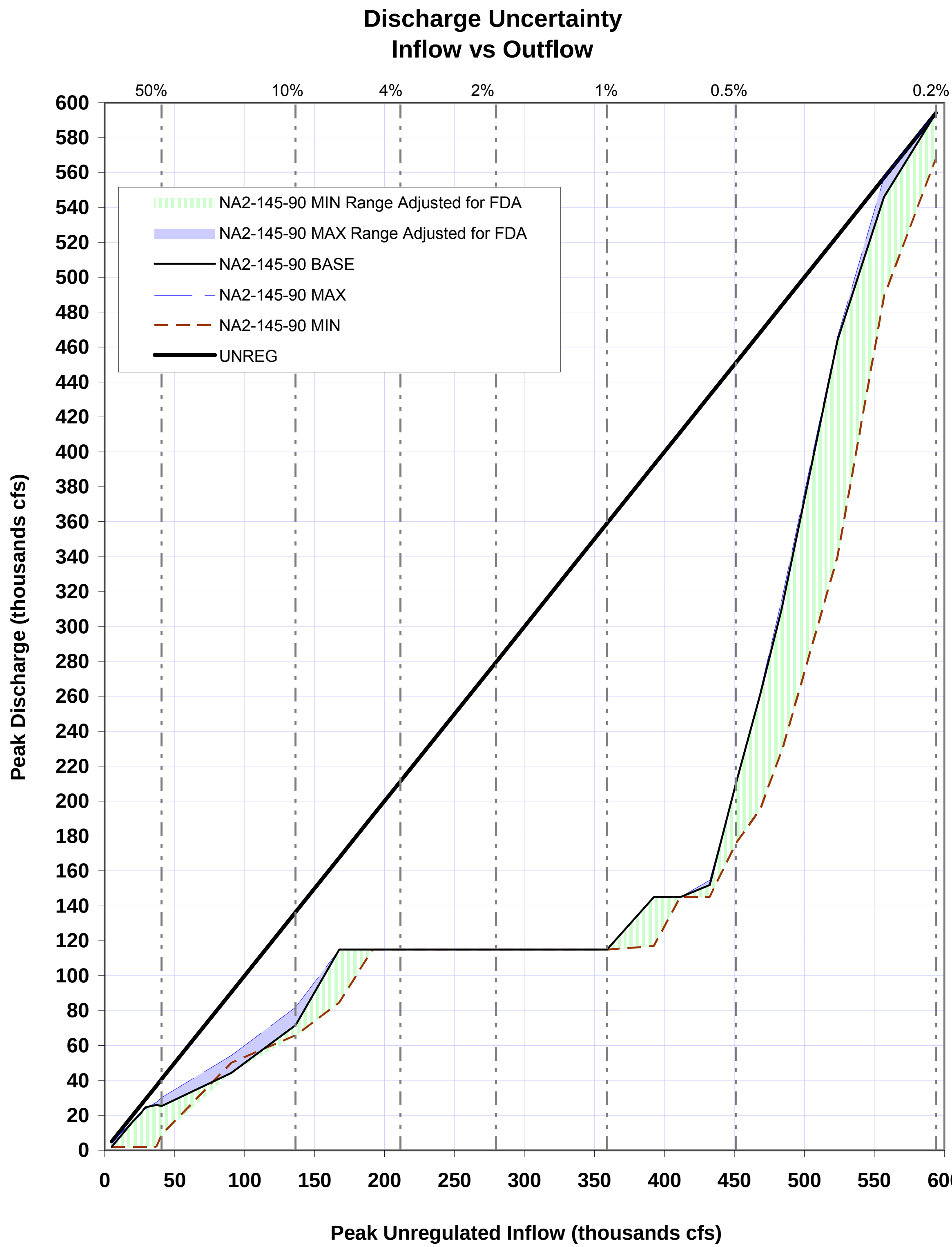


FIGURE A-19: DISCHARGE UNCERTAINTY – NA2 NO ACTION (FUTURE WITHOUT-PROJECT) – 145,000 CFS

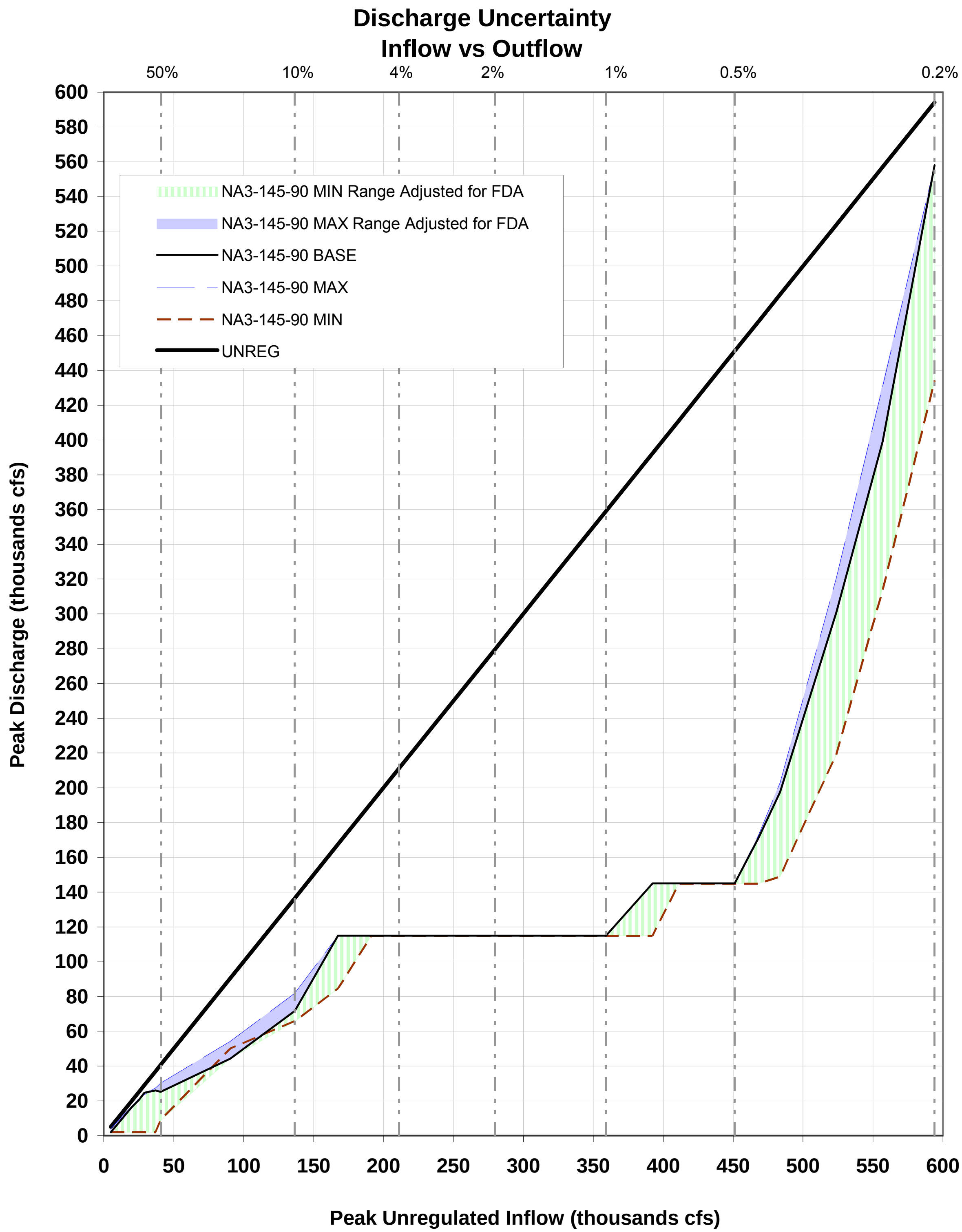


FIGURE A-20: DISCHARGE UNCERTAINTY – NA3 NO ACTION (FUTURE WITHOUT-PROJECT) – 145,000 CFS

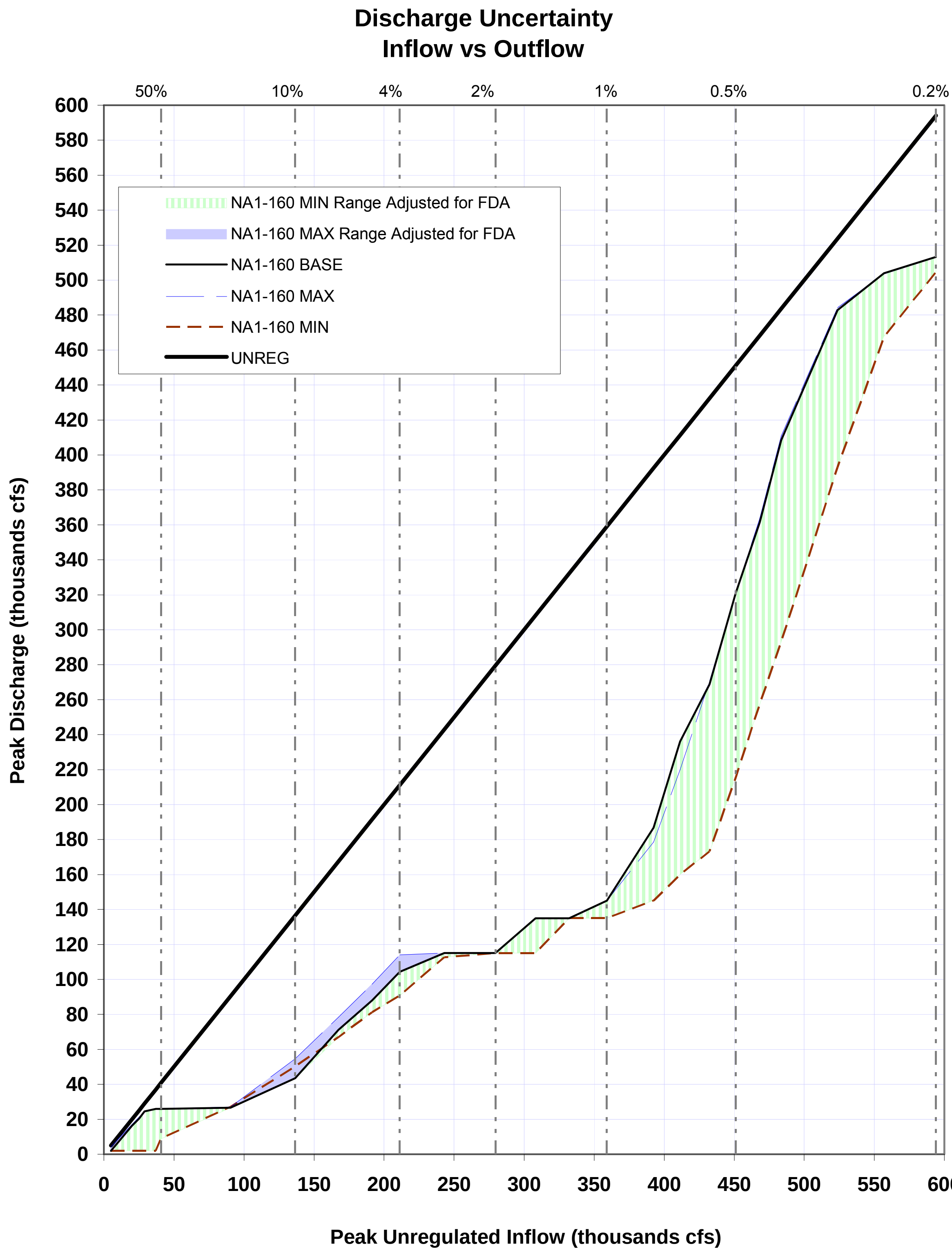


FIGURE A-21: DISCHARGE UNCERTAINTY – W1 WITH-PROJECT – 160,000 CFS

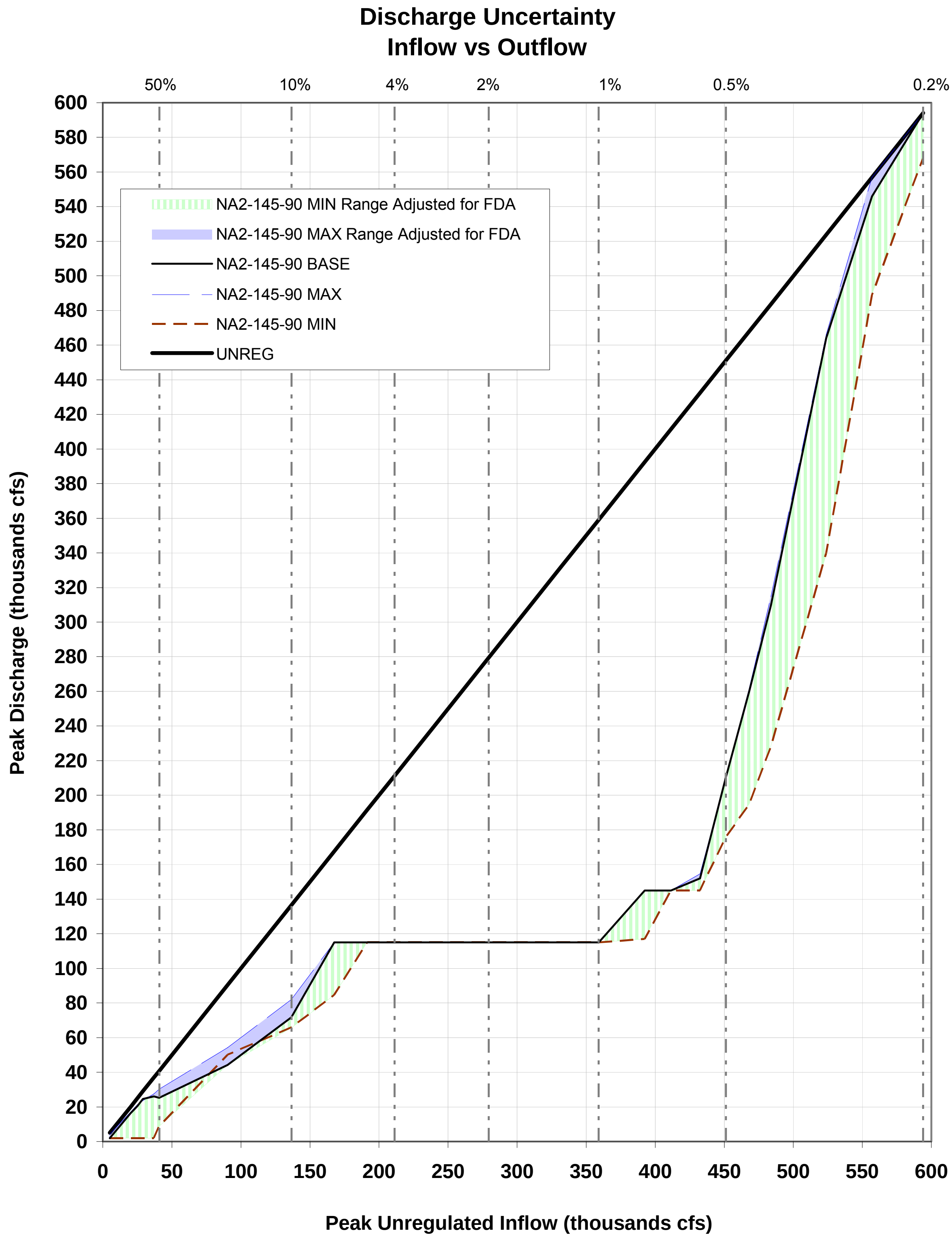


FIGURE A-22: DISCHARGE UNCERTAINTY – W2 WITH-PROJECT – 160,000 CFS

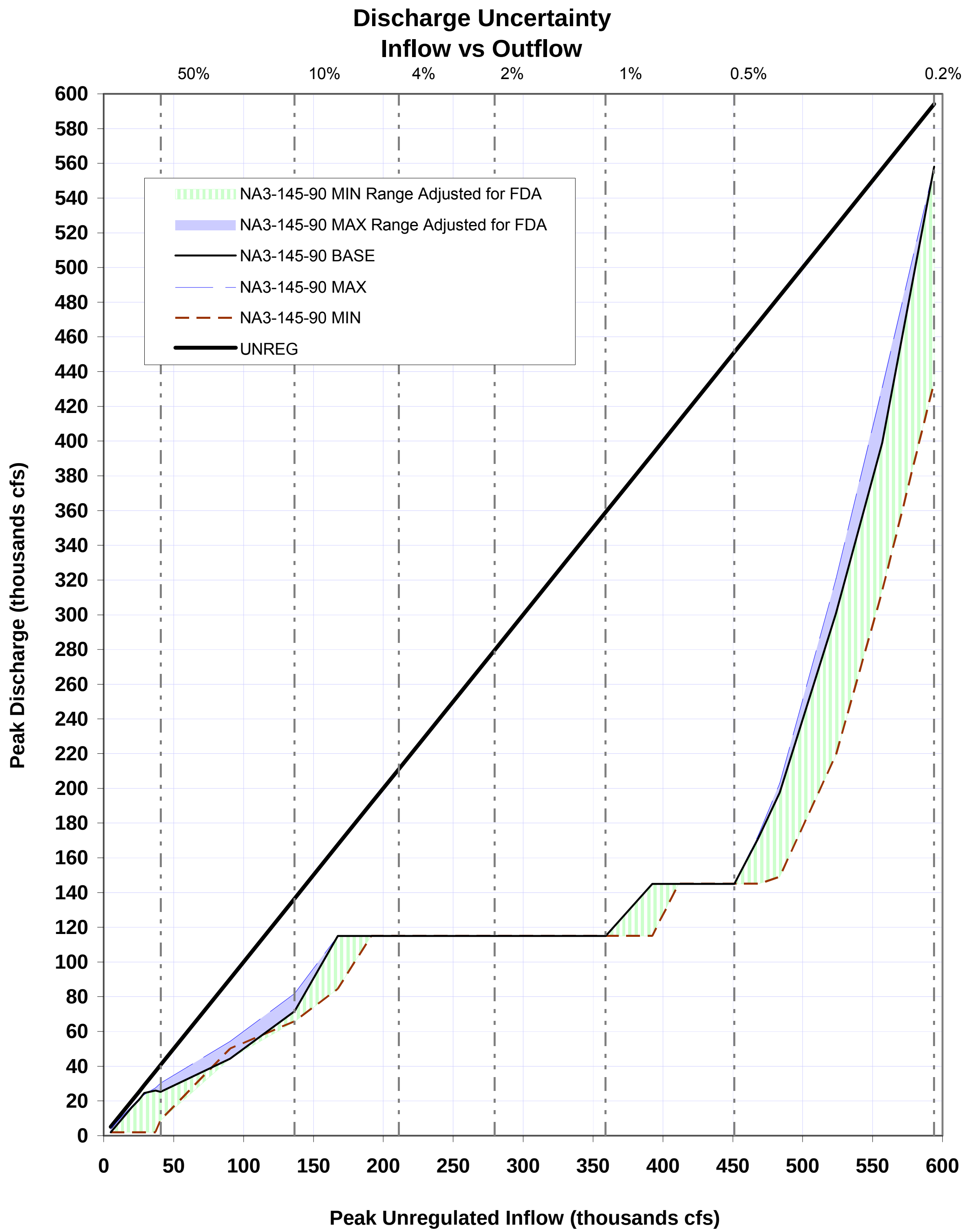


FIGURE A-23: DISCHARGE UNCERTAINTY – W3 WITH-PROJECT – 160,000 CFS

TABLE A-5: W01 BASE (R000_800CF_No Fix_115_FP470_P1_20080914)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.83	391.87	20002	20002	14057	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.49	389.60	25004	25004	18558	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	403.38	395.66	29000	29000	21525	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.75	398.21	37002	37002	30284	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.93	399.44	40722	40722	30295	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	416.82	494.78	90369	90369	30928	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	429.80	602.86	136522	136522	43692	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	435.17	651.53	167533	167533	70490	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	439.45	691.72	191482	191482	87307	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	442.53	721.38	211227	211227	98760	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	448.80	783.74	243016	243016	115000	0	0.00	0	0	0	23	23	0	0	0	0	0	0
50	457.34	872.50	279485	279485	115000	0	0.00	0	0	0	50	50	0	0	0	0	0	0
65	464.38	948.85	308218	308218	115000	0	0.00	0	0	0	68	68	0	0	0	0	0	0
80	470.37	1016.12	332148	332148	115000	0	0.00	0	0	33	84	84	0	0	0	0	0	0
100	**476.35**	**1085.39**	**359078**	**359078**	**123107**	**0**	**0.00**	**0**	**0**	**52**	**105**	**91**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.79	1078.77	392399	392399	222593	0	0.00	0	0	46	100	85	22	0	14	0	20	0
150	476.38	1085.73	411351	411351	292965	0	0.00	0	0	42	96	81	24	0	18	0	-6	28
175	474.78	1066.96	432395	432395	403445	0	0.00	0	0	27	90	74	27	0	21	10	56	146
200	**476.33**	**1085.18**	**451163**	**451163**	**444310**	**0**	**0.00**	**0**	**0**	**27**	**92**	**76**	**29**	**0**	**23**	**12**	**-6**	**88**
225	476.68	1089.26	468139	468139	461029	0	0.00	0	0	28	94	78	33	0	25	14	74	70
250	476.65	1088.92	483665	483665	476319	0	0.00	0	0	28	93	78	34	0	27	16	68	100
325	477.59	1099.94	523757	523757	515802	0	0.00	0	0	31	97	83	38	0	31	20	55	101
400	478.55	1111.42	556967	556967	546433	0	0.00	0	0	34	100	88	42	0	35	22	38	96
500	479.62	1124.16	594159	594159	554268	0	0.00	0	0	39	109	93	48	0	40	26	31	85

TABLE A-6: W01 MAX (R000_800CF_No Fix_115_FP470_P1_20080914)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.76	419.28	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.90	420.25	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	407.01	421.03	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.22	422.55	37002	37002	30425	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.31	423.19	40722	40722	30425	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	417.33	498.81	90369	90369	31248	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	429.70	601.95	136522	136522	52675	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	435.68	656.20	167533	167533	72904	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	440.67	703.42	191482	191482	92040	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	444.39	739.61	211227	211227	108290	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	450.77	803.74	243016	243016	115000	0	0.00	0	0	0	30	30	0	0	0	0	0	0
50	458.92	889.35	279485	279485	115000	0	0.00	0	0	0	54	54	0	0	0	0	0	0
65	465.45	960.66	308218	308218	115000	0	0.00	0	0	0	71	71	0	0	0	0	0	0
80	470.97	1022.96	332148	332148	115000	0	0.00	0	0	36	86	86	0	0	0	0	0	0
100	**476.32**	**1085.03**	**359078**	**359078**	**124034**	**0**	**0.00**	**0**	**0**	**52**	**105**	**91**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.79	1078.80	392399	392399	222320	0	0.00	0	0	46	100	85	22	0	14	0	20	0
150	476.39	1085.81	411351	411351	293316	0	0.00	0	0	42	96	81	24	0	18	0	-6	28
175	474.87	1068.07	432395	432395	411752	0	0.00	0	0	26	90	74	27	0	21	10	57	150
200	**476.37**	**1085.67**	**451163**	**451163**	**444310**	**0**	**0.00**	**0**	**0**	**28**	**93**	**77**	**29**	**0**	**23**	**12**	**-6**	**89**
225	476.67	1089.18	468139	468139	461029	0	0.00	0	0	28	94	78	33	0	25	14	68	70
250	476.66	1089.00	483665	483665	476319	0	0.00	0	0	28	94	78	34	0	27	16	49	99
325	477.74	1101.76	523757	523757	515802	0	0.00	0	0	31	97	83	38	0	32	20	53	95
400	478.68	1112.95	556967	556967	548181	0	0.00	0	0	36	101	88	42	0	36	22	32	98
500	479.76	1125.81	594159	594159	554678	0	0.00	0	0	39	111	93	49	0	40	26	53	81

