

APPENDIX C
ATTACHMENT C

GEO TECHNICAL REPORT

American River Watershed

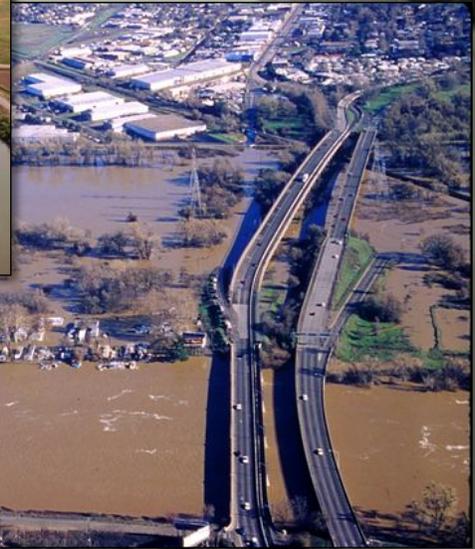
Common Features

General Reevaluation Report

Attachment C

Draft Geotechnical Report

February 2015



Sacramento
Area Flood
Control
Agency



US Army Corps
of Engineers®
Sacramento District

*Cover Photos courtesy of the Sacramento District:
Sacramento Weir during operation
Sacramento River facing south near the Pocket and Little Pocket neighborhoods
High flows on the American River at the Highway 160 overcrossing
Folsom Dam releasing high flows*

**AMERICAN RIVER, CALIFORNIA
COMMON FEATURES PROJECT
GENERAL REEVALUATION REPORT**

**Attachment C
Draft Geotechnical Report**

**U.S. Army Corps of Engineers
Sacramento District**

February 2015

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ABBREVIATIONS

ACN	Arcade Creek North
ACS	Arcade Creek South
ARCF	American River Common Features
ARFCD	American River Flood Control District
ARFCP	American River Flood Control Project
ARN	American River North Basin
ARS	American River South Basin
ASTM	American Society of Testing and Materials
ARWI	American River Watershed Investigation
BGS	Below Ground Surface
BTA	Blanket Theory Analysis
CB	Cement-Bentonite
CFS	Cubic Feet Per Second
CHP	California Highway Patrol
COS	City of Sacramento
CPT	Cone Penetrometer Test
CW	Cutoff Wall
CVFPB	Central Valley Flood Protection Board
CVFPP	Central Valley Flood Protection Plan
CY	Cubic Yard(s)
DBH	Diameter at Breast Height
DCN	Dry Creek North
DCS	Dry Creek South
DMM	Deep Mix Method
DSM	Deep Soil Mixing
DWR	Department of Water Resources
DWSC	Deep Water Ship Channel
EFA	Erosion Function Apparatus
EIP	Early Implementation Project
EM	Engineering Manual
ETL	Engineering Technical Letter
EVS	Environmental Visualization System
FEMA	Federal Emergency Management Agency
FOS	Factor(s) Of Safety
FOSM	First Order Second Moment
FT	Foot/Feet
FT/S	Feet Per Second
GER	Geotechnical Engineering Report
GIS	Geographical Information System
GMS	Groundwater Modeling Software
GRR	General Reevaluation Report
H:V	Horizontal To Vertical Ratio
HTRW	Hazardous, Toxic, and Radioactive Waste
HQUSACE	Headquarters U.S. Army Corps Of Engineers
IBC	International Building Code