TABLE A-7: W01 MIN (R000_800CF_No Fix_115_FP470_P1_20080914)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.84	464.05	40722	38674	9546	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	417.31	498.68	90369	84059	31233	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	428.58	592.07	136522	126249	49521	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	434.39	644.26	167533	154598	68674	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	437.99	677.88	191482	176491	81089	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	440.84	705.03	211227	194541	92975	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	445.30	748.65	243016	223601	111456	0	0.00	0	0	0	0	0	0	0	0	0	0	0
50	451.35	809.74	279485	256938	115000	0	0.00	0	0	0	32	32	0	0	0	0	0	0
65	456.85	867.28	308218	283204	115000	0	0.00	0	0	0	50	50	0	0	0	0	0	0
80	461.62	918.59	332148	305080	115000	0	0.00	0	0	0	62	62	0	0	0	0	0	0
100	**466.62**	**973.78**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**14**	**75**	**75**	**0**	**0**	**0**	**0**	**0**	**0**
130	474.76	1066.72	392399	360850	115000	0	0.00	0	0	49	105	94	0	0	0	0	0	0
150	476.90	1091.89	411351	381289	135890	0	0.00	0	0	54	116	104	0	0	0	0	0	0
175	475.90	1080.08	432395	407507	203141	0	0.00	0	0	48	105	92	22	0	13	0	29	0
200	**475.92**	**1080.38**	**451163**	**429875**	**277213**	**0**	**0.00**	**0**	**0**	**41**	**97**	**84**	**25**	**0**	**18**	**0**	**73**	**27**
225	475.85	1079.48	468139	448786	314895	0	0.00	0	0	36	97	83	27	0	21	10	97	0
250	475.00	1069.51	483665	465352	435147	0	0.00	0	0	23	89	75	29	0	23	11	162	58
325	475.98	1080.98	523757	506439	499294	0	0.00	0	0	23	92	77	33	0	27	15	-6	147
400	476.25	1084.24	556967	540033	509928	0	0.00	0	0	27	96	81	39	0	31	19	106	156
500	477.41	1097.86	594159	577133	529188	0	0.00	0	0	31	104	88	45	0	36	22	70	132

TABLE A-8: W02 BASE (R060_800FM_No Fix_115_FP470_P1_20080908)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.83	391.87	20002	20002	14057	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.49	389.60	25004	25004	18558	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	403.38	395.66	29000	29000	21525	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.67	397.61	37002	37002	33505	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.53	396.67	40722	40722	37708	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	403.99	399.85	90369	90369	83680	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	408.97	435.18	136522	136522	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	415.21	482.19	167533	167533	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	421.58	532.97	191482	191482	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	427.80	585.26	211227	211227	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	433.71	638.02	243016	243016	115000	0	0.00	0	0	0	29	29	0	0	0	0	0	0
50	443.02	726.21	279485	279485	115000	0	0.00	0	0	0	54	54	0	0	0	0	0	0
65	449.41	789.86	308218	308218	115000	0	0.00	0	0	0	68	68	0	0	0	0	0	0
80	454.76	845.21	332148	332148	115000	0	0.00	0	0	0	81	81	0	0	0	0	0	0
100	**461.00**	**911.80**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**102**	**102**	**0**	**0**	**0**	**0**	**0**	**0**
130	467.81	987.06	392399	392399	115000	0	0.00	0	0	23	127	127	0	0	0	0	0	0
150	472.83	1044.29	411351	411351	115000	0	0.00	0	0	43	138	126	0	0	0	0	0	0
175	476.38	1085.74	432395	432395	129972	0	0.00	0	0	54	147	133	0	0	0	0	0	0
200	**476.65**	**1088.93**	**451163**	**451163**	**169173**	**0**	**0.00**	**0**	**0**	**152**	**53**	**14**	**20**	**0**	**0**	**0**	**3**	**0**
225	474.79	1067.11	468139	468139	268061	0	0.00	0	0	80	56	15	24	0	17	0	55	0
250	475.23	1072.29	483665	483665	331691	0	0.00	0	0	62	58	14	25	0	19	8	-5	45
325	478.02	1105.13	523757	523757	415711	0	0.00	0	0	62	62	15	30	0	24	13	-6	80
400	479.76	1125.81	556967	556967	465830	0	0.00	0	0	63	65	15	34	0	28	16	-5	80
500	480.97	1140.34	594159	594159	627077	25	0.47	0	0	47	68	13	40	0	32	20	-2	77

TABLE A-9: W02 MAX (R060_800FM_No Fix_115_FP470_P1_20080908)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.76	419.28	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.90	420.25	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	407.01	421.03	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.22	422.55	37002	37002	31387	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.31	423.19	40722	40722	34542	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	408.59	432.46	90369	90369	76656	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	412.41	460.77	136522	136522	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	418.11	505.04	167533	167533	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	423.57	549.39	191482	191482	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	428.52	591.54	211227	211227	115000	0	0.00	0	0	0	10	10	0	0	0	0	0	0
35	434.66	646.79	243016	243016	115000	0	0.00	0	0	0	32	32	0	0	0	0	0	0
50	443.71	732.89	279485	279485	115000	0	0.00	0	0	0	55	55	0	0	0	0	0	0
65	450.81	804.21	308218	308218	115000	0	0.00	0	0	0	72	72	0	0	0	0	0	0
80	456.24	860.80	332148	332148	115000	0	0.00	0	0	0	86	86	0	0	0	0	0	0
100	**463.05**	**934.22**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**113**	**113**	**0**	**0**	**0**	**0**	**0**	**0**
130	470.95	1022.74	392399	392399	115000	0	0.00	0	0	37	135	135	0	0	0	0	0	0
150	475.46	1074.96	411351	411351	118897	0	0.00	0	0	51	146	130	0	0	0	0	0	0
175	476.56	1087.80	432395	432395	148180	0	0.00	0	0	53	149	132	0	0	0	0	0	0
200	**474.94**	**1068.80**	**451163**	**451163**	**228405**	**0**	**0.00**	**0**	**0**	**99**	**57**	**17**	**22**	**0**	**15**	**0**	**53**	**0**
225	474.96	1069.02	468139	468139	318472	0	0.00	0	0	63	59	17	26	0	18	6	64	38
250	476.00	1081.29	483665	483665	357755	0	0.00	0	0	59	60	16	26	0	20	9	-5	57
325	478.22	1107.42	523757	523757	421382	0	0.00	0	0	63	64	16	31	0	25	14	-6	80
400	479.93	1127.86	556967	556967	470310	0	0.00	0	0	62	66	15	35	0	29	17	-5	77
500	481.08	1141.63	594159	594159	663803	33	0.58	0	0	47	70	14	41	0	33	21	-2	78

TABLE A-10: W02 MIN (R060_800FM_No Fix_115_FP470_P1_20080908)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.84	464.05	40722	38674	9546	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	415.33	483.15	90369	84059	50000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	418.32	506.69	136522	126249	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	421.73	534.18	167533	154598	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	425.96	569.46	191482	176491	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	429.85	603.33	211227	194541	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	434.59	646.18	243016	223601	115000	0	0.00	0	0	0	22	22	0	0	0	0	0	0
50	441.93	715.57	279485	256938	115000	0	0.00	0	0	0	46	46	0	0	0	0	0	0
65	448.57	781.34	308218	283204	115000	0	0.00	0	0	0	63	63	0	0	0	0	0	0
80	453.72	834.34	332148	305080	115000	0	0.00	0	0	0	79	79	0	0	0	0	0	0
100	**460.26**	**903.83**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**100**	**100**	**0**	**0**	**0**	**0**	**0**	**0**
130	467.01	978.09	392399	360850	115000	0	0.00	0	0	16	127	127	0	0	0	0	0	0
150	469.43	1005.36	411351	381289	115000	0	0.00	0	0	32	134	134	0	0	0	0	0	0
175	474.36	1062.07	432395	407507	115124	0	0.00	0	0	50	147	130	0	0	0	0	0	0
200	**476.58**	**1088.06**	**451163**	**429875**	**132562**	**0**	**0.00**	**0**	**0**	**55**	**152**	**136**	**0**	**0**	**0**	**0**	**0**	**0**
225	476.63	1088.71	468139	448786	166144	0	0.00	0	0	155	55	15	20	0	0	0	3	0
250	474.88	1068.11	483665	465352	238763	0	0.00	0	0	95	57	16	24	0	17	0	50	15
325	476.43	1086.35	523757	506439	375796	0	0.00	0	0	55	61	15	28	0	22	10	-6	68
400	478.35	1108.97	556967	540033	425258	0	0.00	0	0	57	64	15	33	0	26	14	-1	84
500	480.11	1129.97	594159	577133	475823	0	0.00	0	0	47	68	15	39	0	30	17	-2	83

TABLE A-11: W03 BASE (R060_800DR3.5e_115_FP470_P1_20080907)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.83	391.87	20002	20002	14057	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.49	389.60	25004	25004	18558	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	403.38	395.66	29000	29000	21525	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.67	397.61	37002	37002	33505	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.53	396.67	40722	40722	37708	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	403.99	399.85	90369	90369	83680	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	408.97	435.18	136522	136522	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	415.21	482.19	167533	167533	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	421.58	532.97	191482	191482	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	427.80	585.26	211227	211227	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	433.71	638.02	243016	243016	115000	0	0.00	0	0	0	29	29	0	0	0	0	0	0
50	443.02	726.21	279485	279485	115000	0	0.00	0	0	0	54	54	0	0	0	0	0	0
65	449.41	789.86	308218	308218	115000	0	0.00	0	0	0	68	68	0	0	0	0	0	0
80	454.76	845.21	332148	332148	115000	0	0.00	0	0	0	81	81	0	0	0	0	0	0
100	**461.00**	**911.80**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**102**	**102**	**0**	**0**	**0**	**0**	**0**	**0**
130	467.81	987.06	392399	392399	115000	0	0.00	0	0	23	127	127	0	0	0	0	0	0
150	472.81	1044.11	411351	411351	115088	0	0.00	0	0	43	138	126	0	0	0	0	0	0
175	476.78	1090.42	432395	432395	122131	0	0.00	0	0	55	149	135	0	0	0	0	0	0
200	**478.67**	**1112.80**	**451163**	**451163**	**138359**	**0**	**0.00**	**0**	**0**	**60**	**154**	**140**	**0**	**0**	**0**	**0**	**0**	**0**
225	478.38	1109.31	468139	468139	176952	0	0.00	0	0	162	56	15	24	0	0	0	15	0
250	477.27	1096.17	483665	483665	232803	0	0.00	0	0	150	58	14	25	0	18	0	34	0
325	477.72	1101.50	523757	523757	394043	0	0.00	0	0	72	62	15	30	0	24	13	-6	75
400	479.49	1122.65	556967	556967	457102	0	0.00	0	0	76	65	15	34	0	28	16	-5	74
500	481.31	1144.50	594159	594159	510279	0	0.00	0	0	77	68	14	40	0	32	20	-2	75
PMF	486.00	1201.47	905770	905770	1105372	214	2.00	0	0	131	134	18	93	0	61	49	247	146

TABLE A-12: W03 MAX (R060_800DR3.5e_115_FP470_P1_20080907)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.76	419.28	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.90	420.25	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	407.01	421.03	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.22	422.55	37002	37002	31387	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.31	423.19	40722	40722	34542	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	408.59	432.46	90369	90369	76656	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	412.41	460.77	136522	136522	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	418.11	505.04	167533	167533	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	423.57	549.39	191482	191482	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	428.52	591.54	211227	211227	115000	0	0.00	0	0	0	10	10	0	0	0	0	0	0
35	434.66	646.79	243016	243016	115000	0	0.00	0	0	0	32	32	0	0	0	0	0	0
50	443.71	732.89	279485	279485	115000	0	0.00	0	0	0	55	55	0	0	0	0	0	0
65	450.81	804.21	308218	308218	115000	0	0.00	0	0	0	72	72	0	0	0	0	0	0
80	456.24	860.80	332148	332148	115000	0	0.00	0	0	0	86	86	0	0	0	0	0	0
100	**463.05**	**934.22**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**113**	**113**	**0**	**0**	**0**	**0**	**0**	**0**
130	470.95	1022.74	392399	392399	115000	0	0.00	0	0	37	135	135	0	0	0	0	0	0
150	475.37	1073.85	411351	411351	118010	0	0.00	0	0	51	146	130	0	0	0	0	0	0
175	477.72	1101.61	432395	432395	129653	0	0.00	0	0	57	152	135	0	0	0	0	0	0
200	**478.72**	**1113.42**	**451163**	**451163**	**157288**	**0**	**0.00**	**0**	**0**	**60**	**156**	**138**	**0**	**0**	**0**	**0**	**0**	**0**
225	477.50	1098.98	468139	468139	219082	0	0.00	0	0	151	59	18	25	0	17	0	24	0
250	476.79	1090.52	483665	483665	275466	0	0.00	0	0	105	60	16	26	0	20	0	53	7
325	478.10	1106.01	523757	523757	408380	0	0.00	0	0	73	64	16	31	0	25	13	74	61
400	479.87	1127.11	556967	556967	467639	0	0.00	0	0	77	66	15	35	0	29	16	71	59
500	481.43	1147.07	594159	594159	513811	0	0.00	0	0	78	70	15	41	0	33	21	-2	72

TABLE A-13: W03 MIN (R060_800DR3.5e_115_FP470_P1_20080907)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.84	464.05	40722	38674	9546	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	415.33	483.15	90369	84059	50000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	418.32	506.69	136522	126249	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	421.73	534.18	167533	154598	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	425.96	569.46	191482	176491	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	429.85	603.33	211227	194541	90000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	434.59	646.18	243016	223601	115000	0	0.00	0	0	0	22	22	0	0	0	0	0	0
50	441.93	715.57	279485	256938	115000	0	0.00	0	0	0	46	46	0	0	0	0	0	0
65	448.57	781.34	308218	283204	115000	0	0.00	0	0	0	63	63	0	0	0	0	0	0
80	453.72	834.34	332148	305080	115000	0	0.00	0	0	0	79	79	0	0	0	0	0	0
100	**460.26**	**903.83**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**100**	**100**	**0**	**0**	**0**	**0**	**0**	**0**
130	467.01	978.09	392399	360850	115000	0	0.00	0	0	16	127	127	0	0	0	0	0	0
150	469.43	1005.36	411351	381289	115000	0	0.00	0	0	32	134	134	0	0	0	0	0	0
175	474.21	1060.36	432395	407507	115938	0	0.00	0	0	49	146	128	0	0	0	0	0	0
200	**477.18**	**1095.22**	**451163**	**429875**	**122877**	**0**	**0.00**	**0**	**0**	**57**	**154**	**138**	**0**	**0**	**0**	**0**	**0**	**0**
225	478.80	1114.32	468139	448786	135427	0	0.00	0	0	61	158	143	0	0	0	0	0	0
250	479.05	1117.37	483665	465352	161583	0	0.00	0	0	171	57	16	23	0	0	0	0	0
325	476.54	1087.58	523757	506439	310409	0	0.00	0	0	83	61	15	28	0	22	9	55	38
400	478.05	1105.51	556967	540033	406670	0	0.00	0	0	64	64	15	33	0	26	14	-1	78
500	479.89	1127.38	594159	577133	468932	0	0.00	0	0	68	68	16	39	0	30	16	77	70

TABLE A-14: NA1-145 BASE (R000_800CF_No Fix_145_FP470_P1_20080919)																			
1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.39	388.94	20002	20002	16328	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.53	389.84	25004	25004	20411	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	402.78	391.51	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.17	394.19	37002	37002	30237	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	402.43	389.16	40722	40722	30183	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	415.74	486.34	90369	90369	30848	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	429.13	596.95	136522	136522	43127	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	434.65	646.67	167533	167533	69092	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	439.01	687.48	191482	191482	86500	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	442.69	722.96	211227	211227	99738	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	448.44	780.03	243016	243016	115000	0	0.00	0	0	0	22	22	0	0	0	0	0	0
50	457.01	868.92	279485	279485	115000	0	0.00	0	0	0	49	49	0	0	0	0	0	0
65	461.09	912.80	308218	308218	135000	0	0.00	0	0	0	61	61	0	0	0	0	0	0
80	466.44	971.77	332148	332148	135000	0	0.00	0	0	9	74	74	0	0	0	0	0	0
100	**470.81**	**1021.10**	**359078**	**359078**	**145000**	**0**	**0.00**	**0**	**0**	**39**	**92**	**92**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.31	1073.15	392399	392399	177012	0	0.00	0	0	49	105	105	20	1	0	0	14	0
150	475.74	1078.26	411351	411351	218286	0	0.00	0	0	49	106	106	23	0	16	0	18	0
175	476.07	1082.12	432395	432395	268700	0	0.00	0	0	47	104	103	25	0	19	0	-6	16
200	**476.40**	**1086.00**	**451163**	**451163**	**320142**	**0**	**0.00**	**0**	**0**	**45**	**104**	**102**	**28**	**0**	**21**	**11**	**65**	**42**
225	476.58	1088.08	468139	468139	363164	0	0.00	0	0	41	103	102	31	0	24	12	48	42
250	476.67	1089.18	483665	483665	412114	0	0.00	0	0	36	101	99	32	0	26	14	-5	47
325	477.17	1095.00	523757	523757	484550	0	0.00	0	0	35	102	101	37	0	30	18	56	49
400	477.83	1102.80	556967	556967	503557	0	0.00	0	0	38	107	107	41	0	35	21	37	46
500	479.01	1116.85	594159	594159	512982	0	0.00	0	0	42	129	115	47	0	39	25	30	52

TABLE A-15: NA1-145 MAX (R000_800CF_No Fix_145_FP470_P1_20080919)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.74	419.09	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.86	419.97	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	406.97	420.78	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.19	422.35	37002	37002	30423	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.29	423.06	40722	40722	30424	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	417.33	498.81	90369	90369	31248	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	429.70	601.95	136522	136522	52675	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	435.68	656.20	167533	167533	72904	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	440.67	703.42	191482	191482	92040	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	444.39	739.61	211227	211227	108290	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	450.77	803.74	243016	243016	115000	0	0.00	0	0	0	30	30	0	0	0	0	0	0
50	458.92	889.35	279485	279485	115000	0	0.00	0	0	0	54	54	0	0	0	0	0	0
65	462.29	925.84	308218	308218	135000	0	0.00	0	0	0	65	65	0	0	0	0	0	0
80	467.20	980.23	332148	332148	135000	0	0.00	0	0	16	78	78	0	0	0	0	0	0
100	**471.07**	**1024.11**	**359078**	**359078**	**145000**	**0**	**0.00**	**0**	**0**	**40**	**92**	**92**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.32	1073.28	392399	392399	178071	0	0.00	0	0	48	105	105	20	0	0	0	13	0
150	475.75	1078.35	411351	411351	218943	0	0.00	0	0	49	106	106	23	0	17	0	14	0
175	476.08	1082.17	432395	432395	269028	0	0.00	0	0	47	104	103	25	0	19	0	-6	15
200	**476.41**	**1086.03**	**451163**	**451163**	**320618**	**0**	**0.00**	**0**	**0**	**45**	**104**	**102**	**28**	**0**	**22**	**11**	**56**	**42**
225	476.59	1088.23	468139	468139	366078	0	0.00	0	0	40	104	102	31	0	24	12	50	42
250	476.67	1089.19	483665	483665	413033	0	0.00	0	0	36	102	99	32	0	26	14	-5	46
325	477.19	1095.24	523757	523757	485904	0	0.00	0	0	35	102	101	37	0	31	18	45	47
400	477.84	1102.98	556967	556967	503676	0	0.00	0	0	39	108	108	41	0	35	21	36	47
500	479.02	1116.98	594159	594159	513077	0	0.00	0	0	42	126	115	48	0	39	25	44	52

TABLE A-16: NA1-145 MIN (R000_800CF_No Fix_145_FP470_P1_20080919)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.98	465.10	40722	38674	8916	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	417.13	497.21	90369	84059	31233	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	428.58	592.07	136522	126249	49521	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	433.77	638.60	167533	154598	67040	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	437.47	672.98	191482	176491	78462	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	440.35	700.34	211227	194541	90191	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	444.94	745.01	243016	223601	109419	0	0.00	0	0	0	0	0	0	0	0	0	0	0
50	451.06	806.80	279485	256938	115000	0	0.00	0	0	0	31	31	0	0	0	0	0	0
65	456.61	864.68	308218	283204	115000	0	0.00	0	0	0	48	48	0	0	0	0	0	0
80	458.74	887.48	332148	305080	135000	0	0.00	0	0	0	56	56	0	0	0	0	0	0
100	**463.26**	**936.53**	**359078**	**329825**	**135000**	**0**	**0.00**	**0**	**0**	**0**	**68**	**68**	**0**	**0**	**0**	**0**	**0**	**0**
130	468.87	998.98	392399	360850	145000	0	0.00	0	0	26	84	84	0	0	0	0	0	0
150	472.83	1044.26	411351	381289	145000	0	0.00	0	0	47	103	103	0	0	0	0	0	0
175	475.66	1077.28	432395	407507	164525	0	0.00	0	0	52	118	118	14	1	0	0	2	0
200	**475.58**	**1076.33**	**451163**	**429875**	**207706**	**0**	**0.00**	**0**	**0**	**50**	**117**	**117**	**23**	**0**	**17**	**0**	**4**	**0**
225	475.81	1079.03	468139	448786	249188	0	0.00	0	0	48	118	118	25	0	19	0	-5	5
250	476.10	1082.45	483665	465352	290568	0	0.00	0	0	45	106	106	27	0	21	0	-5	21
325	476.63	1088.66	523757	506439	397225	0	0.00	0	0	36	105	104	32	0	26	13	-6	92
400	476.97	1092.73	556967	540033	470504	0	0.00	0	0	32	104	103	37	0	30	18	-1	63
500	478.01	1105.02	594159	577133	505120	0	0.00	0	0	36	121	110	43	0	34	21	-2	50

TABLE A-17: NA2-145 BASE (R060_800FM_No Fix_145_FP466_P1_20080916)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.39	388.94	20002	20002	16328	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.53	389.84	25004	25004	20411	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	402.78	391.51	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.59	397.11	37002	37002	26005	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.18	394.30	40722	40722	25215	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	413.74	470.92	90369	90369	44261	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	421.65	533.58	136522	136522	71655	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	424.92	560.66	167533	167533	115000	0	0.00	0	0	0	26	26	0	0	0	0	0	0
20	428.02	587.24	191482	191482	115000	0	0.00	0	0	0	36	36	0	0	0	0	0	0
25	431.43	617.37	211227	211227	115000	0	0.00	0	0	0	45	45	0	0	0	0	0	0
35	437.15	669.98	243016	243016	115000	0	0.00	0	0	0	57	57	0	0	0	0	0	0
50	442.97	725.68	279485	279485	115000	0	0.00	0	0	0	75	75	0	0	0	0	0	0
65	449.11	786.85	308218	308218	115000	0	0.00	0	0	0	103	103	0	0	0	0	0	0
80	453.74	834.47	332148	332148	115000	0	0.00	0	0	0	127	127	0	0	0	0	0	0
100	**460.46**	**906.02**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**137**	**137**	**0**	**0**	**0**	**0**	**0**	**0**
130	461.49	917.22	392399	392399	145000	0	0.00	0	0	0	144	125	0	0	0	0	0	0
150	466.34	970.57	411351	411351	145000	0	0.00	0	0	11	153	134	0	0	0	0	0	0
175	469.94	1011.13	432395	432395	151924	0	0.00	0	0	33	160	142	0	0	0	0	0	0
200	**470.02**	**1012.12**	**451163**	**451163**	**210332**	**0**	**0.00**	**0**	**0**	**25**	**160**	**142**	**21**	**0**	**15**	**0**	**0**	**0**
225	470.31	1015.33	468139	468139	260498	0	0.00	0	0	21	160	143	25	0	18	0	-5	19
250	470.65	1019.31	483665	483665	309673	0	0.00	0	0	18	160	144	26	0	20	9	-5	35
325	471.23	1025.87	523757	523757	464074	0	0.00	0	0	13	183	144	30	0	24	13	-6	111
400	471.61	1030.22	556967	556967	545951	0	0.00	0	0	12	189	146	34	0	28	16	-5	176
500	472.08	1035.62	594159	594159	594159	0	0.00	0	0	14	196	150	40	0	32	20	-2	153
PMF	477.51	1099.03	905770	905770	812199	0	0.00	0	0	57	255	129	85	0	55	43	0	146

TABLE A-18: NA2-145 MAX (R060_800FM_No Fix_145_FP466_P1_20080916)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.74	419.09	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.86	419.97	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	406.97	420.78	29000	29000	23588	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.19	422.35	37002	37002	27464	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.29	423.06	40722	40722	30225	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	414.02	473.00	90369	90369	54221	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	423.56	549.32	136522	136522	81913	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	427.18	579.96	167533	167533	115000	0	0.00	0	0	0	28	28	0	0	0	0	0	0
20	430.25	606.84	191482	191482	115000	0	0.00	0	0	0	38	38	0	0	0	0	0	0
25	433.17	633.13	211227	211227	115000	0	0.00	0	0	0	47	47	0	0	0	0	0	0
35	438.57	683.35	243016	243016	115000	0	0.00	0	0	0	59	59	0	0	0	0	0	0
50	444.11	736.88	279485	279485	115000	0	0.00	0	0	0	77	77	0	0	0	0	0	0
65	450.07	796.61	308218	308218	115000	0	0.00	0	0	0	105	105	0	0	0	0	0	0
80	456.02	858.42	332148	332148	115000	0	0.00	0	0	0	128	128	0	0	0	0	0	0
100	**461.90**	**921.63**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**139**	**139**	**0**	**0**	**0**	**0**	**0**	**0**
130	460.99	911.73	392399	392399	145000	0	0.00	0	0	0	142	124	0	0	0	0	0	0
150	466.51	972.53	411351	411351	145000	0	0.00	0	0	14	153	135	0	0	0	0	0	0
175	469.98	1011.59	432395	432395	154629	0	0.00	0	0	32	178	142	0	0	0	0	0	0
200	**470.03**	**1012.15**	**451163**	**451163**	**209579**	**0**	**0.00**	**0**	**0**	**25**	**180**	**142**	**21**	**0**	**15**	**0**	**0**	**0**
225	470.31	1015.44	468139	468139	262069	0	0.00	0	0	21	182	143	25	0	18	0	-5	10
250	470.71	1019.97	483665	483665	314605	0	0.00	0	0	18	183	143	26	0	20	9	-5	25
325	471.20	1025.58	523757	523757	466105	0	0.00	0	0	13	187	144	31	0	25	14	-6	100
400	471.75	1031.94	556967	556967	556967	0	0.00	0	0	13	193	145	35	0	29	17	-5	143
500	471.90	1033.55	594159	594159	594159	0	0.00	0	0	13	201	150	40	0	32	21	-2	140

TABLE A-19: NA2-145 MIN (R060_800FM_No Fix_145_FP466_P1_20080916)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.98	465.10	40722	38674	8916	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	416.16	489.65	90369	84059	50000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	424.05	553.36	136522	126249	65753	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	428.10	587.92	167533	154598	84559	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	428.64	592.60	191482	176491	115000	0	0.00	0	0	0	25	25	0	0	0	0	0	0
25	431.48	617.85	211227	194541	115000	0	0.00	0	0	0	33	33	0	0	0	0	0	0
35	436.67	665.48	243016	223601	115000	0	0.00	0	0	0	48	48	0	0	0	0	0	0
50	442.99	725.90	279485	256938	115000	0	0.00	0	0	0	62	62	0	0	0	0	0	0
65	448.29	778.56	308218	283204	115000	0	0.00	0	0	0	81	81	0	0	0	0	0	0
80	451.74	813.77	332148	305080	115000	0	0.00	0	0	0	106	106	0	0	0	0	0	0
100	**458.45**	**884.30**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**131**	**131**	**0**	**0**	**0**	**0**	**0**	**0**
130	465.67	963.13	392399	360850	116941	0	0.00	0	0	0	146	118	0	0	0	0	0	0
150	463.97	944.31	411351	381289	145000	0	0.00	0	0	0	146	127	0	0	0	0	0	0
175	468.50	994.84	432395	407507	145000	0	0.00	0	0	31	156	138	0	0	0	0	0	0
200	**470.21**	**1014.21**	**451163**	**429875**	**175825**	**0**	**0.00**	**0**	**0**	**32**	**159**	**141**	**19**	**1**	**0**	**0**	**5**	**0**
225	470.28	1015.10	468139	448786	194960	0	0.00	0	0	29	160	144	21	0	0	0	0	0
250	470.16	1013.63	483665	465352	228088	0	0.00	0	0	23	161	146	23	0	17	0	-5	0
325	470.84	1021.39	523757	506439	339880	0	0.00	0	0	17	162	147	28	0	22	10	-6	67
400	471.55	1029.61	556967	540033	488931	0	0.00	0	0	13	162	148	33	0	26	14	-1	119
500	471.80	1032.40	594159	577133	568037	0	0.00	0	0	12	180	150	39	0	30	17	-2	229

TABLE A-20: NA3-145 BASE (R060_800DR3.5e_145_FP471.5_P1_20080916)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.39	388.94	20002	20002	16328	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.53	389.84	25004	25004	20411	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	402.78	391.51	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.59	397.11	37002	37002	26005	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.18	394.30	40722	40722	25215	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	413.74	470.92	90369	90369	44261	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	421.65	533.58	136522	136522	71655	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	424.92	560.66	167533	167533	115000	0	0.00	0	0	0	26	26	0	0	0	0	0	0
20	428.02	587.24	191482	191482	115000	0	0.00	0	0	0	36	36	0	0	0	0	0	0
25	431.43	617.37	211227	211227	115000	0	0.00	0	0	0	45	45	0	0	0	0	0	0
35	437.15	669.98	243016	243016	115000	0	0.00	0	0	0	57	57	0	0	0	0	0	0
50	442.97	725.68	279485	279485	115000	0	0.00	0	0	0	75	75	0	0	0	0	0	0
65	449.11	786.85	308218	308218	115000	0	0.00	0	0	0	103	103	0	0	0	0	0	0
80	453.74	834.47	332148	332148	115000	0	0.00	0	0	0	127	127	0	0	0	0	0	0
100	**460.46**	**906.02**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**137**	**137**	**0**	**0**	**0**	**0**	**0**	**0**
130	461.49	917.22	392399	392399	145000	0	0.00	0	0	0	144	125	0	0	0	0	0	0
150	466.26	969.69	411351	411351	145000	0	0.00	0	0	7	152	133	0	0	0	0	0	0
175	469.90	1010.67	432395	432395	145000	0	0.00	0	0	34	160	142	0	0	0	0	0	0
200	**474.92**	**1068.57**	**451163**	**451163**	**145000**	**0**	**0.00**	**0**	**0**	**53**	**171**	**153**	**0**	**0**	**0**	**0**	**0**	**0**
225	477.03	1093.42	468139	468139	171154	0	0.00	0	0	56	173	156	19	1	0	0	4	0
250	477.36	1097.31	483665	483665	197562	0	0.00	0	0	56	174	158	23	0	0	0	7	0
325	477.22	1095.62	523757	523757	300796	0	0.00	0	0	48	196	157	28	0	22	10	-6	78
400	477.90	1103.69	556967	556967	399130	0	0.00	0	0	39	201	158	32	0	26	14	-5	100
500	478.32	1108.60	594159	594159	558062	0	0.00	0	0	28	205	159	38	0	30	18	-2	151

TABLE A-21: NA3-145 MAX (R060_800DR3.5e_145_FP471.5_P1_20080916)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.74	419.09	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.86	419.97	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	406.97	420.78	29000	29000	23588	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.19	422.35	37002	37002	27464	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.29	423.06	40722	40722	30225	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	414.02	473.00	90369	90369	54221	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	423.56	549.32	136522	136522	81913	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	427.18	579.96	167533	167533	115000	0	0.00	0	0	0	28	28	0	0	0	0	0	0
20	430.25	606.84	191482	191482	115000	0	0.00	0	0	0	38	38	0	0	0	0	0	0
25	433.17	633.13	211227	211227	115000	0	0.00	0	0	0	47	47	0	0	0	0	0	0
35	438.57	683.35	243016	243016	115000	0	0.00	0	0	0	59	59	0	0	0	0	0	0
50	444.11	736.88	279485	279485	115000	0	0.00	0	0	0	77	77	0	0	0	0	0	0
65	450.07	796.61	308218	308218	115000	0	0.00	0	0	0	105	105	0	0	0	0	0	0
80	456.02	858.42	332148	332148	115000	0	0.00	0	0	0	128	128	0	0	0	0	0	0
100	**461.90**	**921.63**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**139**	**139**	**0**	**0**	**0**	**0**	**0**	**0**
130	460.99	911.73	392399	392399	145000	0	0.00	0	0	0	142	124	0	0	0	0	0	0
150	466.41	971.34	411351	411351	145000	0	0.00	0	0	9	151	133	0	0	0	0	0	0
175	470.14	1013.42	432395	432395	145000	0	0.00	0	0	34	179	143	0	0	0	0	0	0
200	**#N/A**	**#N/A**	**451163**	**451163**	**#N/A**	**#N/A**	**#N/A**	**0**	**0**	**15**	**74**	**#N/A**	**5**	**#N/A**	**0**	**0**	**#N/A**	**#N/A**
225	477.06	1093.72	468139	468139	172840	0	0.00	0	0	56	195	156	19	0	0	0	4	0
250	477.19	1095.30	483665	483665	202925	0	0.00	0	0	55	197	157	23	0	17	0	1	0
325	477.39	1097.70	523757	523757	320734	0	0.00	0	0	47	199	156	29	0	22	11	100	35
400	478.01	1104.97	556967	556967	430723	0	0.00	0	0	37	205	157	32	0	26	14	-5	100
500	478.08	1105.79	594159	594159	558062	0	0.00	0	0	27	210	159	38	0	30	18	-2	167

TABLE A-22: NA3-145 MIN (R060_800DR3.5e_145_FP471.5_P1_20080916)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.98	465.10	40722	38674	8916	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	416.16	489.65	90369	84059	50000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	424.05	553.36	136522	126249	65753	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	428.10	587.92	167533	154598	84559	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	428.64	592.60	191482	176491	115000	0	0.00	0	0	0	25	25	0	0	0	0	0	0
25	431.48	617.85	211227	194541	115000	0	0.00	0	0	0	33	33	0	0	0	0	0	0
35	436.67	665.48	243016	223601	115000	0	0.00	0	0	0	48	48	0	0	0	0	0	0
50	442.99	725.90	279485	256938	115000	0	0.00	0	0	0	62	62	0	0	0	0	0	0
65	448.29	778.56	308218	283204	115000	0	0.00	0	0	0	81	81	0	0	0	0	0	0
80	451.74	813.77	332148	305080	115000	0	0.00	0	0	0	106	106	0	0	0	0	0	0
100	**458.45**	**884.30**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**131**	**131**	**0**	**0**	**0**	**0**	**0**	**0**
130	465.98	966.59	392399	360850	115000	0	0.00	0	0	0	147	147	0	0	0	0	0	0
150	463.97	944.31	411351	381289	145000	0	0.00	0	0	0	146	127	0	0	0	0	0	0
175	468.21	991.54	432395	407507	145000	0	0.00	0	0	23	155	137	0	0	0	0	0	0
200	**471.93**	**1033.95**	**451163**	**429875**	**145000**	**0**	**0.00**	**0**	**0**	**44**	**164**	**146**	**0**	**0**	**0**	**0**	**0**	**0**
225	474.23	1060.55	468139	448786	145000	0	0.00	0	0	52	169	153	0	0	0	0	0	0
250	476.87	1091.48	483665	465352	145000	0	0.00	0	0	60	176	161	0	0	0	0	0	0
325	481.26	1143.79	523757	506439	182469	0	0.00	0	0	69	184	169	22	0	0	0	10	0
400	485.04	1189.63	556967	540033	406641	80	1.04	0	0	61	182	168	25	0	17	6	167	63
500	486.04	1201.93	594159	577133	670948	220	2.04	0	0	49	197	167	32	0	23	10	-2	440

TABLE A-23: NA1-160 BASE (R000_800CF_No Fix_160_FP470_P1_20081214)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.39	388.94	20002	20002	16328	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.53	389.84	25004	25004	20411	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	402.78	391.51	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.60	397.18	37002	37002	25945	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.08	393.58	40722	40722	25891	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	418.26	506.23	90369	90369	26643	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	431.09	614.37	136522	136522	43519	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	436.58	664.64	167533	167533	71079	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	440.71	703.82	191482	191482	87949	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	444.54	741.14	211227	211227	104311	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	450.77	803.76	243016	243016	115000	0	0.00	0	0	0	28	28	0	0	0	0	0	0
50	459.13	891.67	279485	279485	115000	0	0.00	0	0	0	54	54	0	0	0	0	0	0
65	462.93	932.91	308218	308218	135000	0	0.00	0	0	0	65	65	0	0	0	0	0	0
80	468.15	990.91	332148	332148	135000	0	0.00	0	0	23	78	78	0	0	0	0	0	0
100	**472.32**	**1038.47**	**359078**	**359078**	**145000**	**0**	**0.00**	**0**	**0**	**43**	**96**	**96**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.29	1072.93	392399	392399	186741	0	0.00	0	0	48	104	102	30	10	0	0	22	0
150	475.86	1079.58	411351	411351	236150	0	0.00	0	0	48	105	102	32	8	17	0	36	5
175	476.03	1081.62	432395	432395	268705	0	0.00	0	0	48	105	103	34	9	19	0	82	17
200	**476.37**	**1085.66**	**451163**	**451163**	**321017**	**0**	**0.00**	**0**	**0**	**45**	**104**	**102**	**37**	**9**	**22**	**11**	**53**	**42**
225	476.56	1087.85	468139	468139	361431	0	0.00	0	0	41	104	102	40	9	24	12	64	42
250	476.64	1088.84	483665	483665	408551	0	0.00	0	0	38	103	100	41	9	26	14	71	46
325	477.14	1094.64	523757	523757	482854	0	0.00	0	0	35	102	101	48	12	30	18	86	49
400	477.86	1103.24	556967	556967	503865	0	0.00	0	0	38	107	107	41	0	35	21	36	48
500	479.04	1117.15	594159	594159	513195	0	0.00	0	0	43	129	115	48	0	39	25	32	51

TABLE A-24: NA1-160 MAX (R000_800CF_No Fix_160_FP470_P1_20081214)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.74	419.09	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.86	419.97	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	406.97	420.78	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.19	422.35	37002	37002	26113	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.29	423.06	40722	40722	26125	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	419.62	517.12	90369	90369	27084	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	431.28	616.04	136522	136522	54716	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	438.30	680.77	167533	167533	78433	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	443.02	726.15	191482	191482	97176	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	446.43	759.84	211227	211227	114092	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	452.77	824.45	243016	243016	115000	0	0.00	0	0	0	34	34	0	0	0	0	0	0
50	460.96	911.37	279485	279485	115000	0	0.00	0	0	0	59	59	0	0	0	0	0	0
65	464.23	947.20	308218	308218	135000	0	0.00	0	0	0	69	68	0	0	0	0	0	0
80	468.90	999.41	332148	332148	135000	0	0.00	0	0	27	82	80	0	0	0	0	0	0
100	**472.51**	**1040.60**	**359078**	**359078**	**145000**	**0**	**0.00**	**0**	**0**	**44**	**96**	**96**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.13	1071.01	392399	392399	178470	0	0.00	0	0	48	102	102	30	10	0	0	9	0
150	475.70	1077.77	411351	411351	219943	0	0.00	0	0	48	104	103	32	9	17	0	45	0
175	476.05	1081.83	432395	432395	270444	0	0.00	0	0	48	105	103	34	9	19	0	86	15
200	**476.38**	**1085.69**	**451163**	**451163**	**321521**	**0**	**0.00**	**0**	**0**	**45**	**104**	**102**	**37**	**9**	**22**	**11**	**51**	**42**
225	476.57	1087.95	468139	468139	363644	0	0.00	0	0	41	104	102	40	9	24	12	69	42
250	476.65	1088.93	483665	483665	410822	0	0.00	0	0	37	102	99	41	9	26	14	77	46
325	477.16	1094.95	523757	523757	484299	0	0.00	0	0	35	103	101	48	11	31	18	48	48
400	477.82	1102.73	556967	556967	503510	0	0.00	0	0	38	107	107	51	10	35	21	49	46
500	479.01	1116.84	594159	594159	512982	0	0.00	0	0	43	127	115	57	9	39	25	36	52

TABLE A-25: NA1-160 MIN (R000_800CF_No Fix_160_FP470_P1_20081214)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.98	465.10	40722	38674	8916	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	418.60	508.91	90369	84059	27017	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	430.16	606.08	136522	126249	50159	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	435.32	652.92	167533	154598	66881	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	438.84	685.88	191482	176491	81222	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	441.67	713.08	211227	194541	90784	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	446.19	757.45	243016	223601	112689	0	0.00	0	0	0	0	0	0	0	0	0	0	0
50	452.28	819.34	279485	256938	115000	0	0.00	0	0	0	33	33	0	0	0	0	0	0
65	457.86	877.97	308218	283204	115000	0	0.00	0	0	0	50	50	0	0	0	0	0	0
80	459.50	895.65	332148	305080	135000	0	0.00	0	0	0	56	56	0	0	0	0	0	0
100	**464.68**	**952.16**	**359078**	**329825**	**135000**	**0**	**0.00**	**0**	**0**	**0**	**70**	**70**	**0**	**0**	**0**	**0**	**0**	**0**
130	470.22	1014.37	392399	360850	145000	0	0.00	0	0	34	87	87	0	0	0	0	0	0
150	471.89	1033.48	411351	381289	160000	0	0.00	0	0	45	99	98	22	24	0	0	0	0
175	475.26	1072.52	432395	407507	173176	0	0.00	0	0	50	111	111	31	10	0	0	8	0
200	**475.59**	**1076.50**	**451163**	**429875**	**215563**	**0**	**0.00**	**0**	**0**	**50**	**110**	**110**	**33**	**9**	**17**	**0**	**27**	**0**
225	475.83	1079.24	468139	448786	257292	0	0.00	0	0	48	107	106	34	8	20	0	65	28
250	476.10	1082.41	483665	465352	293291	0	0.00	0	0	46	107	106	36	8	22	0	65	49
325	476.60	1088.34	523757	506439	392913	0	0.00	0	0	36	106	105	40	8	26	14	90	61
400	476.93	1092.23	556967	540033	467440	0	0.00	0	0	32	105	104	49	12	30	17	74	65
500	477.96	1104.34	594159	577133	504670	0	0.00	0	0	36	124	111	55	12	34	20	87	52

TABLE A-26: NA2-160 BASE (R060_800FM_No Fix_160_FP466_P1_20090106)																			
1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.39	388.94	20002	20002	16328	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.53	389.84	25004	25004	20411	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	402.78	391.51	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.59	397.11	37002	37002	26005	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.18	394.30	40722	40722	25215	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	413.74	470.92	90369	90369	44261	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	421.65	533.58	136522	136522	71655	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	425.56	566.08	167533	167533	115000	0	0.00	0	0	0	24	24	0	0	0	0	0	0
20	428.70	593.15	191482	191482	115000	0	0.00	0	0	0	33	33	0	0	0	0	0	0
25	432.02	622.68	211227	211227	115000	0	0.00	0	0	0	42	42	0	0	0	0	0	0
35	437.51	673.33	243016	243016	115000	0	0.00	0	0	0	56	56	0	0	0	0	0	0
50	444.04	736.19	279485	279485	115000	0	0.00	0	0	0	75	75	0	0	0	0	0	0
65	449.69	792.72	308218	308218	115000	0	0.00	0	0	0	101	101	0	0	0	0	0	0
80	454.35	840.92	332148	332148	115000	0	0.00	0	0	0	125	125	0	0	0	0	0	0
100	**461.31**	**915.15**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**134**	**134**	**0**	**0**	**0**	**0**	**0**	**0**
130	459.65	897.26	392399	392399	160000	0	0.00	0	0	0	137	121	30	31	0	0	0	0
150	464.33	948.31	411351	411351	160000	0	0.00	0	0	0	146	131	32	33	0	0	0	0
175	467.74	986.26	432395	432395	160000	0	0.00	0	0	26	156	141	30	31	0	0	0	0
200	**470.09**	**1012.88**	**451163**	**451163**	**196633**	**0**	**0.00**	**0**	**0**	**26**	**157**	**143**	**37**	**17**	**0**	**0**	**37**	**0**
225	470.16	1013.72	468139	468139	248894	0	0.00	0	0	21	156	143	40	16	17	0	85	3
250	470.44	1016.88	483665	483665	296022	0	0.00	0	0	18	157	144	41	16	19	0	128	7
325	471.17	1025.16	523757	523757	458379	0	0.00	0	0	13	176	144	44	14	24	13	127	110
400	471.32	1026.88	556967	556967	523129	0	0.00	0	0	13	183	146	47	13	28	16	120	127
500	471.57	1029.74	594159	594159	594159	0	0.00	0	0	12	191	149	53	14	31	20	175	146
PMF	477.46	1098.49	905770	905770	811646	0	0.00	0	0	58	246	124	109	18	56	44	100	139

TABLE A-27: NA2-160 MAX (R060_800FM_No Fix_160_FP466_P1_20090106)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.74	419.09	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.86	419.97	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	406.97	420.78	29000	29000	23588	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.19	422.35	37002	37002	27464	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.29	423.06	40722	40722	30225	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	415.59	485.14	90369	90369	54221	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	423.58	549.49	136522	136522	81913	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	427.91	586.22	167533	167533	115000	0	0.00	0	0	0	26	26	0	0	0	0	0	0
20	430.94	612.96	191482	191482	115000	0	0.00	0	0	0	36	36	0	0	0	0	0	0
25	434.32	643.67	211227	211227	115000	0	0.00	0	0	0	45	45	0	0	0	0	0	0
35	439.26	689.88	243016	243016	115000	0	0.00	0	0	0	59	59	0	0	0	0	0	0
50	446.18	757.39	279485	279485	115000	0	0.00	0	0	0	78	78	0	0	0	0	0	0
65	451.66	812.98	308218	308218	115000	0	0.00	0	0	0	104	104	0	0	0	0	0	0
80	457.54	874.57	332148	332148	115000	0	0.00	0	0	0	126	126	0	0	0	0	0	0
100	**462.60**	**929.22**	**359078**	**359078**	**115155**	**0**	**0.00**	**0**	**0**	**0**	**136**	**106**	**0**	**0**	**0**	**0**	**0**	**0**
130	461.22	914.25	392399	392399	160000	0	0.00	0	0	0	136	123	30	31	0	0	0	0
150	464.46	949.80	411351	411351	160000	0	0.00	0	0	0	145	131	32	33	0	0	0	0
175	467.90	988.09	432395	432395	160000	0	0.00	0	0	27	154	141	30	31	0	0	0	0
200	**470.10**	**1012.99**	**451163**	**451163**	**195966**	**0**	**0.00**	**0**	**0**	**26**	**176**	**143**	**37**	**17**	**0**	**0**	**33**	**0**
225	470.20	1014.16	468139	468139	254566	0	0.00	0	0	20	177	143	40	16	17	0	92	2
250	470.35	1015.82	483665	483665	288029	0	0.00	0	0	18	179	144	41	16	19	0	118	7
325	471.14	1024.90	523757	523757	452926	0	0.00	0	0	13	185	145	44	14	24	14	175	94
400	471.33	1027.02	556967	556967	540048	0	0.00	0	0	12	189	146	47	13	28	17	175	120
500	471.55	1029.62	594159	594159	594159	0	0.00	0	0	13	197	149	53	13	32	21	175	107

TABLE A-28: NA2-160 MIN (R060_800FM_No Fix_160_FP466_P1_20090106)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.98	465.10	40722	38674	8916	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	416.16	489.65	90369	84059	50000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	424.05	553.36	136522	126249	65753	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	428.30	589.62	167533	154598	84559	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	429.46	599.84	191482	176491	115000	0	0.00	0	0	0	23	23	0	0	0	0	0	0
25	432.18	624.13	211227	194541	115000	0	0.00	0	0	0	31	31	0	0	0	0	0	0
35	437.35	671.86	243016	223601	115000	0	0.00	0	0	0	45	45	0	0	0	0	0	0
50	443.50	730.90	279485	256938	115000	0	0.00	0	0	0	61	61	0	0	0	0	0	0
65	448.76	783.26	308218	283204	115000	0	0.00	0	0	0	80	80	0	0	0	0	0	0
80	452.90	825.76	332148	305080	115000	0	0.00	0	0	0	116	116	0	0	0	0	0	0
100	**459.22**	**892.62**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**129**	**129**	**0**	**0**	**0**	**0**	**0**	**0**
130	466.33	970.48	392399	360850	121233	0	0.00	0	0	9	143	119	0	0	0	0	0	0
150	463.02	933.87	411351	381289	160000	0	0.00	0	0	0	140	125	27	28	0	0	0	0
175	467.23	980.56	432395	407507	160000	0	0.00	0	0	23	150	137	25	26	0	0	0	0
200	**470.09**	**1012.92**	**451163**	**429875**	**176230**	**0**	**0.00**	**0**	**0**	**30**	**153**	**140**	**33**	**14**	**0**	**0**	**6**	**0**
225	470.13	1013.35	468139	448786	198409	0	0.00	0	0	26	154	143	36	15	0	0	35	0
250	470.11	1013.14	483665	465352	227979	0	0.00	0	0	23	156	146	38	15	17	0	68	0
325	470.67	1019.48	523757	506439	327556	0	0.00	0	0	17	158	147	42	14	22	10	125	38
400	471.18	1025.30	556967	540033	463776	0	0.00	0	0	13	159	149	47	14	26	14	125	114
500	471.42	1028.03	594159	577133	544670	0	0.00	0	0	12	175	151	52	14	29	17	175	177