IWM	In-Stream Woody Material
JET	Jet Erosion Test
K	Coefficient Of Permeability
K_H	Horizontal Hydraulic Conductivity Under Fully Saturated Conditions
K_H/K_V	Ratio Between The Vertical And Horizontal Conductivities; Anisotropic Ratio
K_V	Vertical Hydraulic Conductivity Under Fully Saturated Conditions
K_y	Yield Acceleration
LAR	Lower American River
LiDAR	Light Detection and Ranging
LM	Levee Mile
MA	Maintenance Area
MCDC	Magpie Creek Diversion Canal
MCY	Million Cubic Yards
MSWL	Mean Summer Water Level
MUSYM	Map Unit Symbol
Mw	Moment Magnitude
NAD83	North American Datum of 1983
NAFCI	Natomas Area Flood Control Improvement
NALP	North Area Levee Project
NAT	Natomas Basin
NAVD88	North American Vertical Datum of 1988
NCC	Natomas Cross Canal
NCEER	National Center for Earthquake Engineering Research
NCMWC	Natomas Central Mutual Water Company
NCSS	National Cooperative Soil Survey
NGA	Next Generation Attenuation
NGVD29	National Geodetic Vertical Datum of 1929
NEMDC	Natomas East Main Drainage Canal
NLD	National Levee Database
NLIP	Natomas Levee Improvement Project
NPACR	Natomas Post Authorization Change Report
NRCS	National Resources Conservation Service
NSF	National Science Foundation
NULE	Nonurban Levee Evaluations
OCR	Over-Consolidation Ratio
PACR	Post Authorization Change Report
PDT	Project Delivery Team
PED	Pre-Construction Engineering and Design
PGA	Peak Ground Acceleration
PGCC	Pleasant Grove Creek Canal
PGL	Policy Guidance Letter
Pr(f)	Probability of Failure
PSHA	Probabilistic Seismic Hazard Analysis
P1GDR	Phase 1 Geotechnical Data Report
P1GER	Phase 1 Geotechnical Engineering Report
QA	Quality Assurance

QC	Quality Control
RD	Reclamation District
RM	River Mile
SAFCA	Sacramento Area Flood Control Agency
SB	Soil-Bentonite
SCB	Soil-Cement-Bentonite
SGDR	Supplemental Geotechnical Data Report
SOP	Standard Operating Procedure
SPT	Standard Penetration Test
SRBPP	Sacramento River Bank Protection Project
SRFCP	Sacramento River Flood Control Project
SRN	Sacramento River North
SRS	Sacramento River South
SSURGO	Soil Survey Geographic Database
SUALRP	Sacramento Urban Area Levee Reconstruction Project
SWIF	System-Wide Improvement Framework Policy
TEC	Topographic Engineering Center
TM	Technical Memorandum
TRM	Technical Review Memorandum
ULE	Urban Levee Evaluations
USACE	U.S. Army Corps Of Engineers
USDA	United States Department of Agriculture
USGS	United States Geological Society
V:H	Vertical To Horizontal Ratio
\bar{V}_{S30}	Velocity Of The Upper 30 Meters
VVR	Vegetation Variance Request
WRDA	Water Resources Development Act
WSAFCA	West Sacramento Area Flood Control Agency
WSE	water surface elevation

1.0 INTRODUCTION

The following briefly outlines pertinent geotechnical information regarding a General Reevaluation Report (GRR) for the American River Common Features (ARCF) Project. This Report presents the results of geotechnical analyses and feasibility level geotechnical design recommendations to address levee height, geometry, erosion, access, vegetation, seepage, and slope stability deficiencies within the ARCF GRR study area.

The project area includes portions of the Sacramento and American River Watersheds. The flood plain includes most of the developed portions of the City of Sacramento, the Natomas basin, and portions of Sacramento and Sutter Counties. The study area also includes other flood facilities, including the Fremont and Sacramento Weirs and Yolo Bypass. A Post Authorization Change Report (PACR) for the Natomas portion of the Common Features project and a Chief's Report for the Natomas Post Authorization Change Report (NPACR) were completed in December 2010. The remaining portion of the project, including potential Natomas Basin levee raises, is being addressed in this report.

The ARCF GRR study area, shown in Figure 1-1, has been divided into three basins; Natomas, American River North, and American River South, which were further subdivided into study reaches. This report covers the following areas:

- About 12 miles of the north and south banks of the American River
- About 33 miles of the east bank of the Sacramento River
- About 5 miles of the south bank of the Natomas Cross Canal (NCC)
- About 3 miles of the Pleasant Grove Creek Canal (PGCC)
- About 26 miles of the Natomas East Main Drainage Canal and tributaries (NEMDC)

The ARCF GRR study area has been divided into three basins; Natomas, American River North, and American River South, which were further subdivided into study reaches. For the purposes of the feasibility planning process, the three study area basins were further subdivided into reaches based on common properties (both technical and non technical), such as geographic and geomorphic features. The levees in the American River North Basin have been divided into nine planning reaches; ARN A, B, C, D, E, F, G, H, and I. The levees in the American River South Basin have been divided into seven planning reaches; ARS A, B, C, D, E, F, and G. The levees in the Natomas Basin have been divided into nine planning reaches; NAT A, B, C, D, E, F, G, H, and I.

The ARCF GRR is evaluating federal interest in alternatives to reduce flood risk in the study area. The ARCF GRR has identified several technical deficiencies associated with the flood risk management system protecting the study area. There are various alternatives under consideration to address these deficiencies and the geotechnical components of those alternatives are discussed and or evaluated in this report. The alternatives consist of a combination of structural measures to mitigate seepage and slope stability, erosion protection, the widening and construction of new levees on the Sacramento Bypass and Weir, or a Diversion Structure on the Sacramento River.

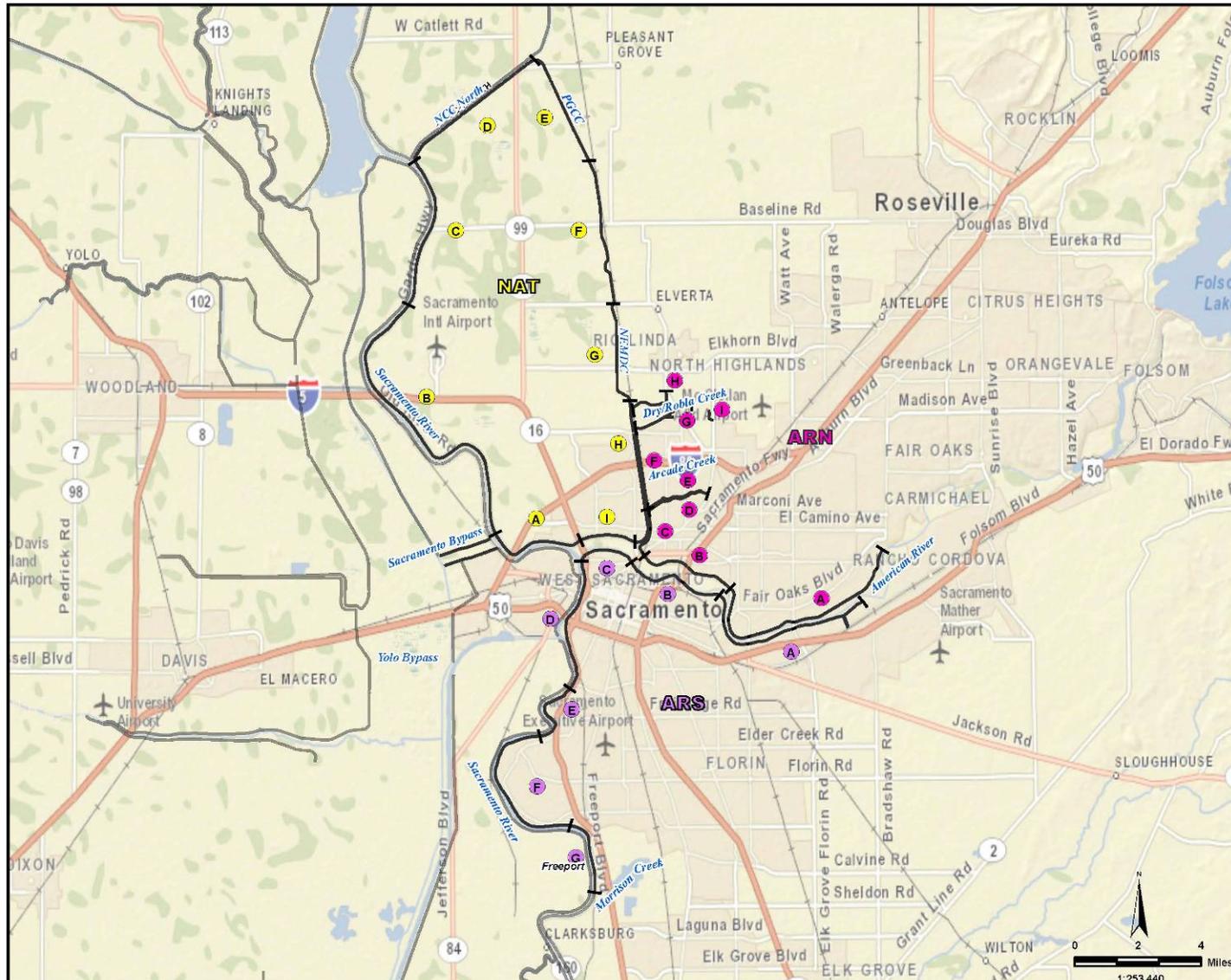


Figure 1-1: Study Area Map of the American River Common Features Project

2.0 GEOLOGY AND GEOMORPHOLOGY

2.1 GEOLOGIC SETTING

The ARCF GRR study area lies in the central portion of the Sacramento Valley which lies in the northern portion of the Great Valley Geomorphic Province of California. The Sacramento Valley lies between the northern Coast Ranges to the west and the northern Sierra Nevada to the east, and has been a depositional basin throughout most of the late Mesozoic and Cenozoic time. A large accumulation of sediments, estimated over two vertical miles in thickness in the Sacramento area, were deposited during cyclic transgressions and regressions of a shallow sea that once inundated the valley. This thick sequence of clastic sedimentary rock units was derived from adjoining easterly highlands erosion during the Late Jurassic period with interspersed Tertiary volcanics. They form bedrock units now buried in mid-basin valley areas. These bedrock units were covered by coalescing alluvial fans during Pliocene-Pleistocene periods by major ancestral west-flowing Sacramento Valley rivers (Feather, Yuba, Bear, and American). These rivers funneled large volumes of sediment into the Sacramento basin. Late Pleistocene and Holocene (Recent) alluvial deposits now cover low-lying areas. These deposits consist largely of reworked fan and stream materials deposited by meandering rivers prior to construction of existing flood control systems.