TABLE A-29: NA3-160 BASE (R060_800DR3.5e_160_FP471.5_P1_20081215)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	399.42	369.20	5000	5000	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	402.39	388.94	20002	20002	16328	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.53	389.84	25004	25004	20411	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	402.78	391.51	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	403.59	397.11	37002	37002	26005	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	403.18	394.30	40722	40722	25215	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	413.74	470.92	90369	90369	44261	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	421.65	533.58	136522	136522	71655	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	425.56	566.08	167533	167533	115000	0	0.00	0	0	0	24	24	0	0	0	0	0	0
20	428.70	593.15	191482	191482	115000	0	0.00	0	0	0	33	33	0	0	0	0	0	0
25	432.02	622.68	211227	211227	115000	0	0.00	0	0	0	42	42	0	0	0	0	0	0
35	437.51	673.33	243016	243016	115000	0	0.00	0	0	0	56	56	0	0	0	0	0	0
50	444.04	736.19	279485	279485	115000	0	0.00	0	0	0	75	75	0	0	0	0	0	0
65	449.69	792.72	308218	308218	115000	0	0.00	0	0	0	101	101	0	0	0	0	0	0
80	454.35	840.92	332148	332148	115000	0	0.00	0	0	0	125	125	0	0	0	0	0	0
100	**461.31**	**915.15**	**359078**	**359078**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**134**	**134**	**0**	**0**	**0**	**0**	**0**	**0**
130	459.65	897.26	392399	392399	160000	0	0.00	0	0	0	137	121	30	31	0	0	0	0
150	464.33	948.31	411351	411351	160000	0	0.00	0	0	0	146	131	32	33	0	0	0	0
175	467.47	983.26	432395	432395	160000	0	0.00	0	0	17	153	138	35	36	0	0	0	0
200	**472.47**	**1040.14**	**451163**	**451163**	**160000**	**0**	**0.00**	**0**	**0**	**46**	**164**	**150**	**32**	**33**	**0**	**0**	**0**	**0**
225	476.20	1083.60	468139	468139	160000	0	0.00	0	0	57	170	157	34	35	0	0	0	0
250	477.15	1094.82	483665	483665	193667	0	0.00	0	0	54	170	157	41	19	0	0	18	0
325	477.08	1093.97	523757	523757	294943	0	0.00	0	0	48	189	157	44	16	22	0	78	4
400	477.78	1102.31	556967	556967	405477	0	0.00	0	0	38	195	158	47	15	26	14	100	81
500	478.03	1105.25	594159	594159	534386	0	0.00	0	0	28	202	160	53	16	29	17	107	150

TABLE A-30: NA3-160 MAX (R060_800DR3.5e_160_FP471.5_P1_20081215)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	406.42	416.85	5000	5000	4242	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	406.74	419.09	20002	20002	16967	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	406.86	419.97	25004	25004	21210	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	406.97	420.78	29000	29000	24600	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	407.19	422.35	37002	37002	26113	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	407.29	423.06	40722	40722	26125	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	419.62	517.12	90369	90369	27084	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	431.28	616.04	136522	136522	54716	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	438.30	680.77	167533	167533	78433	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	443.02	726.15	191482	191482	97176	0	0.00	0	0	0	0	0	0	0	0	0	0	0
25	446.43	759.84	211227	211227	114092	0	0.00	0	0	0	0	0	0	0	0	0	0	0
35	452.77	824.45	243016	243016	115000	0	0.00	0	0	0	34	34	0	0	0	0	0	0
50	460.96	911.37	279485	279485	115000	0	0.00	0	0	0	59	59	0	0	0	0	0	0
65	464.23	947.20	308218	308218	135000	0	0.00	0	0	0	69	68	0	0	0	0	0	0
80	468.90	999.41	332148	332148	135000	0	0.00	0	0	27	82	80	0	0	0	0	0	0
100	**472.51**	**1040.59**	**359078**	**359078**	**145000**	**0**	**0.00**	**0**	**0**	**44**	**96**	**96**	**0**	**0**	**0**	**0**	**0**	**0**
130	475.46	1074.96	392399	392399	187473	0	0.00	0	0	49	103	103	20	0	0	0	5	0
150	475.86	1079.56	411351	411351	229570	0	0.00	0	0	49	104	103	23	0	17	0	-6	6
175	476.20	1083.56	432395	432395	279376	0	0.00	0	0	47	104	102	26	0	20	0	47	8
200	**476.47**	**1086.84**	**451163**	**451163**	**329343**	**0**	**0.00**	**0**	**0**	**44**	**104**	**102**	**28**	**0**	**22**	**11**	**-6**	**41**
225	476.60	1088.35	468139	468139	376509	0	0.00	0	0	40	103	101	31	0	24	12	-5	42
250	476.70	1089.55	483665	483665	421762	0	0.00	0	0	37	102	99	33	0	26	14	56	48
325	477.24	1095.85	523757	523757	488365	0	0.00	0	0	35	103	101	37	0	31	19	-6	42
400	477.87	1103.32	556967	556967	503936	0	0.00	0	0	38	107	107	42	0	35	21	45	48
500	479.04	1117.22	594159	594159	513243	0	0.00	0	0	43	127	115	48	0	39	25	30	51

TABLE A-31: NA3-160 MIN (R060_800DR3.5e_160_FP471.5_P1_20081215)

1 in X chance per year	Peak Elev	Storage	Peak Unreg Inflow	Peak Regulated Inflow	Peak Discharge	Crest Overflow	Amount above top of dam	Duration Pool >= PE 480.5 ft	Duration Pool >= PE 471 ft	Duration Pool >= PE 466 ft	Event Total Duration Q >= 115 tcfs	Main Wave Duration Q >= 115 tcfs	Event Total Duration Q >= 160 tcfs	Main Wave Duration Q = 160 tcfs	Event Total Duration Q >= 200 tcfs	Event Total Duration Q >= 300 tcfs	Max ROI 160-220k cfs	Max ROI >220k cfs
	Ft	TAF	Ft	cfs	cfs	cfs	ft	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	Hrs	cfs	cfs
1.01569	386.34	290.07	5000	6419	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.2977	398.86	365.50	20002	20133	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.4393	402.75	391.34	25004	24305	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.5655	405.77	412.26	29000	27959	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
1.8517	411.50	453.90	37002	35274	2000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
2	412.98	465.10	40722	38674	8916	0	0.00	0	0	0	0	0	0	0	0	0	0	0
5	416.16	489.65	90369	84059	50000	0	0.00	0	0	0	0	0	0	0	0	0	0	0
10	424.05	553.36	136522	126249	65753	0	0.00	0	0	0	0	0	0	0	0	0	0	0
15	428.30	589.62	167533	154598	84559	0	0.00	0	0	0	0	0	0	0	0	0	0	0
20	429.46	599.84	191482	176491	115000	0	0.00	0	0	0	23	23	0	0	0	0	0	0
25	432.18	624.13	211227	194541	115000	0	0.00	0	0	0	31	31	0	0	0	0	0	0
35	437.35	671.86	243016	223601	115000	0	0.00	0	0	0	45	45	0	0	0	0	0	0
50	443.50	730.90	279485	256938	115000	0	0.00	0	0	0	61	61	0	0	0	0	0	0
65	448.76	783.26	308218	283204	115000	0	0.00	0	0	0	80	80	0	0	0	0	0	0
80	452.90	825.76	332148	305080	115000	0	0.00	0	0	0	116	116	0	0	0	0	0	0
100	**459.22**	**892.62**	**359078**	**329825**	**115000**	**0**	**0.00**	**0**	**0**	**0**	**129**	**129**	**0**	**0**	**0**	**0**	**0**	**0**
130	467.26	980.92	392399	360850	115000	0	0.00	0	0	19	144	144	0	0	0	0	0	0
150	463.02	933.87	411351	381289	160000	0	0.00	0	0	0	140	125	27	28	0	0	0	0
175	467.04	978.39	432395	407507	160000	0	0.00	0	0	14	147	134	31	32	0	0	0	0
200	**470.56**	**1018.21**	**451163**	**429875**	**160000**	**0**	**0.00**	**0**	**0**	**36**	**155**	**142**	**33**	**34**	**0**	**0**	**0**	**0**
225	472.71	1042.91	468139	448786	160000	0	0.00	0	0	49	162	151	31	32	0	0	0	0
250	474.90	1068.38	483665	465352	160000	0	0.00	0	0	55	168	158	33	35	0	0	0	0
325	477.32	1096.76	523757	506439	214967	0	0.00	0	0	57	173	162	42	17	18	0	40	0
400	477.15	1094.84	556967	540033	310772	0	0.00	0	0	47	172	162	47	16	24	11	89	15
500	477.79	1102.40	594159	577133	420080	0	0.00	0	0	34	187	163	52	16	27	14	125	96

Sub-Appendix A5. Hydraulics

Sacramento River Bank Protection Project
Final Hydraulic Appendix

Todd Rivas

August 01, 2011

Revised November 19, 2018

Table of Contents

1. Introduction and Authorization

The Sacramento River Bank Protection Project (SRBPP) is an erosion control project for the protection of the existing levees (including bank protection) and flood control facilities of the Sacramento River Flood Control Project (SRFCP). This project was originally authorized in 1960 and has included subsequent authorizations and phases. The original Phase II authorization was in 1974. The original linear feet of bank protection authorized for Phase II is nearing completion. Congress has authorized an additional 80,000 linear feet of erosion control work for Phase II per the Water Resources Development Act (WRDA) 2007. To construct this additional 80,000 linear feet authorized by congress, the Sacramento District of the United States Army Corps of Engineers (USACE) is developing a programmatic Post Authorization Change Report (PACR) and EIS/R (Environmental Impact Statement/Report) addressing the additionally authorized 80,000 linear feet of erosion control work. The PACR is necessary prior to constructing the 80,000 linear feet. According to USACE guidance, a PACR requires a hydraulic appendix. This hydraulic appendix is written to meet this requirement and support team efforts in preparing the PACR for constructing the 80,000 linear feet authorized by congress. The proposed new approach applies adaptive management to address cumulative hydraulic impacts as the project is implemented.

The project authorization is to reduce the risk of levee failures within the SRFCP system from erosion. The SRFCP is a dynamic system and it is not possible to predict which sites will be repaired in the future. Therefore this hydraulic appendix is programmatic and not site-specific. Site specific analysis will be conducted as part of developing site-specific Engineering Document Reports (EDR's) and Design Document Reports (DDR's) during site specific design.

2. Background Information

In the late 1800s the flood capacity of the Sacramento River and its tributaries was greatly reduced due to tailings from hydraulic mining. Hydraulic mining was officially halted in 1884 with two court cases (Woodruff v. North Bloomfield Gravel Mining Co. and People v. Gold Run Ditch and Mining Company). Levees were improperly built and rivers in the Sacramento Basin were unable to contain average year floods. It was proposed in 1880 that the state engineer take control of maintaining the drainage of the river basins, however this was never acted on by Congress. In 1894, it was suggested again that improvements to the channel of the lower Sacramento River would lower flood stages, however the construction of engineered levees on the Feather River was very important. Again, the legislature did not act on these recommendations. In 1904, another futile attempt was proposed to modify the channels of the sediment filled streams to increase slope and encourage movement of sediment from the river channel. It also proposed levees on the Yuba and Feather Rivers; however, the state did not take action. In 1905 the Rivers and Harbors Act of 1905 appointed three engineers from the Army to cooperate with the state and determine the feasibility of navigational improvements (Kochis 1963).

The California Debris Commission (CDC) was created in 1893 as part of the Rivers and Harbors Act. It was created by the Federal government and was made up of three army engineers that were appointed by the president. Minor work on debris control and Navigation were performed by the federal government prior to the creation of the CDC. In February 1900, Daguerre Point Dam was proposed on the Lower Yuba as a means to contain mining debris. The first flood control measures were first carried out in the Rivers and Harbors Act of 1910. The report is contained in House Document 81 and is from a report by the CDC. The flood control measures proposed included dredging of the Sacramento River below Cache Slough to increase channel capacity. Dredging was not performed on the Feather River even though it was included in the report (Kochis 1963).

Shortly after the 1910 flood control project approved in House Document 81 the state of California created the Reclamation Board Act of 1911. This was made up of three members appointed by the governor. The board was to examine plans for flood control and reclamation of lands in accordance with the CDC. If the Reclamation Board did not approve the plans then they could not be pursued. In 1913, the Reclamation Board's duties were more clearly defined to not include channel expansion or construction of weirs on the Sacramento River. The number of board members was also increased to seven (Kochis 1963).

In House Document 81, it suggests that the capacity of the Sacramento River at Collinsville needed to be in excess of 600000 cfs, where prior to the floods of 1907 and 1909, the capacity was recommended to be 250000 cfs. In the document, the reasoning for not simply widening the channel is articulated to be due to the need for scour flows to wash continued sediment downstream from the hydraulic mining tailings. Also, a wider channel would lower the depth of low flow events causing navigation to be an issue. As a result, the Basins surrounding the Sacramento River were investigated for reclamation. The two largest were the Sutter Basin and the Yolo Basin with 1,038,000 AF and 1,126,000 AF, respectively. Evaluation of the capacity needed in the river at various points showed that it needed a much greater capacity than was there at the time (Stimson 1911).

Table 1: Channel Capacity at locations along the Sacramento River

Localities	Distance	Capacity, cfs (1911)	Capacity, cfs (required)
Chico Landing	202	235000	235000
Colusa	151	70000	250000
Knights Landing	94	25000	250000
Below Feather River	81	65000	450000
Below American River	62	80000	525000
Below Cache Slough	16	165000	600000

The bypass system was first proposed in 1894 by Marsden, Manson, and Grunsky who were consultants to the commissioner of public works. This bypass system using the reclaimed basins along with channel improvements to various reaches along the Sacramento River, Feather River, Yuba River and smaller tributaries became the foundation for the Sacramento River Flood Control Project (Stimson 1911). The bypasses are designed to move flood waters around protected areas, such as communities, to reduce flood risk. The Sutter and Yolo bypasses are shown in Figure 1.

Overtime the bypass system developed into the SRFCP with recommended freeboard requirements described in what is known as the 1957 Profiles (see Appendix B for full description and background of the 1957 profiles). These profiles describe the minimum freeboard required for each segment of the SRFCP for the project discharge for that reach. The SRBPP provides erosion repairs to the SRFCP but does not change the SRFCP discharges or the required minimum freeboard requirements for these discharges. Please see Appendix B for more information on the development of the 1957 profiles.

2.1 Hydrologic and Hydraulic Criteria

There are three main scenarios that need to be considered for the hydrologic and hydraulic criteria. These are:

1) The SRFCP discharge and freeboard requirements
2) Hydrology and hydraulic criteria for economic analysis
3) Hydrology and hydraulic criteria for engineering analysis and design

2.2 SRFCP Discharge and Freeboard Requirements

The SRBPP provides erosion repairs to the SRFCP. The SRFCP project discharges and minimum required freeboard for each reach of the SRFCP are described in the 1957 profiles. The 1957 profiles describe the SRFCP discharge and associated minimum freeboard for each reach of the SRFCP (see Appendix B for more information on the 1957 profiles).The SRBPP does not change the SRFCP project discharges or minimum required freeboard.

2.3 Hydrology and Hydraulic Criteria for Economic Analysis

The hydrology used for Economic Analysis is developed from the Comprehensive Study (Comp Study), which included the SRFCP. See the hydrology appendix for a description of the Comp Study hydrology.

The Sacramento Bank Protection Project is dynamic and it is not possible to determine exact location of repairs, repair alternative, or timing of repair construction. However, the type of project authorization (Flood Damage Risk Reduction) requires that an economic analysis be conducted that includes a benefit to cost ratio. Therefore, a coarse scale economic analysis was conducted using a representative selection of 101 sites with representative repair alternatives assumed to be implemented over the life of the Phase II 80,000 LF portion of the project. It is anticipated that the economic analysis will be improved in future economic updates depending on availability of funding and resources. It should be noted that the project is only authorized to protect banks from erosion only and not other mechanisms such as seepage. One challenge in developing the economic analysis is that there is not enough reliable scientific information available to determine inundation areas from erosion only caused levee failure. The team decided to use a coarse scale economic analysis using the representative sites to develop a project wide benefit to cost ratio. The team decided to use the inundation areas developed as part of the 2002 Comprehensive Study (Comp Study). This approach has a number of advantages described below:

I. The inundation areas have been developed and are readily available
II. The inundation area development uses a consistent approach applicable for large scale rough analysis like this large scale economic analysis.
III. The hydrology for the inundation area development has been mostly certified for the Comp Study

IV. The hydraulic models used for estimating the inundation areas are calibrated to known events.

However, using the Comprehensive Study data also has some disadvantages. These include:

I. Inundation areas may not be based on the most recent and/or accurate hydraulic model available.
II. The inundation areas do not assume failure of levees by erosion only. This introduces inconsistency to the economic analysis and likely overestimates damages.

Inundation areas from the USACE included multiple storm centerings (see hydrology appendix) and required that the data be processed so it can be used readily for economic analysis. A contractor for DWR combined storm centerings and processed the Comp Study floodplains for economic analysis for another project. See the economic appendix for further information on the inundation areas. The team decided that the DWR contractor data modified for economic analysis was the most suitable inundation areas for use in a rough large scale economic analysis for the project and provides acceptable results for this coarse level of analysis. This is because the inundation areas are:

I. Based on relatively recent and consistent hydraulic modeling
II. Calibrated to known events
III. Based largely on certified hydrology
IV. Based on multiple storm centerings where deemed reasonable, providing a better overall picture of the damages.
    a. The DWR contractor modified the inundation areas by using only the greatest depth for any location from the multiple storm centering.
V. Based on a dataset that is in a format readily available for economic analysis.

The results are only appropriate for a rough scale project-wide economic analysis for estimating the project's overall benefit to cost ratio. It is not recommended to use the results to screen damage areas from future project actions or individual sites from future construction. Economic updates are required periodically for the project. Future economic updates are expected to include improvements, which may include using new hydrology and hydraulic models and the resulting new floodplains.

2.4. Hydrology and Hydraulic Criteria for Analysis and Design

2.4.1. Hydrology and Discharge for Site Specific Check of Repair Hydraulic Impacts

The SRBPP does not need to use the exact 1957 SRFCP discharges because the SRFCP is designed and constructed to convey the SRFCP discharges with the minimum freeboard described in the 1957 profiles. Assuming the SRFCP is maintained properly, the SRBPP only needs to know if the relative hydraulic changes from constructing the repairs. Therefore, the SRBPP will check for changes in the estimated water surface elevation for before and after each repair is constructed. In addition, the project will check on a site specific basis changes to other hydraulic characteristics that need to be considered.

These may include changes in flow patterns, velocity distribution, sedimentation, and other hydraulic characteristics. Some of these considerations are discussed in the reach scale hydraulic analysis and design considerations below.

The 1957 profile stages for many reaches within the SRFCP are close to a 1/100 Annual Chance Exceedance (ACE) event. The hydraulic differences before and after the repairs will be apparent for a range of discharges near the 1957 SRFCP discharge for the reach. Repairs will be modeled for the 1/100 ACE event or similar discharge to ensure the repairs will not increase the water surface elevation beyond the noise of the hydraulic model. The 1/100 ACE event or similar discharge is selected for this hydraulic check for convenience and to use the latest existing hydrology. Using the 1/100 ACE or similar event provides results consistent with the 1957 SRFCP discharges with the added convenience of readily available hydrology and ability to use the latest hydraulic models. In addition, a range of smaller discharges will also be evaluated to ensure that changes at lower flows do not occur, such as for ½ ACE events. Repairs will be designed so that the before and after hydraulic conditions are similar without negative impacts such as changes to flow splits or excessive induced erosion or sedimentation off-site. Hydrology will be based on the latest readily available hydrology applicable to assess the existing and future conditions.

2.4.2. Hydrology and Discharge for Site Specific Design of Hydraulic Features

Design of hydraulic features, such as sizing rock for stone protection, will be conducted according to USACE Criteria with consideration of other design requirements. For example, the sponsor may have requirements, such as protection for a 1/200 ACE event, exceeding USACE standards. Or the sponsor required discharge for design may exceed the 1957 SRFCP discharges. In these cases the rock will be sized for the more stringent requirement. It is possible that the discharge used for the hydraulic design of features will be different than the discharge used for the hydraulic check. However, more likely these will be the same discharge. Hydrology will be based on the latest readily available hydrology applicable to assess the existing and future conditions.

During site specific design of SRBPP, it may be advantageous to use to conduct hydraulic analysis for other than design discharges. This may include more frequent events that may be important for environmental analysis and design or less frequent events considered for other purposes. In addition, this may be necessary to check for hydraulic changes at other than design flows. This may be needed because even if there are no significant changes for the design flow, there could be unintended impacts for lower flows as the flow patterns can change significantly as the stage changes. The need to analyze other flows will be determined on a case by case basis. The analysis of other than design flows will also use the best available data and tools to conduct an appropriate level of hydraulic analysis.

2.4.3. Levee Height Considerations

The SRBPP does not modify the height of levees but seeks to reduce flood damage risk from erosion for existing levees in the SRFCP. Furthermore, the SRBPP assumes any reduction in levee crown elevation will be regularly repaired as part of maintenance. This assumption is consistent with USACE policy and project documents. According to 33 CFR 208.10, cited in every project Operation, Maintenance, Repair, Rehabilitation, and Replacement (OMRRR) Manual:

> “(b) Levees (1) Maintenance: Periodic inspections shall be made by the Superintendent to insure that the above maintenance measures are being effectively carried out and, further, to be certain that: (i) No unusual settlement, sloughing, or material loss of grade or levee cross section has taken place.”

Also, the current SRBPP OMRRR manual states that “immediate steps will be taken to correct dangerous conditions disclosed by such inspections” (USACE). For that reason, assuming the levee height is maintained to its original design elevation is a valid assumption for hydraulic analysis and design of SRBPP repairs. If this assumption is not correct, the levee height needs to be addressed in accordance with the OMRRR manual and not the SRBPP project.

3. Cumulative Hydraulic Impacts of the Project

Construction of a repair site is likely to alter the hydraulics at least locally to some degree. Whether the effect on the hydraulics is significant or not depends on the site specific conditions and the characteristics of the repair. Even if the changes to the hydraulics are local and not significant from construction of a particular repair site, a series of repairs in the same general vicinity over time could together alter the hydraulics significantly leading to a cumulative hydraulic impact. For this report a cumulative hydraulic impact will be defined as a significant hydraulic effect resulting from implementing a single project action or a collection of project actions measured from a common baseline hydraulic condition. These actions can include actions from multiple projects and entities that are spatially and temporally distinct. However, for this project the focus will be on the cumulative hydraulic impacts from implementing this project. It is important to consider the cumulative hydraulic impacts of the Phase II additional 80,000 LF to ensure the SRFCP continues to operate according to design without increasing flood damage risk.

3.1. Current Cumulative Hydraulic Impacts Approach.

Recently potential cumulative hydraulic impacts on water surface elevation for the design discharge have been addressed by ensuring the water surface elevation does not increase within the project site by more than 0.1 feet. The value of 0.1 feet was selected based on engineering judgment and is a conservative estimate of the limit of hydraulic model accuracy. Anything below 0.1 feet can be reasonably interpreted as model "noise." By limiting the changes to within the project site and to a relatively small value, the cumulative hydraulic impacts on water surface elevation from multiple repair sites in the same vicinity over a period of time can be reduced.

The advantage of this approach is that it can be implemented relatively easily and quickly at a low cost. A disadvantage is that the approach does not robustly model multiple actions to accurately reflect the actual cumulative hydraulic impacts from previous actions. Another disadvantage is that this approach does not measure affects from a baseline condition, making it very difficult to ensure that there really is no significant cumulative hydraulic impacts.

3.2. Proposed Cumulative Hydraulic Impacts Approach

The proposed new approach applies adaptive management to address cumulative hydraulic impacts as the project is implemented. To measure cumulative hydraulic impacts, a baseline condition needs to be established from which hydraulic changes can be measured. To establish a baseline hydraulic condition it is proposed that a current hydraulic model of the SRFCP system be developed based on the best available information. This model will then become the baseline hydraulic condition from which to measure cumulative hydraulic impacts of the project.

Changes to the system from project implementation will be added to the baseline model incrementally as construction is completed. New project actions will incorporate the proposed action into this updated hydraulic model and analyze the results to estimate if there are any significant negative cumulative hydraulic impacts from implementing the proposed action. The model with the proposed actions included will serve to estimate the cumulative hydraulic impacts up to that point of project implementation. Any proposed actions that are estimated to trigger negative cumulative hydraulic impacts will be either modified to avoid negative cumulative hydraulic impacts or the negative cumulative hydraulic impacts will be mitigated. An updated cumulative hydraulic impacts analysis will be conducted and reported in each site-specific EDR that is developed during project implementation.