The Sacramento River is the main drainage feature of the region flowing generally southward from the Klamath Mountains to its discharge point into the Suisun Bay in the San Francisco Bay area. Located in central northern California, the Sacramento River is the largest river system and basin in the state. The 27,000 square mile Sacramento River Basin includes the eastern slopes of the Coast Ranges, Mount Shasta, and the western slopes of the southernmost region of the Cascades and the northern portion of the Sierra Nevada. The Sacramento River, stretching from the Oregon border to the Bay-Delta, carries 31% of the state's total runoff water. Primary tributaries to the Sacramento River include the Pit, McCloud, Feather, and American Rivers. Within the Sacramento area, the Sacramento and American Rivers have been confined by man-made levees since the turn of the century. The confluence with the Sacramento River, only 20 feet above sea level, is subject to tidal fluctuation although more than 100 miles north of the Golden Gate and San Francisco Bay. Within the study area, these levees were generally constructed on Holocene age alluvial and fluvial sediments deposited by the current and historical Sacramento River and its tributaries. Pleistocene deposits underlie the Holocene deposits.

2.2 GEOMORPHOLOGY

Prior to the late Pleistocene (10,000 to 30,000 years ago), the Sacramento River Basin depositional environment was influenced by a lowered base level due to sea levels as low as 400 feet below present (Harden 1998). These lowered global sea levels would have had their greatest influence in present coastal areas such as the San Francisco Bay area, but based on interpretation of the depth to denser, coarser Pleistocene soils it is estimated that average river levels in this area could have been 50 to 60 feet below current levels. The rivers would have been characterized by high energy flow with greater downward erosion rather than deposition, and would have had greater capacity to carry and deposit sand and gravel deposits into the project

area. This older geomorphology is largely covered by the more recent (Holocene) sediments in the project area. The thick zone of materials deposited above the dense, older Pleistocene alluvial deposits are therefore less than 10,000 to 30,000 years old, which is reflected in these deposits consisting of very soft to firm clays and silts and abundant loose to medium dense sands.