The actual data used to develop the model will be determined in the future. However, a good initial candidate for the source of some of this data is from either the Comp Study or from new efforts by the State of California to collect recent topography data (including bathymetry) and develop new hydrology. It may be necessary to collect new topography data and/or develop new hydrology for some reaches.

It is proposed that the 1-dimensional (1D) HEC-RAS hydraulic model environment be used for estimating cumulative hydraulic impacts. There are 2-dimensional (2D) hydraulic models available. However, they can be computationally intensive and may not provide a lot of additional benefit compared to the effort for a system-wide cumulative hydraulic impacts analysis. It should be noted that the 2D hydraulic models may be used for site-specific design and analysis for smaller scale hydraulic impacts. The cumulative hydraulic impacts analysis is more concerned with larger scale changes to the system, not reach and local scale effects.

The advantage of the proposed approach is that it estimates the cumulative hydraulic impacts from a baseline condition in a manner that incorporates past project actions. In addition, it uses the best available information and hydraulic modeling to accurately measure the cumulative hydraulic impacts of the project on the SRFCP as it is implemented. This allows the project to adjust as needed to eliminate or reduce negative cumulative hydraulic impacts. The disadvantage is that this approach does not necessarily include effects from actions outside of the SRBPP project. Another disadvantage is that a 1D model may not accurately portray 2D and 3D processes. A third disadvantage is that it is more resource intensive than the current approach to cumulative hydraulic impacts. It should be noted that this approach can only be implemented if resources are available.

3.3. Significant Cumulative Hydraulic Impact Threshold

As noted earlier, construction of repair sites is likely to alter the hydraulics to some degree at some scale. A threshold could be reached where the magnitude of the cumulative changes from project implementation endangers the original design of the SRFCP. In general terms, this threshold is when the cumulative hydraulic impacts from project implementation significantly increases the flood damage risk at some point within the system. In all cases, the term “significant” is subjective. Based on engineering judgment and consistent with how cumulative hydraulic impacts are currently addressed, it is proposed that a significant cumulative hydraulic impact be defined as a greater than 0.1 foot change in the water surface elevation for the design discharge at any given point in the system. However, if the project can demonstrate that a greater than 0.1 foot change in the water surface elevation for the design discharge at any point does not alter the flood damage risk at any point of the SRFCP, than that particular cumulative hydraulic impact will not be considered significant. One possible example is to demonstrate that the 90% confidence interval high and low estimate of the water surface is still below the freeboard requirement for the design discharge and the probability of failure at the location is unchanged.

Therefore, even though the best estimate of the cumulative hydraulic impacts exceeds the 0.1 foot threshold, it may be concluded that the level of flood damage risk is unchanged and it is not a significant cumulative hydraulic impact.

4. Sea Level Change

The USACE EC 1165-2-211 requires all USACE coastal activity within the extent of the estimated tidal influence be considered for relative sea-level change effects. The southern portion of the project is within the estimated extent of the tidal influence and therefore sea level change needs to be considered for these areas. See Table 2 and Appendix B. For this programmatic analysis, the extent of the project within the estimated tidal influence subject to potential sea level rise needs to be determined. Sites within this extent will need further site-specific sea-level change analysis. Sites outside the extent of the estimated tidal influence from sea level rise will not need a site-specific sea level change analysis. Changes in relative sea level could impact hydraulic, geotechnical, economic, real estate, and environmental analysis and considerations of the project. An analysis was conducted to determine this extent and a report written and included in Appendix A. The report focuses on the hydraulic considerations and provides information for other disciplines to include in their analysis and documentation.

The southern portion of the SRBPP project is subject to tidal affects and the range of potential sea level rise at the downstream (southern) boundary is estimated to be between 0.42 feet (low estimate) and 2.79 feet (high estimate) between 2013 (estimated construction start) and 2075 (50 years from estimated construction end in 2025).

The high and low value estimate of potential future sea level change determined in accordance with EC 1165-2-211 was used to modify the Common Features HEC-RAS model to estimate the extent of potential sea level change within the life of the project at a programmatic scale. This analysis was conducted for the 1% (100-year) flood and 50% (2-year) flood in order to approximate a reasonable range of conditions. The 1% flood is representative of design conditions and the 50% flood is included to consider potential environmental impacts.

The analysis indicates that the high estimate of potential sea level change (2.79 feet) increases the water surface elevation by greater than 0.1-foot for the areas shown in table 2 and figure 3. The 0.1 foot value was used as the smallest reasonable value for detecting meaningful changes in water surface elevation similar to what is described in the section on Cumulative Hydraulic Effects (section 3).

Table 2. Summary Table of Reaches Affected by Sea Level Change High Estimate

50% flood (2-year)

Reach	Area Affected
Sacramento River	USGS River Mile 48.85 (Downstream of River Landing Drive in the Pocket Area of Sacramento) to the downstream end
Yolo Bypass	2.4 miles south of Delhi Road on Solano County Road 5190C to the downstream end
DWSC	Entire Reach
Lindsey Slough	Entire Reach
Cache Slough	Entire Reach
Haas Slough	Entire Reach
Horseshoe Bend	Entire Reach
3 Mile Slough	Entire Reach
Georgiana Slough	Entire Reach
Miner Slough	Entire Reach
Steamboat Slough	Entire Reach
Sutter Slough	Entire Reach

1% flood (100-year)

Reach	Area Affected
Sacramento River	USGS River Mile 50.85 (Downstream of Dumfries Court in the Pocket Area of Sacramento) to the downstream end
Yolo Bypass	0.1 miles South of Yolo County Road 155 and 104 intersection to the downstream end
DWSC	Entire Reach
Lindsey Slough	Entire Reach
Cache Slough	Entire Reach
Haas Slough	Entire Reach
Horseshoe Bend	Entire Reach
3 Mile Slough	Entire Reach
Georgiana Slough	Entire Reach
Miner Slough	Entire Reach
Steamboat Slough	Entire Reach
Sutter Slough	Entire Reach

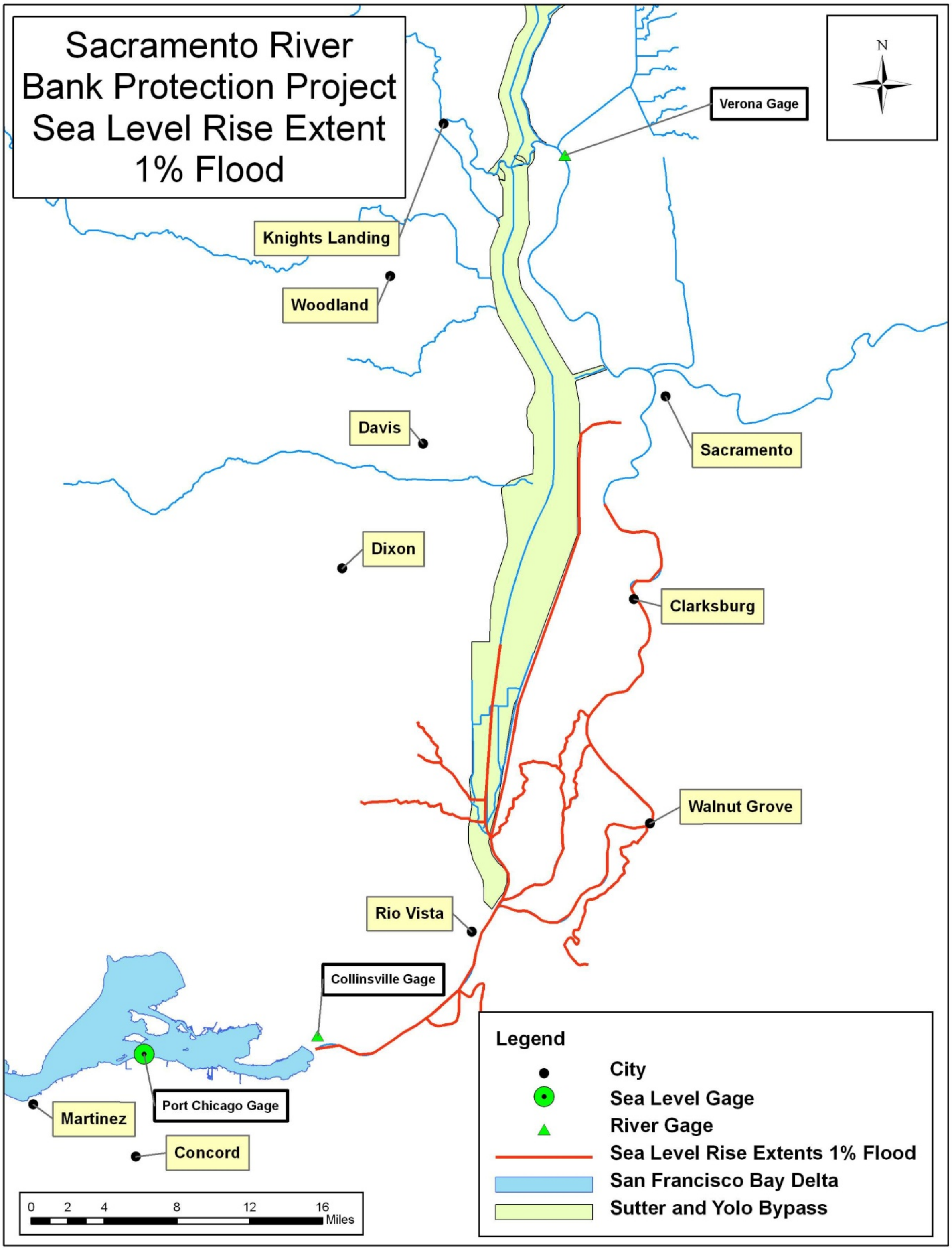


Figure 1. Maximum Estimated Extent of Sea Level Rise – 1% flood

The Yolo bypass and Sacramento River upstream limit of affects was increased by approximately 2 miles from the analysis results to provide a conservative estimate of the upstream limit of future sea level change impacts. Future erosion repair sites outside this adjusted area of potential sea level rise impacts shown below in Table 3 will not need to incorporate sea level change into site specific analysis and design. Future erosion repairs within this adjusted area shown in table 3 will need to address sea level change in their site specific analysis and design.

Table 3.Adjusted Areas Potentially Affected by Sea Level Change

Reach	Area Affected
Sacramento River	Downstream of USGS River Mile 57.5 (Deep Water Ship Channel and Sacramento River intersection in the city of West Sacramento) to the downstream end of the channel at the Collinsville Gage in the Delta
Yolo Bypass	Downstream of Yolo County Road 152 to the downstream end of the channel
DWSC	Entire Reach
Lindsey Slough	Entire Reach
Cache Slough	Entire Reach
Haas Slough	Entire Reach
Horseshoe Bend	Entire Reach
3 Mile Slough	Entire Reach
Georgiana Slough	Entire Reach
Miner Slough	Entire Reach
Steamboat Slough	Entire Reach
Sutter Slough	Entire Reach

The requirements of EC 1165-2-211 apply to this federal project but are different than the state of California requirements and procedures for addressing sea level change. Both procedures yield similar numbers for the high sea level rise estimate. Since the high estimate provides the maximum estimated extent of sea level rise, the differences in the procedures are not significant. In fact, the USACE procedure provides a slightly more conservative estimate of the geographic extent of sea level rise than the state guidance.

A preliminary programmatic stone protection analysis indicates that sea level change is not likely to impact the size and gradation of stone protection. This is discussed in more detail in Appendix A. However, the site specific hydraulic analysis should consider addressing future local changes to stage, velocity, wave characteristics, and other site-specific hydraulic considerations for these reaches affected by sea level change.

5. Hydraulic Analysis and Design Considerations

The repair sites for the SRBPP will be analyzed and designed at a site specific level. The hydraulic analysis and design considerations included in this report are programmatic in nature and do not necessarily apply to every site. Similarly, there may be other hydraulic analysis and design considerations that need to be considered for a particular site that are not mentioned in this report. The information in this report does not prescribe or limit the hydraulic analysis and design considerations for site specific design. This will be determined on a case by case basis. This report simply outlines the hydraulic considerations that may be included in site specific design and analysis and how they could be included in general terms.

The without project condition will be analyzed using the baseline hydraulic model used for estimating the cumulative hydraulic impacts as described in section 3. The with-project conditions will use the hydraulic model used for estimating the cumulative hydraulic impacts as described in section 3. The cumulative hydraulic impacts model is the without project conditions model (the baseline model) with the addition of all implemented project features as of the date of the analysis. This approach provides a way to adaptively compare the with-project and the without project conditions as the project is implemented.

The scale of the hydraulic analysis will consider the level of risk and cost of the analysis and the anticipated repair construction cost. For example, an expensive, thorough, and detailed data collection and analysis effort may not be warranted for lower risk repairs that are relatively inexpensive. Further, it is imprudent to spend more money collecting data than the entire cost of the repair. In these situations, a less robust hydraulic analysis will likely occur. In general it is anticipated that a 2D hydraulic model based analysis is needed for most repair sites to assess changes in flow patterns and hydraulics due to the repair. However, for smaller channels and some other conditions a 1D hydraulic model based analysis may be appropriate. The level of analysis conducted for each repair site will be determined on a case-by-case basis using engineering judgment considering risk, funding, repair costs, and other considerations.

Some of the hydraulic considerations occur at the reach scale while others are more appropriately considered at the smaller local scale. Generally the reach scale considerations will be addressed in the site-specific EDR's (Engineering Document Reports) while the local scale considerations will be addressed in the site-specific analysis and design leading to plans and specifications, such as in the DDR's (Design Document Report's). Currently repair sites only have a DDR for the repair sites and generally only consider local scale hydraulic analysis. Future repairs conducted for the SRBPP Phase II additional 80,000 linear feet will consider both the reach and local scale factors in their hydraulic analysis as determined appropriate using engineering judgment.

5.1. Reach Scale Hydraulic Analysis and Design Considerations

Currently repair measure selection and design generally only consider local scale hydraulic factors. However, there are times where reach scale issues are a significant issue contributing to erosion at an erosion repair site. Therefore, reach scale issues will be addressed to best select and design erosion repairs as part of the SRBPP Phase II additional 80,000 LF implementation. It is generally anticipated that larger scale issues will be addressed in the site specific EDR. Some of these reach scale issues include operation of weirs, channel stability and sediment trends, river meander migration and cut-offs, and reservoir operations. However, there could be other issues that are identified and addressed during site specific analysis and design.

5.1.1. Weir and Flow Splits

The SRFCP operates as a system with a series of connected channels and bypasses. The bypasses are designed to divert flood flows from the main channel of the Sacramento River into either the Sutter or Yolo Bypass. The intent is that the water diverted from the main channel of the Sacramento River reduces flood damage risk along the Sacramento River. Changes such as topography and vegetation in the vicinity of the weirs that control the flow into the bypasses could alter the flow splits between the main channel and the bypasses. This could increase flood damage risk for the portions of the SRFCP that experience greater flows as a result of the changes. Such a change could be a setback levee in the vicinity of a weir that reduces the water surface elevation for flood flows. This increases the proportion of the flow that continues in the main channel of the Sacramento River. Options for repair measures of erosion sites could be limited for this reason in the vicinity of the weirs. This will be addressed as needed during site development and selection of repair measures such as in the site specific EDR

5.1.2. Channel Stability and Sediment Trends

Reach scale channel instability and sediment trends could be a significant factor affecting erosion at a repair site. Repair sites could be located in a reach of a channel that is unstable. Repairs in unstable reaches need to be analyzed at a reach scale to ensure the repair does not contribute to further instability in the reach. This includes a repair contributing to creation of new or further degradation of existing erosion sites. A good indication of channel instability could be the presence of a lot of historical repair sites clustered in the same vicinity. This would indicate that larger scale reach hydraulic analysis is needed.

Similarly, large scale sediment trends could be contributing to erosion at a repair site. For example, a repair site could be located in a predominantly aggregating reach located downstream of a reach that is predominantly degrading. This point could potentially experience large amounts of deposition that could force the main channel against the bank, contributing significantly to erosion at the site. Similarly, a site in a predominantly degrading reach could be subject to erosion caused as the channel incises and erodes outward toward the channel banks. The selection and design of repair measures needs to adequately address reach scale channel stability and sediment trends to maximize the effectiveness of the repair.

5.1.3 River Meander Migration and Cut-offs

River channels tend to develop looping “S” patterns called meander bends when looking from above. These meanders bends tend to move downstream over time. If a meander bend makes too “sharp” of a turn, it becomes more efficient for the water to move in a generally straight line across one of the “C” shaped meander bends. Eventually the straight line portion of the flow becomes the predominant channel and the “C” portion becomes abandoned and gradually fills in. The formation of the straight line portion of the flow is generally called a “cut-off. “ The migration of the meander bends downstream and the formation of “cut-offs” can significantly alter the hydraulic conditions at repair sites over the life of the project. Repair sites located in channels with active meander bend and cut-off processes need to account for changing hydraulic conditions during the life of the project when selecting and designing the repair measure. This may include need to expand the study area to include more of the channel to adequately address possible changing hydraulic conditions. The potential for river meander migration and cut-offs will generally be addressed during site selection and development of the site specific-EDR.

5.1.4. Reservoir Operations

Reservoir Operations need to be considered when selecting and designing repair alternatives. Over time, consistent reservoir operations are included in the hydrologic record and the channel reaches a state of dynamic equilibrium. These conditions may be fairly accurately represented in hydraulic models. If the operation of a reservoir is altered significantly, however, the channel may undergo significant and rapid changes before reaching a new state of dynamic equilibrium. If it can be reasonably anticipated that a reservoir that controls flows in the channel of a repair site may be altered in the future, a conservative selection and design of a repair measure is needed that addresses the changed reservoir operation. Often times changes to reservoir operations may induce channel instability which was discussed previously. Therefore, hydraulic analysis for repair measure selection and design needs to include possible future changes to reservoir operations and the resulting channel behavior.

5.1.5. Other Reach Scale Hydraulic Analysis and Design Considerations

There could be other reach scale hydraulic analysis and design considerations. One example is the narrowing of a channel from one reach to another. The narrowing of the reach would tend to cause erosion as the water moves through the narrower reach at a higher velocity, contributing to additional erosion. Such sites may benefit from a setback levee that allows the channel to flow unrestricted at a lower velocity. This and other reach scale considerations will be addressed during selection and design of repair measures.

5.2. Local Scale Hydraulic Analysis and Design Considerations

As noted in the background information, the SRBPP provides erosion repairs to the SRFCP but does not change the SRFCP discharges or the required minimum freeboard requirements for these discharges.

5.2.2. Stage, Discharge, and Velocity Considerations

Changes to water surface elevations (stage), discharge, and velocities from project implementation need to be considered during repair measure selection and design. This is typically analyzed with 1D or 2D hydraulic models during hydraulic analysis. For most sites, it is anticipated that a 2D analysis is needed to better account for changes in velocity patterns and magnitude. A 1D model tends to “average” out the changes over a larger area and does not allow for analyzing changes in flow patterns. These flow patterns can be a very important consideration during measure selection and design.

For example, a repair could encroach too far into the channel, resulting in locally increased velocities and water surface elevations leading to increased erosion. Similarly, a repair could move the point of the higher velocity closer to the opposite bank, increasing erosion pressures on the opposite bank that may or may not be adequately protected. The repairs could also redirect higher velocities against a nearby bank that may not be adequately protected. This is often seen when a new erosion site appears downstream of a recently repaired site. Even repairs that may not seem to have negative hydraulic impacts could have issues. For example, a relatively short set-back levee could induce a large eddy that reduces the effective conveyance area with similar results to a repair that encroaches too much on the channel. Another item that could affect the stage, discharge, and velocities in the vicinity of a repair site includes the hydraulic roughness. This could be due to planting new or a different type of vegetation on a repair site or removing vegetation from the repair site. This could affect the water surface elevations, discharge of the channel, and the velocities in the vicinity of the project. Hydraulic analysis will be conducted on a site-specific level during repair measure selection and design as determined by engineering judgment.

5.2.3. Rock Protection Design

While not all repairs include rock protection, those that require rock protection will need site specific hydraulic analysis to support site-specific measure selection and design of the rock protection. This includes the size and gradations for the rock used in SRBPP repairs. This may include analysis of rock to protect against erosion from channel flow, boat waves, and/or wind waves. Much of the information in this section originate from a draft, non-certified, unpublished report (USACE 2006) but is considered the best available information at this time and is appropriate for this programmatic level report since no designs decisions are being made in this report.

5.2.3.1. History of Rock Gradation for SRBPP Repairs.

Historically rock used on SRFCP has followed standardized gradations that were designed to protect against erosion from channel velocity. These were generally based on USACE studies in 1948, 1956, 1973, and 1992. The standardized gradations used in the SRFCP varied over the years starting in 1936 to present. A significant revision of the rock size and gradation occurred in 1974. This resulted in two standard rock gradations that were used on SRBPP projects until about 2006. These gradations are shown below in Table 4 and Table 5 and both used a filter for the design.

Table 4. 1974 Standard 160 lb Rock Gradation

Stone Weight (lbs.)	% Smaller by Weight
160	100
100	80-95
50	45-80
20	15-45
5	0-15

Table 5. 1974 Standard 200 lb Rock Gradation

Stone Weight (lbs.)	% Smaller by Weight
220	100
176	85-100
110	60-85
55	35-65
22	15-35
11	0-15

In 2006 USACE developed a new gradation of launch rock for Sacramento River USGS mile 40 to 60 left bank based on EM 1110-2-1601 shown in Table 6 below. The motivation for the design appears to be to account for recreational boat and wind caused wave erosion. This is a significant addition to previous designs that were only designed to protect against erosion from channel velocity. It appears that wind waves controlled the design for this section of the river.

The 2006 recommended gradation uses extra thickness that does not require a filter if designed and constructed properly. This typically means it needs to have a thicker section of rock than with an equivalent design that uses a filter (typically 2.5 – 4 times thicker). A thickness of 3 feet was selected. Also, for underwater placement, it is recommended to increase the volume by 50%.

Table 6. 2006 Recommended Rock Gradation.

Stone Weight (lbs.)	% Smaller by Weight
400	100
250	70-100
100	50-80
30	32-58
5	16-34
1	2-20
Less than ½” max. dimension	0-10

There is another gradation that was developed specifically for the Deep Water Ship Channel shown below in table 7. It is not clear how or when this gradation was developed but USACE 2006 states it is to be used for all slopes facing the Deep Water Ship Channel.

Table 7. Deep Water Ship Channel Gradation

Stone Weight (lbs.)	% Smaller by Weight
1,300	100
1,000	80-90
500	50-70
100	10-30
50	0-10

5.2.3.2. Current and Future Rock Protection Gradations for SRBPP Repairs.

Subsequently to the 2006 recommended gradation, plans and specifications for construction have included and/or adapted this gradation for use in soil filled quarry stone. Soil filled quarry stone is a mixture of the rock gradation and soil such that the rocks maintain three-points of contact with other rock and the entire mass is 70% rock and 30% soil by volume. This implies that the void ratio (volume of the voids divided by the volume of the rock) for the rock is over 40%. This seems unreasonably high and other USACE engineers agree this is an unreasonably high void ratio that is not possible to construct. The result is that it is unlikely that sites constructed to these specifications do not have a majority of the rock in three-point contact as intended. This could lead to faster erosion of the repair and reduce the effectiveness of the repair. The addition of the soil to the quarry stone does not appear to be documented in any design document report (DDR) or similar document. However, it does appear future repairs should reconsider the proportion of the rock to the soil and the intent of the design. It is likely that the proportion of the mixture that is soil will be reduced on future repairs significantly in the future compared to repairs constructed from 2006 to present.

In addition to the rock and soil proportion concern, some designs have altered the proportion of the smaller particles in the gradation and soil properties in an effort to reduce erosion from high water. These changes are documented in individual site-specific DDR's. Any modification to the gradation in future repairs will similarly be documented in a site-specific EDR or DDR.

Another concern with the use of the 2006 recommended gradation is that it was developed for the Sacramento River for USGS river miles 40 – 60 but is applied outside of this reach. While it may be applicable in other situations, this has not been checked. In particular, since wind caused waves controlled the size of the rock for the gradation, it is expected that channels further downstream would require even larger rock size as the wind waves are significantly larger. In addition, they could be subject to large ship traffic similar to the Deep Water Ship Channel. Also, repair sites at other locations could be subject to higher velocities than included in the 2006 recommended gradation. Since channel velocity did not control for Sacramento River USGS river mile 40 – 60, this may not be an issue, but it should be checked. In any case, the sizing and gradation of rock for rock protection will be a site-specific design that considers protection from channel velocity as well as wind and recreational boat waves for inclusion in future SRBPP repairs based on an appropriate level of analysis.