The filling of the Sacramento Valley with sediments following the rise in sea level to the current level has significantly reduced the gradient of the rivers flowing down from the Sierra Nevada and Klamath Mountains (including the Sacramento and American Rivers). This gradient reduction has caused the energy of these rivers to transition from erosional to graded. Graded rivers are characterized by downward erosion that is less dominant and more directed toward side-to-side movements than down-cutting. The lateral energy of a graded river causes synchronous erosion and deposition in sweeping bands commonly referred to as meanders. The outside of the meander is a zone of erosion. Material removed by the river at this zone is then deposited downstream as point bars in zones of decreased velocity on the inside of the subsequent meanders. In this way, the river migrates laterally across the flood plain. Often this erosion is slowed where the river encounters more resistant materials in the flood plain. This allows the next closest upstream meander to catch up and gradually erode away the “neck” between the two meanders. Flooding often accelerates this process as the higher energy flows can more easily cut a new thalweg (base of the active channel). The result of the conjoining meanders is the straightening of the river across the opening of the neck and the creation of an abandoned bend in the river, commonly referred to as an oxbow lake.

2.3 HYDRAULIC MINING

Hydraulic mining activity in the Sierra Nevada during the mid- to late-1800s supplied a substantial amount of sediment to many river channels draining the Sierra Nevada, which resulted in aggradation of the channels and flooding due to decrease in channel cross section area. Gold dredging and mining operations have destroyed some fluvial deposits and surfaces, confounding the understanding of the long-term geomorphic history.

This phenomenon, coupled with a disastrous flood in 1862, prompted the channelization of the Sacramento and American Rivers and re-alignment of the American River to its present-day configuration, from the former confluence with the Sacramento River to about two miles upstream. It was hoped that these actions would provide flood control as well as stimulate the flushing of accumulated mining-derived sediment from the channel.

3.0 CONSTRUCTION HISTORY

A mix of Federal, State, and local agencies have been involved in flood control project construction and operation since levees were first constructed in California in the mid-1800's. Since the creation of the State Reclamation Board (now the Central Valley Flood Protection Board or CVFPB) in 1911 and the authorization of the Sacramento River Flood Control Project (SRFCP) in 1917, most levee improvements have been first Federally authorized by Congress, then subsequently authorized by the State Legislature.

The SRFCP was authorized by the Flood Control Act of 1917 as modified by the Acts of 1928, 1937, 1941 and 1950. Features of the SRFCP, in the study area, consisted of levees along the lower American River, NEMDC, Arcade and Dry (Linda) Creeks, PGCC, NCC, and Sacramento River improvements, including new and reconstructed levees. The American River Flood Control Project (ARFCP), authorized by the Flood Control Act of 1954 (PL 83-780), consisted of a levee along the north bank of the river, extending from the terminus of the SRFCP project levee near Cal Expo upstream about 8 miles to Carmichael Bluffs. The levee along the American River up to Mayhew Canal was enlarged in 1948. Bank protection was installed along the levee in 1951. The SRFCP and ARFCP were completed by 1958.

The levees along the American River were likely originally constructed under the SRFCP likely using clamshell dredges with material sourced from the channel. The levees were then reconstructed on the left bank between 16th Street and Mayhew and on the north bank between NEMDC and Cal Expo with materials sourced from waterside borrow pits using scrapers, dozers, and compactors between 1947 and 1957. Waterside borrow material was used on the right bank of the American River from Carmichael Bluffs downstream to the upper end of the existing flood protection system near Cal Expo under the ARFCP in 1958.

The levees along the Sacramento River south of the confluence with the American River were constructed by local interests using clamshell dredges excavating material from the Sacramento River in the early 1900's. This method of construction usually resulted in loose, sandy fill material that is deepest below the center of the levee. The current materials within the levee embankment are predominantly sands, silty sands, and cohesionless materials. A setback levee at the Edwards Break and riverbank protection was constructed in 1937, in 1939 the Edwards Break levee slope had been rebuilt. In 1941 the levee was enlarged in the vicinity of Richfield Oil Co. Numerous riverbank and levee waterside slope protection were constructed along the Sacramento east bank levee.