5.2.4. Other Hydraulic Considerations

There are other local scale hydraulic considerations that may be included in hydraulic analysis based on engineering judgment. For example, the transitions of repairs should be designed to provide a smooth hydraulic transition and avoid abrupt changes that can contribute to local erosion and sedimentation issues and possibly endanger the functionality of the repair. In addition, the elevation of the top of the rock protection and the upstream and downstream extents of the rock protection needs to be informed by the hydraulic analysis. Another consideration is sedimentation and/or erosion and/or scour near structures within the repair site. For example, repairs could contribute to sedimentation of pumps or contribute to erosion that could threaten the integrity of the SRFCP or nearby structures. These could include water intake and discharge facilities, bridges, docks, pipelines, and similar structures. These details and other items will be included and addressed as needed in hydraulic analysis in support of site-specific measure selection and design as needed based on engineering judgment.

6. Hydraulic and Hydrologic Criteria

The SRBPP is an erosion control project for the protection of the existing levees and flood control facilities of the SRFCP that spans over 50 years. There are many pieces of this complex project. The project authorization is to reduce the risk of levee failures within the SRFCP system from erosion. Site specific analysis will be conducted as part of developing site-specific Engineering Document Reports (EDR's) and Design Document Reports (DDR's) during site specific design. The hydraulic and hydrologic criteria used for the SRFCP and the SRBPP is clarified in this section.

6.1 The Sacramento River Flood Control Project Hydraulic Criteria

The SRFCP is the Federal flood control system for the Sacramento River Valley of California. The design hydraulic criteria for this system was updated, clarified, and summarized in the 1957 profiles (see Appendix B). The 1957 profiles is considered the hydraulic design criteria for the SRFCP. The 1957 profiles define the minimum freeboard requirements for each segment of the Sacramento River Flood Control Project (SRFCP) for a given design discharge. The return frequency of these design discharges varies from reach to reach.

6.2 The Sacramento River Bank Protection Project Hydraulic Criteria

The SRBPP is a project that provides erosion repairs for damaged portions of the SRFCP. The SRBPP project does not modify the SRFCP hydraulic design criteria as outlined in the 1957 profiles. The SRBPP does check to ensure that erosion repairs are designed so that the hydraulic conditions before and after repair are similar with no significant changes. The focus is on the differences between the before and after hydraulic conditions and not whether the levee segment is designed to pass the design discharge of the SRFCP with the required freeboard. SRBPP is not authorized to raise levees or otherwise change the SRFCP hydraulic criteria. It is assumed the freeboard for the design discharge of the 1957 profile before and after the repair will be unchanged if the stage changes less than 0.1 foot for a significant flood event (e.g. 1/100 ACE event). The less than 0.1 foot change in stage hydraulic criteria is considered well within the error of the hydraulic modeling software. If the modeled change is less than 0.1 foot from before and after repair than the stage

In the past, the 1/100 ACE flood event has been used as a representative flood condition to check that the stage before and after repair does not increase in the model by greater than 0.1 foot and that the flow patterns will not change in a manner that may cause significant changes to erosion or sedimentation outside the repair site.

References

McCollam, Colonel A. E. Submittal to Division Engineer. Levee and Channel Profiles, Sacramento River Flood Control Project. 1 July 1957.

Stimson, H. L. Secretary of War. Flood Control – Sacramento and San Joaquin River Systems, California. Letter to the 62nd Congress 1st session House of Representatives Document No. 81. June 27, 1911.

"Water Resource Policies and Authorities Incorporating Sea-Level Change Considerations in Civil WorksPrograms." EC 1165-2-211. 24 Feb. 2011 < http://140.194.76.129/publications/engcirculars/ec1165-2-211/entire.pdf

"Sacramento River Mile 40 to 60 Rock Riprap Gradation Design for River Currents, Wind and Boat Waves." Final Draft Report for Review. USACE 2006.

Appendix A, Final Sea Level Change Analysis Report

Sacramento River Bank Protection Project
Phase II, 80,000 Linear Feet
Post Authorization Change Report
Hydraulic Appendix
FINAL SEA LEVEL CHANGE ANALYSIS REPORT
June, 2011

1. Introduction and Authorization

The Sacramento River Bank Protection Project (SRBPP) is an erosion control project for the protection of the existing levees (including bank protection) and flood control facilities of the Sacramento River Flood Control Project (SRFCP). This project was originally authorized in 1960 and has included subsequent authorizations and phases. The original Phase II authorization was in 1974. The Sacramento District of the United States Army Corps of engineers (USACE) is developing a programmatic Post Authorization Change (PAC) document and EIS/R (Environmental Impact Statement/Report) addressing the additionally authorized 80,000 linear feet of erosion control work to Phase II of erosion as per the Water Resources Development Act (WRDA) 2007. The project authorization is to reduce the risk of levee failures within the SRFCP system from erosion.

2. Objectives and Scope

Recent research indicates continued or accelerated rise in global mean sea level height based on decades (and in some cases centuries) of measurements. Climate change has been identified as a likely cause of the increase in global sea level height by many researchers but is still subject to spirited debate. However, the reality of the observed rise in global sea level height at project specific locations and local vertical land movement needs to be adequately addressed by projects in and near coastal areas regardless of the causes.

EC-1165-2-211 “Water Resource Policies and Authorities Incorporating Sea-Level Change Considerations in Civil Works Programs” was enacted July 1, 2009 to provide guidance for “incorporating the direct and indirect physical effects of projected future sea-level change in managing, planning, engineering, designing, constructing, operating, and maintaining USACE projects and systems of projects.” EC-1165-2-211 requires all USACE coastal activity within the extent of the estimated tidal influence be considered for relative sea-level change effects.

The state of California requirements and procedures for considering sea level rise are not the same as the requirements and procedures outlined in EC-1165-2-211. The reader is referred to the state of California for the most recent requirements and procedures for projects subject only to the requirements of the state of California. However, this is a federal project subject to the requirements of EC-1165-2-211.

The downstream (southern) boundary of the SRBPP project ends in the San Francisco Bay Delta (Delta) as shown in figure 1. The Delta is subject to ocean tidal fluctuations, influence, and any potential sea level change. This impacts the hydraulics of project channels upstream of the Delta for some distance. Therefore, a sea level change analysis is needed to determine the impacts of sea level change on the SRBPP Phase II additional 80,000 linear feet PAC and EIS/R documentation. The Sacramento River at the

Collinsville gage is selected for the downstream boundary of the analysis because it matches existing hydraulic models and has available data necessary for the analysis. See Figure 1 for a map for the area considered in this analysis.

The purpose of this report is to analyze the upstream effects from estimated future changes (increases or decreases) in the downstream sea level elevation on the SRBPP in accordance with EC 1165-2-211. This will be included in the Phase II 80,000 linear feet authorization PAC and EIS/R documents. The PAC and EIS/R documents are programmatic and will be followed by appropriate site-specific engineering document reports (EDR's), designs, and accompanying environmental documentation. Therefore, this analysis is programmatic in nature and subsequent site-specific sea level change analysis and documentation may be needed.

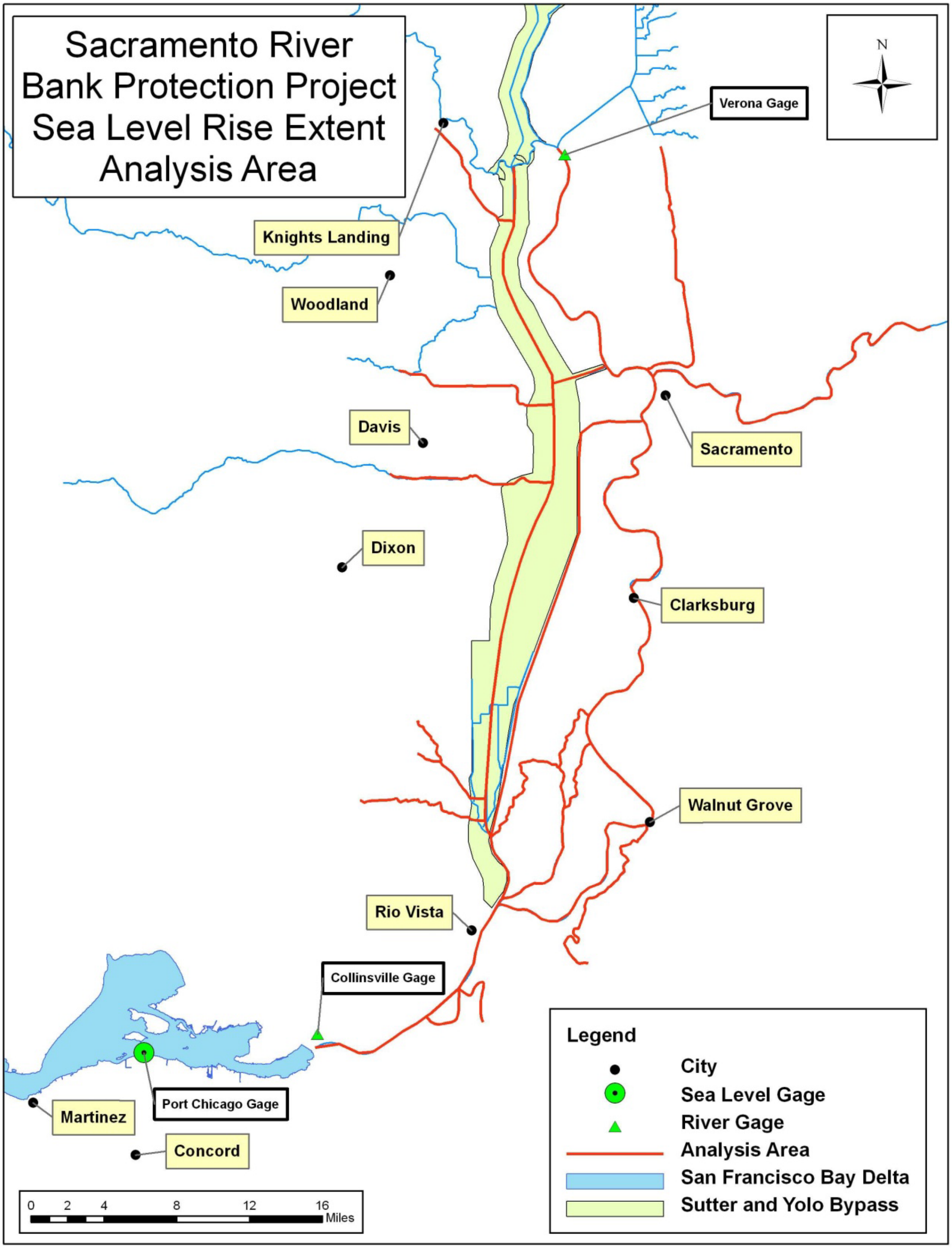


Figure 1: Map Showing Analysis Area

3. Potential Effects of Sea Level Change on the Project

EC 1165-2-211 requires that relative sea level change be considered. This includes both changes in sea level and the adjoining land elevations. Changes in relative sea level could impact hydraulic, geotechnical, economic, real estate, and environmental analysis and considerations of the project.

3.1 Hydraulic Considerations

Sea level changes could affect stages (water surface elevation), velocity magnitude and directions, and wave characteristics. In general, it would be expected to slow down velocities in the vicinity of the sea level rise due to backwater effects. The elevation of the top of the bank protection depends on the water surface elevation and the anticipated wave heights. Wave heights are a function of the fetch (the length of water over which a given wind has blown) and sometimes depth of the water over which the wind blows. Therefore, an increase in sea level could also lead to higher wave heights (from longer fetches or greater depths) in addition to needing the bank protection to be raised based on sea level rise alone. However, the increase in wave height from increased fetch and depth may be insignificant.

3.2 Geotechnical Considerations

3.2.1 Subsidence

Subsidence is a concern in the Sacramento-San Joaquin Delta. Subsidence of land is caused by decomposition of organic carbon in peat soils. The decomposition is occurring due to tilling/burning of soils, erosion by wind or water, lowering of water surface elevation and compaction/desiccation of organic soils with high saturated water contents (USGS/SF Estuary and Watershed Science). These factors contributing to subsidence occur as a result of agricultural practices. Therefore, agricultural areas are subject to subsidence. These agriculture practices do not occur on the levee so the levee is not generally subject to subsidence. However, the levee foundation (or possibly the levee itself) may consolidate from the weight of the levee and other items on the levee (e.g. trees, vehicles) which leads to lowering of the levee crown elevation.

Lowering of the levee crown due to consolidation is dependent upon localized conditions and is difficult to estimate over a broad geographic area as needed for this analysis. For this programmatic analysis, it is assumed that there is no reduction in levee crown elevation due to consolidation. This assumption is not conservative from an engineering perspective but it aligns with the project's authorization and Corp's policy. It is assumed any reduction in levee crown elevation will be regularly repaired as part of maintenance. This assumption is consistent with USACE policy and project documents. According to 33 CFR 208.10, cited in every project O&M Manual:

> "(b) Levees (1) Maintenance: Periodic inspections shall be made by the Superintendent to insure that the above maintenance measures are being effectively carried out and, further, to be certain that: (i) No unusual settlement, sloughing, or material loss of grade or levee cross section has taken place."

Also, the current O&M manual states that "immediate steps will be taken to correct dangerous conditions disclosed by such inspections" (USACE). For that reason, assuming the levee height is maintained to its original design elevation is a valid assumption for this analysis. It is also assumed that the channel is not subsiding (i.e. there is no large scale subsidence that would include the channel). If the above assumptions are correct, subsidence should not be a significant issue for hydraulic modeling. These assumptions are consistent with USACE policy and the project. As a result of these assumptions, for this large scale hydraulic analysis, relative sea level change is the same as sea level change. That is, the levees and channels have no vertical movement.

3.2.2 Probability of Levee Breach

An increase in water surface elevation increases the probability of levee breach due to internal erosion and slope instability. This is due to an increase in seepage forces (pore pressure) and due to an increase in the water level (phreatic surface) within the levee, which affects the seepage exit area on the landside slope of the levee. This could lead to increased probability of levee breach. In addition, an increase in water surface elevation increases levee loading duration, increasing likelihood of levee breach formation.

3.3 Economic Considerations

Another consideration for this study is the project's economic analysis. If consolidation does not occur but the land protected by the levee subsides, this could cause the land-side levee height to increase. If the water surface elevation also increases, there would be a greater difference between the landside levee elevation and the water surface elevation, which could increase the probability of the levee breaching in the future. Furthermore, the larger height differential could lead to greater discharges through larger levee breaches and cause increased flood depths and damages. If relative sea level change is not considered, then the damages and benefits could potentially be underestimated. Seepage and slope stability are outside the scope of the SRBPP project.

3.4 Real Estate Considerations

Sea level changes can also have an impact on Real Estate since the designs needed to address future sea level rise and subsidence may require additional real estate needs. Future repairs and/or construction may require additional real estate to address higher landside slopes and/or increased seepage. Potential items that could be incorporated into the designs include seepage berms or stability berms on the landside of the levee to stabilize its slope. In addition, future maintenance, repair, rehabilitation, and replacement activities may require additional real estate due to relative sea level change.

3.5. Operation, Maintenance, Repair, Rehabilitation, and Replacement (OMRRR) manuals

According to EC-1165-2-211, relative sea level change also needs to be considered for Operation, Maintenance, Repair, Rehabilitation, and Replacement (OMRRR). It may be necessary or most cost effective to address sea level change in the OMRR manuals rather than during initial design and construction. For example, the actual sea level change is not specifically known and it may be more cost effective to address sea level change through the life of the project as it occurs, than to overdesign the project for a level of relative sea level change that may not occur. This needs to be considered in the development of OMRR manuals for project repair sites.

3.6 Environmental Considerations

Ecosystems in the vicinity of the existing and future water surfaces could also be impacted from relative sea level changes. This could destroy, damage, or change ecosystems that are:

1) Currently infrequently inundated but would become more frequently inundated due to sea level rise,
2) Currently regularly inundated but would become permanently inundated due to sea level rise,
3) Currently regularly inundated but would become less frequently inundated due to a drop in sea level,
4) Currently shallow water habitats but would become deeper due to sea level rise,
5) Or currently deep water habitats but would become shallow water habitats due to a drop in sea level.
6) Currently exhibiting consistent salinity characteristics that would change due to a rise or drop in sea level

Sea level change therefore could potentially alter the ecosystem of the Sacramento River system, including the delta region, but these environmental impacts will be addressed in other reports.

4. Sea Level Change Analysis

4.1 Analysis Introduction

The impact of sea level change depends on the magnitude of the elevation change for a given location. The focus of this report is to develop potential hydraulic analysis considerations for the SRBPP from a large-scale programmatic level and not a detailed site specific design. The purpose of the report is to determine the potential geographic extent of the impacts of sea level change over the next 50-year life of the project, and to determine what hydraulic analysis considerations are important to address in site specific analysis and design. The use of this analysis, hydraulic model, and results are only appropriate for the large scale programmatic analysis in this report and are not appropriate for site specific analysis or decision making.

4.2 Potential Geographic Extents of Sea Level Change Estimate Procedure

4.2.1 Background of Geographic Extent Estimation

Tidal effects are generally accepted to be negligible above the Verona gage on the Sacramento River. The results from the Common Features model sensitivity analysis confirm that there are no significant tidal impacts at or above the Verona gage on the Sacramento River. A rough sea level change estimate was modeled using an existing HEC-RAS model (a 1D hydraulic model developed by USACE) developed by the Common Features Project for estimating the sensitivity of the model from changes in the downstream water surface elevations and datum uncertainties. This model (called “the Common Features HEC-RAS model” in this report) was modified and analyzed to estimate the potential geographic extent of sea level rise and hydraulic considerations for future site-specific analysis following guidance in EC-1165-2-211. It was assumed that SRBPP construction would start in the year 2013 and end in 2025. The project's design life was taken as 50 years, so the change in elevation was analyzed from the year 2013 to 2075 (62 years). This assumes construction starts in 2013 and ends in 2025 with a 50-year project life (12 years of construction and 50-years after construction ends is the 62 years).

EC-1165-2-211 requires a low, medium, and high estimate for relative sea level change and provides estimating procedures. For this analysis only the high and low estimate are used to give the maximum estimated extents of sea level rise that needed to be considered for the project.

4.2.2 Low Sea Level Change Estimate

The low rate of sea level change was determined based on the historic rate of sea level change and from the mean sea level trends for the US Tide Stations in accordance with EC-1165-2-211. The downstream end of the hydraulic model used for this analysis is approximately the Collinsville river gage. Since sea level change trend information for the Collinsville gage is not readily available, the Port Chicago, Ca. tidal gage information was used to estimate the expected trend for the downstream stage boundary conditions for the hydraulic model. This gage was selected since it is in the vicinity of the Collinsville gage. It is assumed that the Collinsville gage would experience similar changes in sea level elevation to the Port Chicago gage. See figure 1 for the location of the Port Chicago and Collinsville gages.

Information on the Port Chicago, California gage was found at:
http://tidesandcurrents.noaa.gov/sltrends/sltrends_station.shtml?stnid=9415144 (2/22/2011). A screen shot of the website is shown in Appendix 1. The expected mean sea level trend at the Port Chicago tidal gage is 2.08 mm/year with a 95% confidence interval of + / -2.74 mm/yr (NOAA). This trend is based on monthly mean sea level data from 1976-2006. Since data has been recorded at that gage for less than 40 years, the range of uncertainty is large as expected based on EC-1165-2-211. EC 1165-2-211 suggests that tide stations should have a minimum of 40 years of data in order to use the trend to estimate future sea level elevations. When the Port Chicago tidal gage trend is compared to the San Francisco tidal gage trend (2.01 mm/yr with a 95% confidence interval of +/-.21 mm/yr, NOAA), the trends are similar. However, the San Francisco gage's range of uncertainty is much smaller since the trend is based on data from 1897-2006 (106 years). Since this study will be used as a large scale programmatic analysis and the Port Chicago gage trend agreed well with the long established San Francisco tidal gage trend, it is concluded that no additional gage analysis is needed.

4.2.3 High Sea Level Change Estimate

The high estimate was determined using equation (3) in Appendix B of EC 1165-2-211. The information provided in this section is either required or provided by EC-1165-2-211. T_1 was taken as the difference between the year 2013 and 1986, while T_2 was taken as the difference between 2075 and 1986. The constant b was taken to be 1.005E-4 for the modified NRC Curve III (provided by EC-1165-2-211). The change in eustatic (global) sea level was estimated to be 2.71 ft over the 62 years. The change in relative (local) sea level is estimated to be 2.79 ft over the 62 years. The computations are shown in Appendix 2. The local sea level rise estimate is what is important for this analysis.

4.2.4 Hydraulic Model Development

For this analysis, an existing HEC-RAS model (a 1D hydraulic model developed by USACE) was used, which was developed by the Common Features Project for estimating the sensitivity of the model from changes in the downstream water surface elevations and datum uncertainties. This model (called “the Common Features HEC-RAS model” in this report) was modified and analyzed to estimate the potential geographic extent of sea level rise and hydraulic considerations for future site-specific analysis. This common features model has been previously reviewed and is appropriate to use for this broad-scale programmatic analysis.

As mentioned in section 3.2.1, for this analysis it is assumed that any reduction in levee crown elevation is repaired as part of on-going maintenance activities so that there is no change in levee crown elevation. Furthermore, it is assumed that the channel is not subsiding or otherwise changing geometry. It is expected that sediment movement in the channel will change the channel geometry to some degree. However, for this broad scale programmatic analysis, sediment effects are not considered and should not have a significant impact on the analysis.

In Hec-DSSVue (a program developed by USACE for managing and modifying hydraulic and hydrologic data), the downstream stage hydrograph boundary condition was modified by duplicating the existing conditions hydrographs for the Georgiana Slough, Sacramento River, and Three Mile Slough, and adding the high and low estimates for sea level rise. This was done for the 1% chance exceedance flood (1% flood, 100-year flood) and 50% (2-year) flood. (The one percent flood has 1 chance in 100 of being exceeded in any given year, while the fifty percent flood has a 1 chance in 2 of being exceeded in any given year). The 1% flood is representative of engineering analysis considerations and the 50% flood representative of environmental analysis considerations. In HEC-RAS, the unsteady flow data was edited so that the stage hydrographs corresponded to the modified hydrographs with the added estimates. The unsteady flow analysis was run for the 4 conditions (1% flood high estimate, 1% flood low estimate, 50% flood high estimate, and 50% flood low estimate) and the results were analyzed.

4.3. Potential Geographic Extent of Sea Level Change Results

After the models were run, the output files were opened up in HEC-DSSVue and the High/Low estimates for the 1% flood and 50% flood, were compared against the existing conditions model. Changes of less than 0.1 feet (ft) were considered insignificant and well within the range of model error. Reaches with a change in water surface elevation of greater than 0.1 ft were determined, and a summary of the results is shown in Table 1. The maps which show the extents of the affected reaches for the 1% and 50% floods are shown in Figure 2 and Figure 3, respectively.

Table 1: Summary Table of Reaches Affected by a 2.71 Ft Increase in Sea Level

50% flood (2-year)

Reach	Area Affected
Sacramento River	USGS River Mile 48.85 (Downstream of River Landing Drive in the Pocket Area of Sacramento) to the downstream end
Yolo Bypass	2.4 miles south of Delhi Road on Solano County Road 5190C to the downstream end
DWSC	Entire Reach
Lindsey Slough	Entire Reach
Cache Slough	Entire Reach
Haas Slough	Entire Reach
Horseshoe Bend	Entire Reach
3 Mile Slough	Entire Reach
Georgiana Slough	Entire Reach
Miner Slough	Entire Reach
Steamboat Slough	Entire Reach
Sutter Slough	Entire Reach

1% flood (100-year)

Reach	Area Affected
Sacramento River	USGS River Mile 50.85 (Downstream of Dumfries Court in the Pocket Area of Sacramento) to the downstream end
Yolo Bypass	0.1 miles South of Yolo County Road 155 and 104 intersection to the downstream end
DWSC	Entire Reach
Lindsey Slough	Entire Reach
Cache Slough	Entire Reach
Haas Slough	Entire Reach
Horseshoe Bend	Entire Reach
3 Mile Slough	Entire Reach
Georgiana Slough	Entire Reach
Miner Slough	Entire Reach
Steamboat Slough	Entire Reach
Sutter Slough	Entire Reach

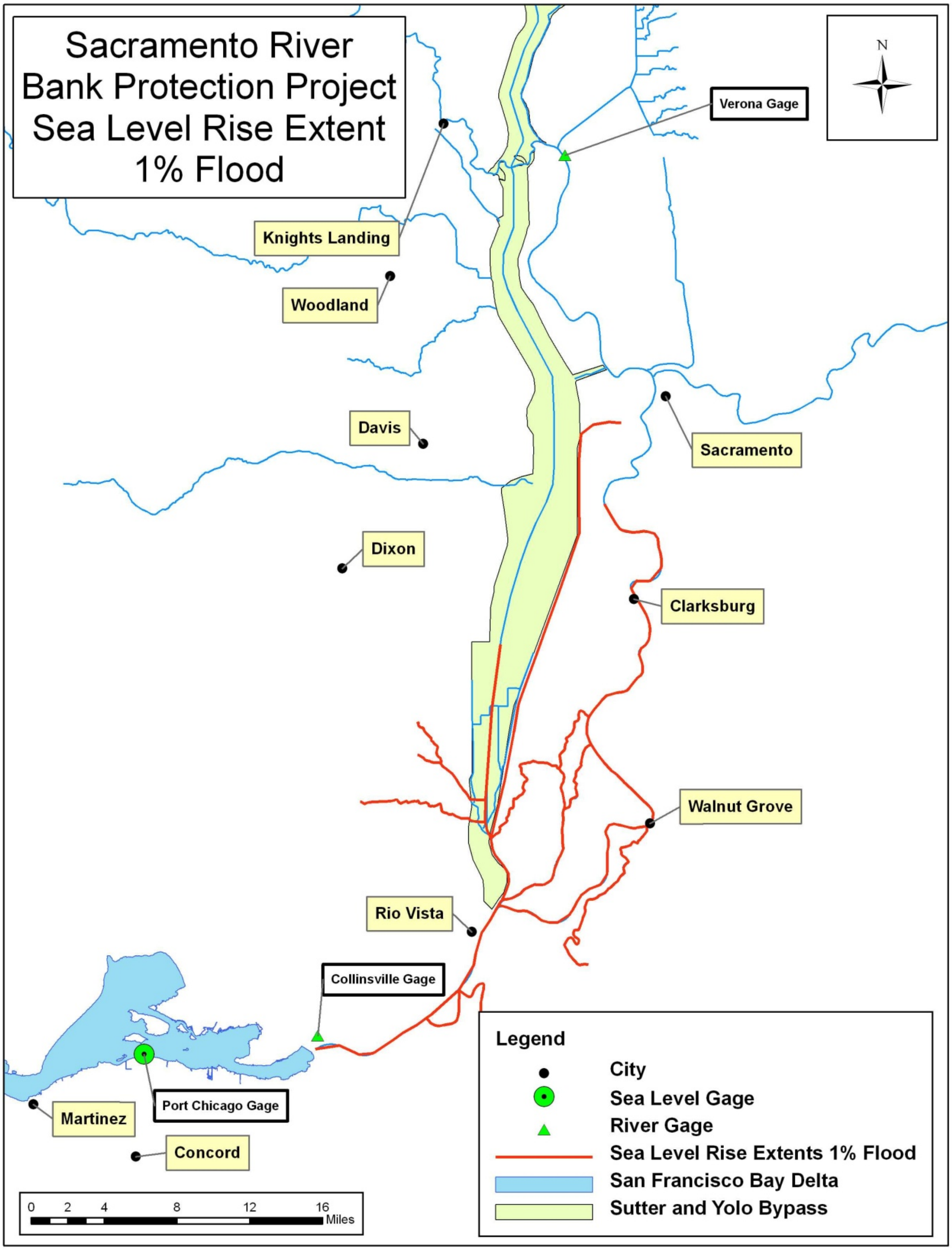


Figure 2: Maximum Estimated Extent of Sea Level Rise – 1% flood assuming 2.71 ft rise at Collinsville

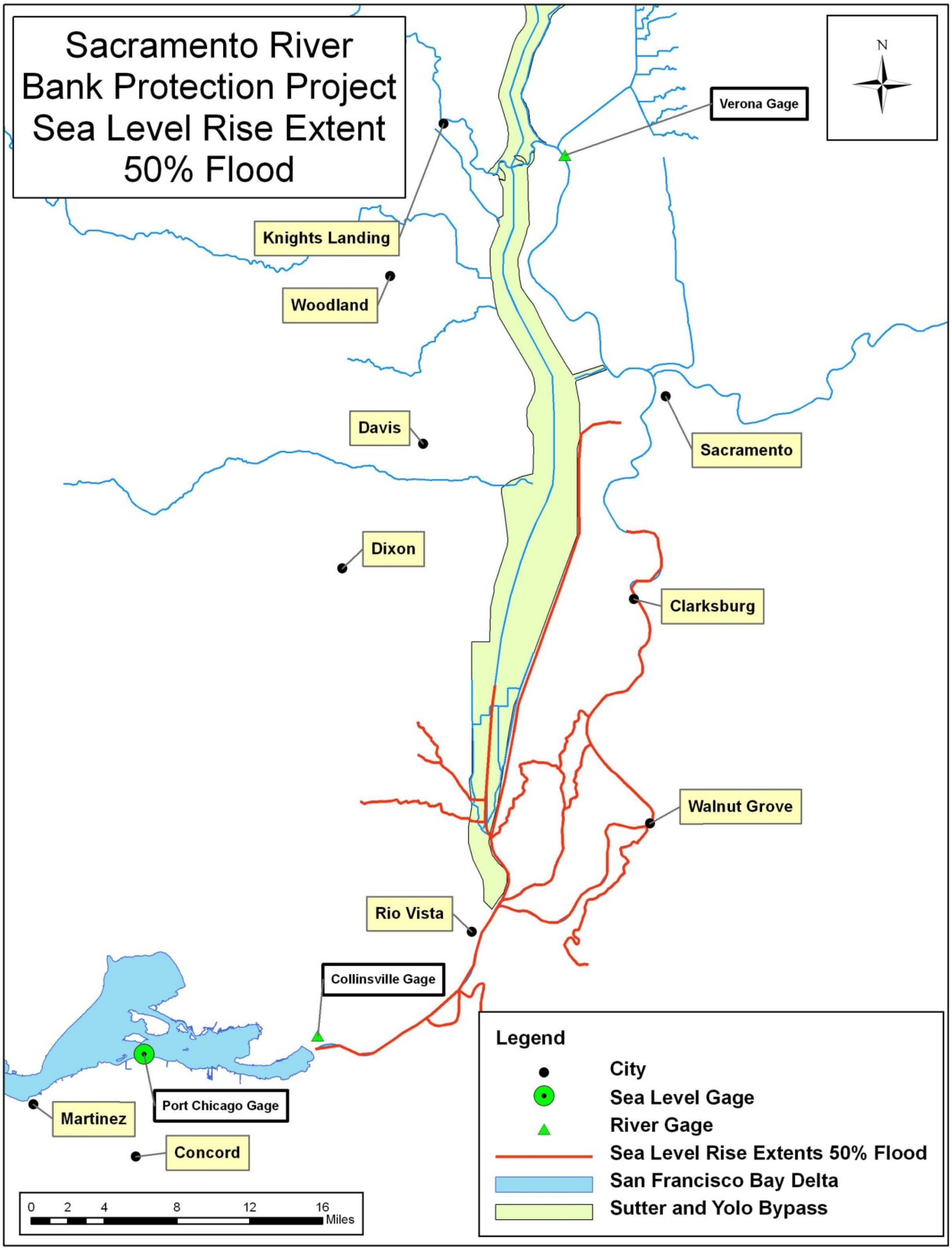


Figure 3: Maximum Estimated Extent of Sea Level Rise – 50% flood assuming 2.71 ft rise at Collinsville

A hydraulic model station is the number of miles from the downstream end of the channel in the Common Features HEC-RAS model. This is different than USGS miles but is generally close in value. It is appropriate here for relative comparison as the Yolo Bypass does not have USGS miles associated with it for its entire length. This way all reaches can be referenced using a common measuring system. This can be determined in GIS by overlaying the HEC-RAS cross-sections over aerial photos or other GIS data. Roads intersecting the river perpendicular to the channel that approximate these locations (located slightly upstream) are provided to provide an easier method to find the locations in the field.

The results from the analysis show that the 1% flood had a greater impact on the upstream water elevation than the 50% flood. The Sacramento River experienced changes greater than 0.1 ft up to USGS River Mile 50.85 (hydraulic model station 51.247), while the Yolo Bypass was affected up to the Yolo County Road 155 and 104 intersection (hydraulic model station 29.267). The entire Deep Water Ship Channel (DWSC) experienced changes greater than 0.1 ft. All other reaches downstream of the areas listed above were affected by the estimated maximum sea level rise. This includes Lindsey Slough, Cache Slough, Haas Slough, Horseshoe Bend, Three Mile Slough, Georgiana Slough, Miner Slough, Steamboat Slough, and Sutter Slough.

The initial analysis is based on the eustatic sea level rise equation for the high estimate (2.71 ft). If the changes were based on the relative sea level rise equation, the estimated maximum change in sea level is 2.79 ft (high estimate). The difference between the two values is 0.077-feet which is within the hydraulic model's range of error and is insignificant for the purpose of this analysis. The computations for the eustatic and local sea level rise are shown in Appendix 2. This was checked in the hydraulic models and the change does not significantly impact the analysis results.

To provide a conservative estimate, the estimated maximum limit of sea level change affects was increased from the model results by about 2 miles. Therefore the adjusted maximum upstream limits of sea level rise are lines of latitude drawn through Yolo County Road 152 for the Yolo Bypass and USGS River Mile 57.5 (approximately the intersection of the Deep Water Ship Channel and the Sacramento River near the city of West Sacramento). Erosion sites downstream of these locations and in the channels entirely affected by sea level rise (shown in table 1) will need to account for sea level change in site specific analysis. If the erosion site is outside this area it will not need to account for sea level change as it is not expected to be affected by sea level change over the estimated 50 year life of the project.

5. Estimating Seal Level Change Hydraulic Analysis Considerations

Site specific analysis will address potential sea level rise during implementation. This includes considering future changes to stage, velocity magnitude, velocity direction, velocity distribution, and wave characteristics. However, this will only need to be considered for the areas affected by sea level change. A reconnaissance level stone protection analysis was conducted to determine potential impacts of sea level change on velocities and wave heights affecting riprap design using HEC-RAS results, GIS, CHANLPRO Version 2.0 software, and engineering judgment.

5.1 Stone Protection Design Considerations

5.1.1 Velocity Considerations

The velocities from the high/low estimates for the 1% flood were compared against the existing conditions model to see if changes in sea level elevation would increase velocities along the reaches and impact stone protection design. Changes of less than 0.1 feet/second were considered insignificant and well within the range of model error.

After initial review of the velocity comparisons, there were significantly higher velocities on the Yolo Bypass- Egbert Tract reach when compared to other reaches. An investigation of the cross sections along the reach in HEC-RAS showed water being unrealistically confined to the main channel rather than allowed to flow in the main channel and the overbank as it really would. To align the model velocities with what would really occur, several levee heights within the reach were reduced to allow water to flow in the overbank. This resulted in more realistic velocities in the Yolo Bypass- Egbert Tract. The models were then run with the new geometry and the velocities compared.

After comparing the modified high/low sea level rise estimates to the existing conditions, a majority of the reaches either experienced a negligible change in velocity (<0.1 feet/second) or a decrease in velocity for the future sea level rise conditions. HEC-RAS model station 2.944 on the Three Mile Slough experienced the greatest increase in velocity (0.63 feet/second) for the 1% flood high estimate.

To determine if the expected maximum sea level rise (2.79 ft) could increase the size and gradation of stone protection, this site on Three-Mile Slough was analyzed assuming there is an erosion site at this location. There is not an erosion site at this location at this time. It should be noted that the purpose of this analysis is to determine if there is a relative change in the final recommended stone protection size and gradation from the CHANLPRO program. It is not intended to provide an actual design stone protection size and gradation for this or any other project site. The hydraulic variables from this point on Three-Mile Slough with the maximum change in velocity due to sea level rise were inputted into CHANLPRO (a USACE program for determining stone protection size and gradation). This is to determine if this change in velocity would impact stone protection design. The output tables from CHANLPRO for the existing project conditions and with- project conditions are shown in Appendix 3 and Appendix 4, respectively. The only difference between the two tables was the computed D_{30} (30% of the stone protection particles diameter are smaller than this value) for a stable gradation. However, this did not impact the design stone protection gradation. It is concluded from this relative comparison that velocity changes from future sea level rise should not affect stone protection design. However, there may be local 2D/3D effects that need to be considered during site specific analysis and design.

5.1.2 Wind Wave Considerations

The analysis in 5.1.1 only considers changes in stone protection design from changes in velocity due to sea level rise during the life of the project. However, waves from the wind could also impact stone protection design. Wind waves are generally a function of the fetch (the length of water over which a given wind has blown) and sometimes the depth. Changes in fetch lengths due to sea level change should be minimal in the project area, so the design of stone protection is not likely to be impacted by changes in fetch. A draft report (not certified) for designing the stone protection for repair sites along Sacramento River river miles 40 – 60 indicates depth may not be a significant factor in determining the design of stone protection (USACE 2006). This report concludes that depth is not a factor affecting wind caused wave height for the design of stone protection for this reach. It is likely that this is also applies for most or all of the area impacted by sea level change. However, wind waves need to be considered during site specific analysis and design, including potential changes in fetch and depth.

6. Conclusions

The Sacramento District of the United States Army Corps of engineers (USACE) is developing a programmatic Post Authorization Change (PAC) document and EIS/R (Environmental Impact Statement/Report) addressing the additionally authorized 80,000 linear feet of erosion control work to Phase II of erosion as per the Water Resources Development Act (WRDA) 2007. The project authorization is to reduce the risk of levee failures within the SRFCP system from erosion. EC-1165-2-211 requires all USACE coastal activity within the extent of the estimated tidal influence be considered for relative sea-level change effects. Changes in relative sea level could impact hydraulic, geotechnical, economic, real estate, and environmental analysis and considerations of the project. The report focuses on the hydraulic considerations and provides information for other disciplines to include in their analysis and documentation.

The southern portion of the SRBPP project is subject to tidal affects and the range of potential sea level rise at the downstream (southern) boundary is estimated to be between 0.42 feet (low estimate) and 2.79 feet (high estimate) between 2013 (estimated construction start) and 2075 (50 years from estimated construction end in 2025).

The high and low value estimate of potential future sea level change determined in accordance with EC-1165-2-211 was used to modify the Common Features HEC-RAS model to estimate the extent of potential sea level change within the life of the project at a programmatic scale. This analysis was conducted for the 1% (100-year) flood and 50% (2-year) flood in order to approximate a reasonable range of conditions. The 1% flood is representative of design conditions and the 50% flood is included to consider potential environmental impacts.

The analysis indicates that the high estimate of potential sea level change (2.79 feet) increases the water surface elevation by greater than 0.1-foot for the areas shown in table 1. The Yolo bypass and Sacramento River upstream limit of affects was increased by approximately 2 miles from the analysis results to provide a conservative estimate of the upstream limit of future sea level change impacts. Future erosion repair sites outside this adjusted area of potential sea level rise impacts shown below in table 2 will not need to incorporate sea level change into site specific analysis and design. Future erosion repairs within this adjusted area shown in table 2 will need to address sea level change in their site specific analysis and design.

As noted previously, the requirements of EC-1165-2-211 apply to this federal project but are different than the state of California requirements and procedures. Both procedures yield similar numbers for the high sea level rise estimate. Since the high estimate provides the maximum estimated extent of sea level rise, the differences in the procedures are not significant. In fact, the USACE procedure provides a slightly more conservative estimate of the geographic extent of sea level rise than the state guidance.

A preliminary programmatic stone protection analysis indicates that sea level change is not likely to impact the size and gradation of stone protection. However, the site specific hydraulic analysis should consider addressing future local changes to stage, velocity, and wave characteristics for these reaches affected by sea level change.

Table 2. Adjusted Areas Potentially Affected by Sea Level Change

Reach	Area Affected
Sacramento River	Downstream of USGS River Mile 57.5 (Deep Water Ship Channel and Sacramento River intersection in the city of West Sacramento) to the downstream end of the channel at the Collinsville Gage in the Delta
Yolo Bypass	Downstream of Yolo County Road 152 to the downstream end of the channel
DWSC	Entire Reach
Lindsey Slough	Entire Reach
Cache Slough	Entire Reach
Haas Slough	Entire Reach
Horseshoe Bend	Entire Reach
3 Mile Slough	Entire Reach
Georgiana Slough	Entire Reach
Miner Slough	Entire Reach
Steamboat Slough	Entire Reach
Sutter Slough	Entire Reach

7. References

Deverel, Steven, J, Leighton, and Historic, David. “Recent and Future Subsidence, Sacramento-San Joaquin Delta, California, USA.” San Francisco Estuary and Watershed Science. 2010.

Galloway, Devin, Ingebritsen, S.E, and Jones, David. “Land Subsidence in the United States – Part II Drainage of Organic Soils – Sacramento San-Joaquin Delta/Florida Everglades.” USGS Circular 1182. 1 Sep. 2005. 24 Feb. 2011 <http://pubs.usgs.gov/circ/circ1182/>

“Mean Sea Level Trend 9415144 Port Chicago, California.” NOAA. 9 Dec. 2008. 24 Feb. 2011 <http://tidesandcurrents.noaa.gov/sltrends/sltrends_station.shtml?stnid=9415144>

“Standard Operation and Maintenance Manual for the Sacramento River Flood Control Project.” USACE. May 1995.

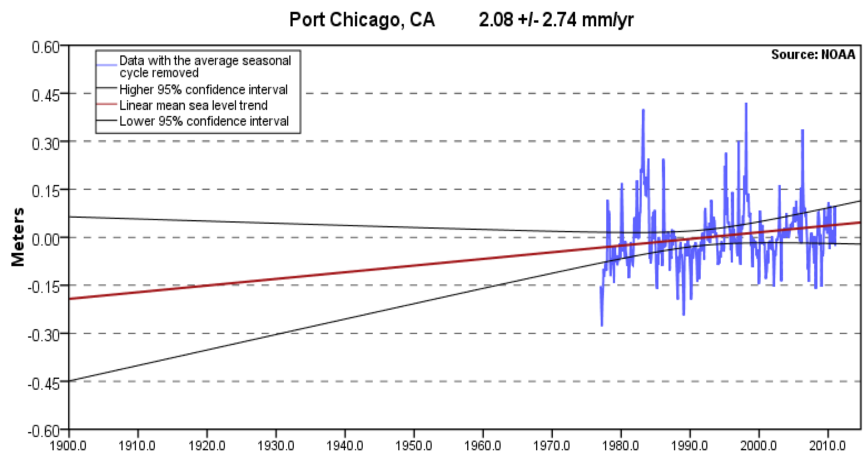
“Water Resource Policies and Authorities Incorporating Sea-Level Change Considerations in Civil Works Programs.” EC 1165-2-211. 24 Feb. 2011 < http://140.194.76.129/publications/eng circulars/ec1165-2-211/entire.pdf>

“§ 208.10 Local flood protection works; maintenance and operation of structures and facilities.” GPO Access. 28 Feb. 2011. 1 March 2011 <http://ecfr.gpoaccess.gov/cgi/t/text/text idx?c=ecfr;rgn=div5;view=text;node=33%3A3.0.1.1.4;idno=33;cc=ecfr#33:3.0.1.1.4.0.1.1>

“Sacramento River Mile 40 to 60 Rock Riprap Gradation Design for River Currents, Wind and Boat Waves.” Final Draft Report for Review. USACE 2006.

Appendix 1

Mean Sea Level Trend
9415144 Port Chicago, California



The mean sea level trend is 2.08 millimeters/year with a 95% confidence interval of +/- 2.74 mm/yr based on monthly mean sea level data from 1976 to 2006 which is equivalent to a change of 0.68 feet in 100 years.

The plot shows the monthly mean sea level without the regular seasonal fluctuations due to coastal ocean temperatures, salinities, winds, atmospheric pressures, and ocean currents. The long-term linear trend is also shown, including its 95% confidence interval. The plotted values are relative to the most recent Mean Sea Level datum established by CO-OPS. The calculated trends for all stations are available as a table in millimeters/year or a table in feet/century (0.3 meters = 1 foot).

If present, solid vertical lines indicate times of any major earthquakes in the vicinity of the station and dashed vertical lines bracket any periods of questionable data.

Source: http://tidesandcurrents.noaa.gov/sltrends/sltrends_station.shtml?stnid=9415144

Appendix 2

Low Estimate Calculation:

$$\frac{2.08\ mm}{yr} \times 62\ yrs = 128.96mm = .128m$$

$$.128m \times \frac{3.28\ ft}{1\ m} = .42\ ft$$

High Estimate Calculation (Eustatic):

$$EQ3 = E(t2) - E(t1) = .0017(t2 - t1) + \ b(t2^2 - t1^2)$$

$$t2 = 2075 - 1986 = 89yrs$$

$$t1 = 2013 - 1986 = 27yrs$$

$$b = .0001005$$

$$EQ3 = .0017(89 - 27) + \ .0001005(89^2\ -\ 27^2) = .827m$$

$$.827m\ \times \frac{3.28ft}{1\ m} = 2.71\ ft$$

High Estimate Calculation (Relative):

$$EQ3 = E(t2) - E(t1) = .00208(t2 - t1) + \ b(t2^2 - t1^2)$$

$$t2 = 2075 - 1986 = 89yrs$$

$$t1 = 2013 - 1986 = 27yrs$$

$$b = .0001005$$

$$EQ3 = .00208(89 - 27) + \ .0001005(89^2\ -\ 27^2) = .8517m$$

$$.8517m\ \times \frac{3.28ft}{1\ m} = 2.794\ ft$$

Appendix 3

3 Mile Slough W/O Project (Station 2.944)

```
PROGRAM OUTPUT FOR A NATURAL CHANNEL SIDE SLOPE RIPRAP, STRAIGHT REACH
               INPUT PARAMETERS
SPECIFIC WEIGHT OF STONE,PCF              135.0
LOCAL FLOW DEPTH,FT                         5.9
CHANNEL SIDE SLOPE,1 VER: 1.79 HORZ
AVERAGE CHANNEL VELOCITY,FPS               4.35
COMPUTED LOCAL DEPTH AVG VEL,FPS           4.35
(LOCAL VELOCITY)/(AVG CHANNEL VEL)         1.00
SIDE SLOPE CORRECTION FACTOR K1             .82
CORRECTION FOR VELOCITY PROFILE IN BEND    1.00
RIPRAP DESIGN SAFETY FACTOR                1.30
```

```
                       SELECTED STABLE GRADATIONS
                              ETL GRADATION

    NAME    COMPUTED D30(MIN)  D100(MAX)  D85/D15  N=THICKNESS/   CT
THICKNESS
             D30 FT     FT         IN                D100(MAX)           IN
     1         .14      .37       9.00       1.70       1.00      1.00
9.0

  D100(MAX)        LIMITS OF STONE WEIGHT,LB          D30(MIN)  D90(MIN)
     IN          FOR PERCENT LIGHTER BY WEIGHT           FT        FT
                   100           50            15
    9.00       30     12       9       6       4       2      .37        .53

           EQUIVALENT SPHERICAL DIAMETERS IN INCHES
  D100(MAX)  D100(MIN)  D50(MAX)  D50(MIN)  D15(MAX)  D15(MIN)
    9.0        6.6        6.0       5.3       4.8       3.6
```

Appendix 4

3 Mile Slough with Project -1% Flood High Est (Station 2.944)

```
    PROGRAM OUTPUT FOR A NATURAL CHANNEL SIDE SLOPE RIPRAP, STRAIGHT REACH
                   INPUT PARAMETERS
    SPECIFIC WEIGHT OF STONE,PCF              135.0
    LOCAL FLOW DEPTH,FT                         8.6
    CHANNEL SIDE SLOPE,1 VER: 1.79 HORZ
    AVERAGE CHANNEL VELOCITY,FPS               4.98
    COMPUTED LOCAL DEPTH AVG VEL,FPS           4.98
    (LOCAL VELOCITY)/(AVG CHANNEL VEL)         1.00
    SIDE SLOPE CORRECTION FACTOR K1             .82
    CORRECTION FOR VELOCITY PROFILE IN BEND    1.00
    RIPRAP DESIGN SAFETY FACTOR                1.30


                       SELECTED STABLE GRADATIONS
                              ETL GRADATION

    NAME    COMPUTED D30(MIN)  D100(MAX)  D85/D15  N=THICKNESS/   CT
THICKNESS
             D30 FT     FT         IN                 D100(MAX)           IN
     1         .17      .37       9.00       1.70       1.00      1.00
9.0

  D100(MAX)        LIMITS OF STONE WEIGHT,LB          D30(MIN)  D90(MIN)
     IN          FOR PERCENT LIGHTER BY WEIGHT           FT        FT
                  100           50            15
    9.00       30      12      9       6       4      2     .37       .53

           EQUIVALENT SPHERICAL DIAMETERS IN INCHES
  D100(MAX)  D100(MIN)  D50(MAX)  D50(MIN)  D15(MAX)  D15(MIN)
    9.0        6.6        6.0       5.3       4.8       3.6
```

Appendix B, History of the 1957 Profiles

History of the 1957 Profiles on the Sacramento River

Background

In the late 1800s the flood capacity of the Sacramento River and its tributaries was greatly reduced due to tailings from hydraulic mining. Hydraulic mining was officially halted in 1884 with two court cases (Woodruff v. North Bloomfield Gravel Mining Co. and People v. Gold Run Ditch and Mining Company). Levees were improperly built and rivers in the Sacramento Basin were unable to contain average year floods. It was proposed in 1880 that the state engineer take control of maintaining the drainage of the river basins, however this was never acted on by Congress. In 1894, it was suggested again that improvements to the channel of the lower Sacramento River would lower flood stages, however the construction of engineered levees on the Feather River was very important. Again, the legislature did not act on these recommendations. In 1904, another futile attempt was proposed to modify the channels of the sediment filled streams to increase slope and encourage movement of sediment from the river channel. It also proposed levees on the Yuba and Feather Rivers; however, the state did not take action. In 1905 the Rivers and Harbors Act of 1905 appointed three engineers from the Army to cooperate with the state and determine the feasibility of navigational improvements (Kochis 1963).

The California Debris Commission (CDC) was created in 1893 as part of the Rivers and Harbors Act. It was created by the Federal government and was made up of three army engineers that were appointed by the president. Minor work on debris control and Navigation were performed by the federal government prior to the creation of the CDC. In February 1900, Daguerre Point Dam was proposed on the Lower Yuba as a means to contain mining debris. The first flood control measures were first carried out in the Rivers and Harbors Act of 1910. The report is contained in House Document 81 and is from a report by the CDC. The flood control measures proposed included dredging of the Sacramento River below Cache Slough to increase channel capacity. Dredging was not performed on the Feather River even though it was included in the report (Kochis 1963).

Shortly after the 1910 flood control project approved in House Document 81 the state of California created the Reclamation Board Act of 1911. This was made up of three members appointed by the governor. The board was to examine plans for flood control and reclamation of lands in accordance with the CDC. If the Reclamation Board did not approve the plans then they could not be pursued. In 1913, the Reclamation Board's duties were more clearly defined to not include channel expansion or construction of weirs on the Sacramento River. The number of board members was also increased to seven (Kochis 1963).

In House Document 81, it suggests that the capacity of the Sacramento River at Collinsville needed to be in excess of 600000 cfs, where prior to the floods of 1907 and 1909, the capacity was recommended to be 250000 cfs. In the document, the reasoning for not simply widening the channel is articulated to be due to the need for scour flows to wash continued sediment downstream from the hydraulic mining tailings. Also, a wider channel would lower the depth of low flow events causing navigation to be an issue. As a result, the Basins surrounding the Sacramento River were investigated for reclamation. The two largest were the Sutter Basin and the Yolo Basin with 1,038,000 AF and 1,126,000 AF, respectively. Evaluation of the capacity needed in the river at various points showed that it needed a much greater capacity than was there at the time (Stimson 1911).

Table 8. Channel Capacity at locations along Sacramento River

Localities	Distance	Capacity, cfs (1911)	Capacity, cfs (required)
Chico Landing	202	235000	235000
Colusa	151	70000	250000
Knights Landing	94	25000	250000
Below Feather River	81	65000	450000
Below American River	62	80000	525000
Below Cache Slough	16	165000	600000

The bypass system was first proposed in 1894 by Marsden, Manson, and Grunsky who were consultants to the commissioner of public works. This bypass system using the reclaimed basins along with channel improvements to various reaches along the Sacramento River, Feather River, Yuba River and smaller tributaries became the foundation for the Sacramento River Flood Control Project (Stimson 1911).

1957 Profiles

The 1957 Profiles for the Sacramento River were developed in a joint effort by the United States Army Engineer Division, the State Department of Water Resources and the State Reclamation Board. The levee and channel profiles were created based on a compilation of all data available from the Sacramento District at the time (McCollam 1957). The basis for most of this data was the investigation for Senate Document No. 23 entitled “Flood Control in the Sacramento and San Joaquin Basins” printed in 1926. For reaches not included in Senate Document No. 23, the data was obtained through hydrologic analysis in order to fill the data gaps necessary to establish channel capacities for the main tributaries of the Sacramento River.

Senate Document No. 23 was the document authorizing the revisions to the Old Sacramento River Flood Control Project in 1928. Further modifications to the flood control system were made after the 1937 flood. The 1938 modifications were mainly along the Feather River because “numerous levee failures occurred along the Feather River levees between 1920 and 1934, these levees were set back and enlarged to accommodate greater flows. These changes were summarized in memorandums issued by the USACE which define the minimum freeboard requirements for each segment of the Sacramento River Flood Control Project (SRFCP), collectively referred to as the ‘USACE 1957 Profile’” (Archer 2009). Further modifications were made to the system in 1951 upstream of the Tisdale Bypass and in the Sutter Basin. The 1951 modifications were done in response to a project authorized to look into reclaiming the Butte Basin. However, the Butte Basin was never reclaimed. The Design flows were updated after the flood in 1955 to include the most current record of flows.

Data Collection

Bank and channel elevations were determined from river surveys from 1951 and levee elevations were determined from a combination of the survey, contract drawings, and from detailed final design surveys for the levees. The surveys were largely performed as part of the investigation for the memorandum of understanding (McCollam 1957).

Floodplains were constructed based on the flows and levees found in Senate Document No. 23. These were subsequently updated after each significant flooding event: 1935, 1936, 1937-38, 1940, 1942, 1950, and 1955. Field surveys and high water marks were obtained for these events and discharge was studied at key river stations.

Profiles

Drawings of the 1957 profiles were created with this information. The drawings are divided up into key stream systems. Each stream system is composed of several reaches, if present. Above the reach, the channel design flow is shown and the extent of the stream where it applies. The vertical datum for the profiles is the United States Engineers Datum (USED). This is different from NAVD88 and NGVD29. Conversions from the USED are an ongoing issue but some values have been suggested for USED to NGVD29 (~+3ft).

Limitations

There have been a number of changes to the Sacramento River since the 1957 Profiles were created. The major one is that the channel has migrated and the river miles described in the 1957 profiles are not the same as those from the Comprehensive Study. There is also a question of whether subsidence has played a role in the elevations of the current stream and the bypasses as the Sutter and Yolo County areas have significant subsidence in certain areas. In the 1957 profile, there is mention of the Butte Basin and its design capacity. However, the Butte Basin was never reclaimed for use as a bypass. Also, the 1957 Profiles predate Oroville. The profiles are not based on frequency as Senate document No. 23 did not account for frequency and only mentioned the design capacity based on a revised high flow event at the Collinsville Gage. Also, the 1957 profiles did not use Manning's equation that is the basis for much of today's hydraulic analysis. In addition, hydraulic modeling has improved dramatically since the 1957 profiles were developed. However, it should be noted that the 1957 profiles are based on observed high water marks for actual large flood events.

Current Efforts

There is an ongoing effort to find more complete documentation of the 1957/1955 profiles but since it is not officially tied to any projects, the funding to do such searches through the archives has not been warranted. The information here should not be considered complete, however is useful as a general background to the 1957 profiles and what lead up to them.

References

Archer, M. Summary Report on Hydraulic Impact Analysis. National Levee Improvement Program. Sacramento Area Flood Control Agency. MBK Engineers, Revised April 6, 2009.

Kochis, Frank. History of Development of the Sacramento River Flood Control System. July 29 1963.

McCollam, Colonel A. E. Submittal to Division Engineer. Levee and Channel Profiles, Sacramento River Flood Control Project. 1 July 1957.

Schwab, Delbert. Water Measurement Units and Conversion Factors. Oklahoma State University. Division of Agricultural Sciences and Natural Resources. May 19 2011. http://pods.dasnr.okstate.edu/docushare/dsweb/Get/Document-2216/BAE-1501web.pdf

Stimson, H. L. Secretary of War. Flood Control – Sacramento and San Joaquin River Systems, California. Letter to the 62nd Congress 1st session House of Representatives Document No. 81. June 27, 1911.

Sub-Appendix A6. Real Estate Maps

(Provided separately due to file size)

Sub-Appendix A7. Operation and Maintenance Manual Template

Operations and Maintenance Manual Template

SACRAMENTO RIVER BANK PROTECTION PROJECT PHASE II, 80,000 LINEAR FEET

Prepared for:
U.S. Army Corps of Engineers
Sacramento District

January 25, 2012
(Updated November 2018)

Prepared by:
HDR Engineering, Inc.

2365 Iron Point Road, Suite 300
Folsom, CA 95630

This page intentionally left blank.

Contents

Figures

Tables

This page intentionally left blank.

1.0 Introduction

1.1 Purpose

This information report and template is an appendix to the Post-Authorization Change Report (PACR) for the Sacramento River Bank Protection Project (SRBPP). This template provides information on revising and adding new material to the existing operations and maintenance (O&M) manuals to take into account new bank protection along the Sacramento River Flood Control Project (SRFCP) that is constructed under the SRBPP authority.

1.2 Use of Operation and Maintenance Manual Template

This template has been developed to be used as guidance for future SRBPP O&M manual preparers and provides general information to Corps of Engineers (Corps) personnel and local interests. This manual template is meant to be used as a general guide; it may be considered a template that can facilitate O&M manual preparation. The use of this as a template will encourage preserving consistency among the O&M manuals as they are individually revised. Future O&M manual revisions and additions will occur as bank protection becomes known and is constructed.

1.3 Background

Operation and maintenance manuals are to inform local interests of the O&M requirements of levees and other flood risk management facilities. Engineering Regulation ER 1110-2-401 (Operation, Maintenance, Repair, Replacement, and Rehabilitation Manual for Projects and Separable Elements Managed by Project Sponsors) provides for the preparation of O&M manuals. Manuals are described as follows in the regulation:

- The purpose of the O&M manual is to assist the responsible authorities in carrying out their obligations through provision of information and advice with respect to the operation and maintenance requirements of the project.

- Manuals will be prepared sufficiently in advance of completion of the project to ensure their readiness for transmission to local interest at the time of formal transfer of the project from the Corps to the non-Federal sponsor.

Since bank protection constitutes modifications to the SRFCP, it has been the practice to modify and amend the SRFCP Supplement to Standard O&M manuals. This practice avoids redundancy and confusion, and is expected to continue with the SRBPP Phase II 80,000 LF as well.

2.0 Sacramento River Flood Control Project Manuals

2.1 SRFCP O&M Manual organization

There is one overarching manual for the SRFCP, and a series of manuals covering levee unit specific details. The overarching manual is the Standard Operations and Maintenance Manual, referred to as the “Standard Manual.” The Standard Manual is dated

May 1955. It has an addendum dated April 1995 and a supplement to the addendum dated March 1996.

For each levee unit (defined below) a Supplement to Standard Operatio~~n~~s and Maintenance Manual (referred to herein as “Supplement Manual”) is prepared. **Figure 1** shows an example cover page of a Supplement Manual. When the Sacramento River Bank Protection Project constructs bank protection, amendments and revisions are made to the Supplement Manual that covers the specific levee unit in which the constructed bank protection is located. No amendment is made to the Standard Manual.

The SRFCP is subdivided into maintenance units generally corresponding to levees associated with a protected floodplain or reclaimed land. The units are numbered starting from Unit 101, Sherman Island levees, near the mouth of the Sacramento River. The units are numbered sequentially, generally south to north, up to Unit 165. The units cover both the Sacramento River and its tributaries and distributaries. **Figure 2** is a map showing the levee maintenance units locations along the SRFCP.

Figure 1 - Cover to Sample Supplemental Manual

Book E

SUPPLEMENT TO STANDARD OPERATION AND MAINTENANCE MANUAL

SACRAMENTO RIVER
FLOOD CONTROL PROJECT

UNIT NO 115

EAST LEVEE OF SACRAMENTO RIVER

FROM

SUTTERVILLE ROAD TO

NORTH BOUNDARY OF RECL. DIST. NO. 744

SACRAMENTO DISTRICT

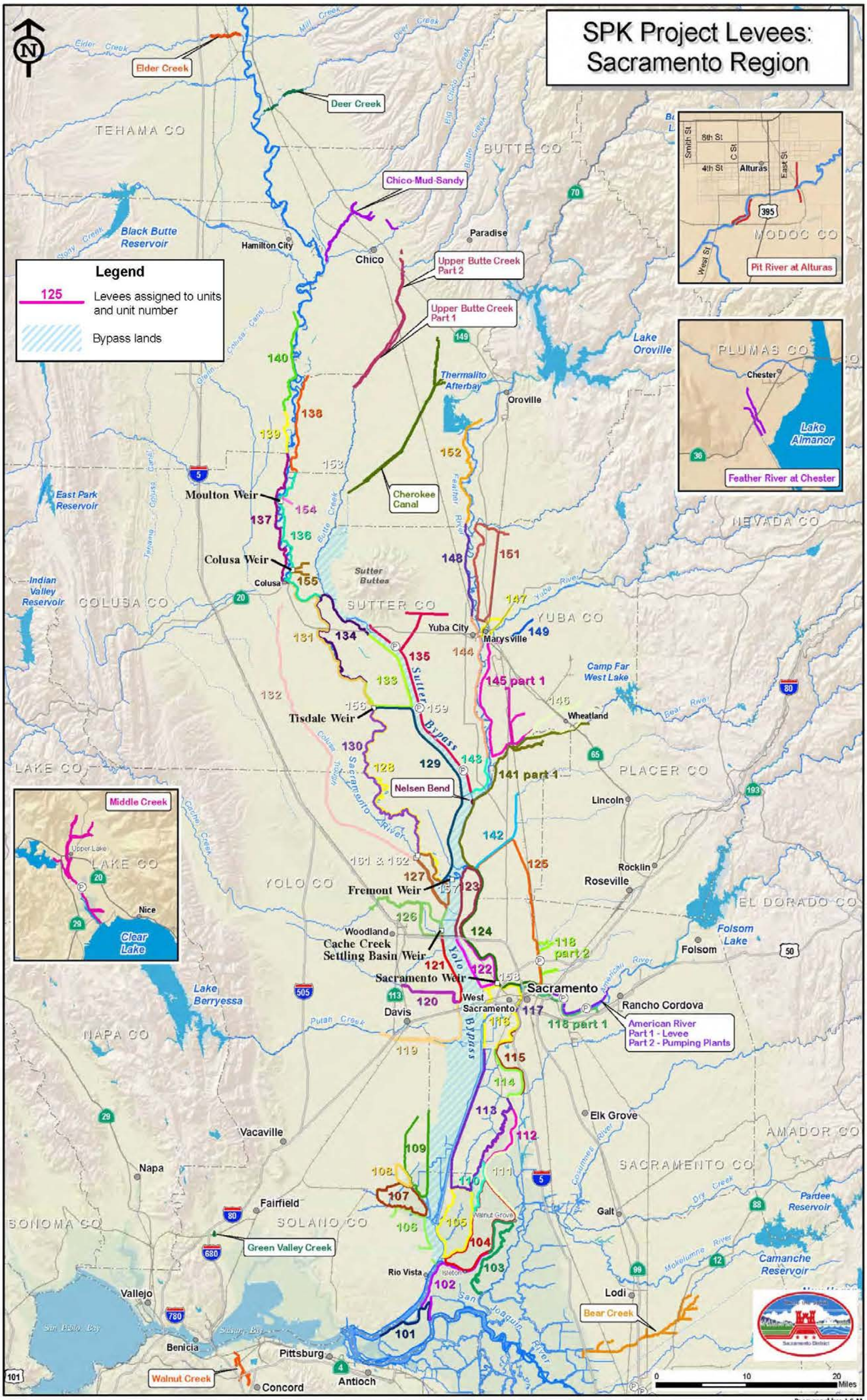
CORPS OF ENGINEERS

U. S. ARMY

SACRAMENTO, CALIFORNIA

This page intentionally left blank.

Figure 2 - SRFP Levee Units Map



This page intentionally left blank.

3.0 Amendments and Revisions

3.1 Typical Amendments and Revisions

Changes to the Supplemental Manuals are completed to document new bank protection features, including engineered structures and vegetation plantings. In most cases the addition of bank protection will not change the O&M requirements. Addition or changes to O&M requirements may be appropriate.

The following is a list of new information that is typically added to Supplemental Manuals when bank protection is constructed:

- Location and extent of construction, including left or right bank, river miles, and latitude and longitude coordinates
- References to “as constructed” or “as-built” drawings, including drawing file numbers
- Construction contract information, including contractor and contract number
- Pertinent correspondence, including formal project transfer and project acceptance
- Environmental mitigation description and location
- Excerpt of mitigation requirements and citation of source (e.g. Environmental Impact Statement, or Biological Opinion)
- Reference to cultural resources recovery information and identification of cultural resource sites (if it is determined, on a case-by-case basis, that there is need for O&M personnel to be aware of cultural resource sites)
- Care and management of mitigation vegetation and in-stream woody material (IWM) that differs from established instructions
- Project Partnership Agreements and Amendments
- Revised map of levee if there is a change to levee alignment
- Real Estate Easement Map showing project-specific easements
- Changes to non-project features in connection with bank protection (e.g. utility relocations, recreation facilities)

3.2 Real Estate Acquisitions and Permits

Often additional rights of way and easements are acquired for construction of bank protection and for mitigation. Existing easements may be modified and/or new easements/property rights may be acquired to accommodate the new bank protection requirements. For instance, flood control easements may be revised to include management and preservation of riparian vegetation. Supplement manuals inform levee maintainers of these changes. Temporary construction easements (TCE) should not be reported, as these are terminated after a period of time; however, if said TCEs are required for an extended period of time due to mitigation establishment requirements, the TCE termination date and requirements should be noted. Lands and easements are the purview of the State of California. Permits are managed by the non-Federal sponsor, the Central Valley Flood Protection Board. Contact information should be referenced in case there is a need for further information on real estate acquisitions and permits.

Negotiated settlements with adjoining property owners and/or utility companies may involve new encroachment permits, joint use agreements, etc., therefore, reference to these new requirements and property rights can be included in the supplement~~al~~ manual.

Environmental mitigation banks are often used as off-site mitigation. It is generally not the duty of levee maintainers to inspect or otherwise contact mitigation banks; therefore, there generally is no practical need to provide contact information for the off-site mitigation banks.

4.0 Preparation, Review and Approval

Authority for approval of O&M manuals is delegated from the South Pacific Division Engineer to the Sacramento District. Delegation of this authority is covered in a memorandum dated June 18, 2010, subject: Delegation of Approval Authority for Operation, Maintenance, Repair, Replacement, and Rehabilitation (OMRR&R) Manuals.

The levee O&M manuals are maintained within Construction-Operations Division, Sacramento District. Revisions to the Supplement Manual are the responsibility of the SRBPP PDT and are typically assigned to Civil Design Branch, Engineering Division, in the Sacramento District.

5.0 Supplemental Manual Template

The organization of the content within each Supplement Manual is generally standardized. When bank protection projects are constructed, changes and additions to sections of manuals should be done with care to preserve organization, consistency with the flood control regulations, and readability. Changes should be noted on the Additions/Revisions table located at the beginning of the manual.

Table 1 is a general outline of Supplement Manuals and serves as a template to guide the PDT on where additions and revisions are to be placed within the Supplement Manual. The table follows the organization of a typical Supplement Manual. Annotations are highlighted in *grey italics* that show where information regarding bank protection should (or could) be added. If there is no highlighted annotation, this template does not anticipate a need for additions or revisions due to typical SRBPP construction. This does not preclude changes and additions to other sections if later found to be useful and appropriate.

The Supplement Manual Template, Table 1, is based on the Supplement Manual to Unit Number 115, East Levee of Sacramento River from Sutterville Road to North Boundary of Reclamation District Number 744. However, a Section 2-06, Real Estate, was added to accommodate changes to rights of way, etc.

Table 1 - Supplement Manual Template

LISTING	ITEM	NOTES, Bank Protection Revisions-Addendums
	Cover	
	Additions / Revisions Log	• Table noting revisions to the Supplemental Manual. • *Revisions & additions due to bank protection are included.*
	Table of Contents	
	Section I - Introduction	
1-1	Location	
1-2	Project Works	• Extent of levee, identification of other major SRFCP features. • *Bank Protection authority, bank protection works are listed.*
1-3	Protection Provided	• Brief description of flood plain behind levee, design flow
1-4	Construction Data and Contractor	• Listing of history of levee work, including description, location, contract number, reference to contract drawings. • *Bank protection construction should be added here.*
1-5	Flood Flows	• Definition of floodflows, for the purposes of the manual. Example is high water at a specified elevation and location.
1-6	Assurances Provided by Local Interests	• May be state legislation citation or other source of assurance.
1-7	Acceptance by Central Valley Flood Protection Board	• Correspondence that formally accepts the project is cited. • *Acceptance of a bank protection project is noted here. Copies of correspondence are included in Exhibit F,* Letter of Acceptance by Central Valley Flood Protection Board
1-8	Superintendent	• Name and address of local levee superintendent that the Corps may contact.
	Section II – Features of the Project Subject to Flood Control Regulations	
2-1	Levees	• Brief description, reference to O&M requirements and special instructions in the Standard Manual, reference to check list in the Supplemental Manual. • *If bank protection results in a major modification such as a setback levee or adjacent levee, the levee description should be edited as appropriate.*
2-2	Drainage and Irrigation Structures	• List of pipes and other structures that extend through the levee, and references to drawings. • *Revise if pipes or structures are relocated due to construction.*
2-3	Channel	• Description of the channel (e.g. Sacramento River) • References to O&M requirements check lists in the Standard and Supplemental Manuals. • Patrolling and other operational requirements during times of flood flow.
2-4	Miscellaneous Facilities	• Detailed instructions for inspections, O&M of levee cutoff walls, observation wells. • References to O&M requirements in the Standard Manual. • List of bridges, utilities, local drainage, and recreation facilities owned & operated by other entities. • *This section would need to be revised if the bank protection results in changes to facilities.*

2-5	Environmental Protection	• Preservation/removal/replacement of live or fallen trees & vegetation. • Disposition of in-stream woody material. • Identification of mitigation areas. • Citation of sources of mitigation requirements (e.g. Biological Opinion, Environmental Impact Statement). • *On-site or near-site bank protection mitigation should be identified and special O&M requirements, if any, described. Mitigation banks need not be included.*
2-06	*Real Estate*	• *Lands acquisitions, changes to rights of way, modified and/or new easements that are connected with the bank protection* • *Changes to encroachment permits connected with bank protection* • *Temporary construction easements with extended termination dates that are required for bank protection*
	Section III – Repair of Damage to Project Works and Suggested Methods of Combating Flood Conditions	
3-1	Repair of Damage	• First responder procedure In the event of serious damage to public works.
3-2	Applicable Methods of Combating Floods	• Reference to the Standard Manual.
Exhibit A – Flood Control Regulations		• Reference to the Standard Manual.
Exhibit A1 – Location Drawing		• Map of the alignment and extend of the levee that is in the O&M unit. Reclamation District boundaries, towns, bridges, and major roads are shown. • *In the case of a setback or adjacent levee, the map is revised.* • *For fix in place bank protection the map need not be revised, but possibly the location of new bank protection could be noted.*
Exhibit B – "As Constructed" Drawings		• Drawings are listed by file number and title. • *Add bank protection drawings including file number, to this list.*
Exhibit C – Plates of Suggested Flood Fighting Methods		• Reference to the Standard Manual.
Exhibit D – Check List No. 1 – Levee Inspection Report		• Reference to the Standard Manual.
Exhibit E – Check Lists, Channels and Structures		• Check list forms for inspections of facilities, with instructions.
Exhibit F – Letter of Acceptance by Central Valley Flood Protection Board		• Correspondence on notification of project completions, emergency repairs, acceptance by sponsor, sponsor requests. • *Correspondence on formal notice of bank protection project completion by the Corps and acceptance by the sponsor are to be included here.*
Exhibit G – Semi Annual Report Form		• Sample Semi Annual Report Form.
Exhibit H – Local Cooperation Agreement		• Local cooperation agreements, declarations of financial capability. • *For bank protection construction, add local cooperation agreements, project partnership agreements, declarations of financial capability, and associated pertinent correspondence.*