Figure 3-1: Dredge Neptune at RM 57.3 in 1942

The levees protecting the Natomas Basin were constructed by a private mining and dredging company with the purpose of reclaiming and selling thousands of acres of farmland. The levees were constructed using large “clam shell” dredging machines. The work began in 1912 and was completed by the end of 1915. Based on typical construction schematics shown on basin-wide maps and historical literature, the levees along the Sacramento River were constructed in the following manner:

- A dragline was used to excavate a trench about 6 to 12 feet deep along the centerline of the levee alignment. The trench bottom width ranged from about 12 to 28 feet. The excavated material was deposited along both sides of the trench forming two small containment dikes.
- Hydraulic dredging operations placed material from the adjacent Sacramento River bottom into the excavation area between the dikes. This material consisted predominately of sands.
- The final levee configuration was achieved by covering the dredged sand with the adjacent dike materials. These materials consisted predominately of silt, clay, and fine sand.

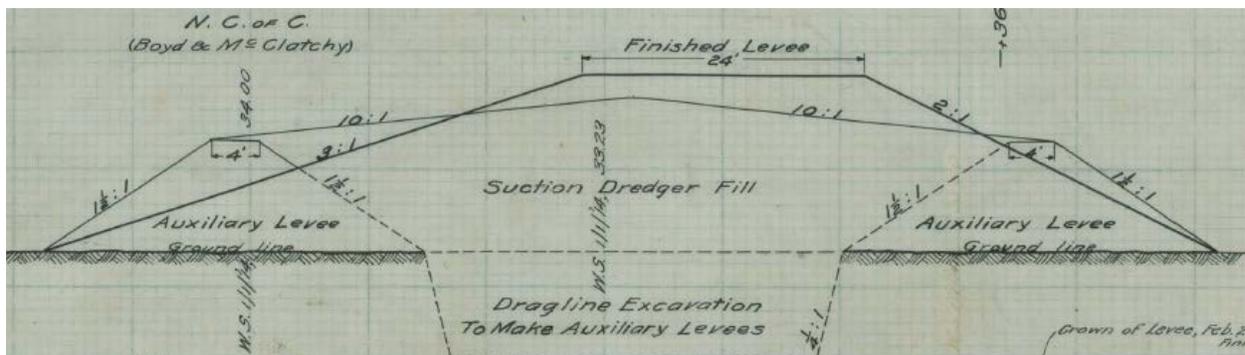


Figure 3-2: Sacramento River Typical Levee Section, 1912-1915

Following the 1986 flood events, structural levee improvements were initiated under several projects such as, the Sacramento Urban Area Levee Remediation Project (SUALRP) and ARCF Water Resources Development Act (WRDA) of 1996 and 1999. These projects differed from their predecessors in that instead of new levee construction or changes to the levee geometry, they incorporated seepage and slope stability improvements. These projects constructed drained stability berms, shallow through seepage cutoff walls, and deep underseepage cutoff walls along various portions of the American and Sacramento Rivers within the study area.

4.0 PAST PERFORMANCE

The study area has experienced several large discharges in recorded history, the earliest was in 1850 when the City of Sacramento was founded and another, larger, flood occurred in January 1862, inundating and substantially damaging Sacramento. These flood events quickly spurred the construction of flood protection levees along many of the banks of the Sacramento River, as well as the cultural re-alignment of the American River in 1868. Additionally, a large flood in 1907 breached the eastern Sacramento River levee near present day Derick Way (known as the “Edwards Break”), with subsequent flooding of reclamation districts on the eastern side of the

river south to the Delta. Additional flood events occurred in 1907, the middle of the century in 1950 and 1955, and more recently in 1986 and 1997, despite the existence of the upstream Folsom Dam. At least eight large floods have occurred in the lower American and Sacramento River basins since Folsom Dam became operational, these occurred in 1955, 1963, 1964, 1969, 1970, 1980, 1982, and 1986. The early floods spurred construction of flood control levees along the lower American River near Sacramento, and the recent floods tested the performance and capacity of the modern levee system.

In February 1986 and January 1997, major storms in Northern California caused record flood flows on the Sacramento and American Rivers. Record high outflows from Folsom Dam and Reservoir, together with high flows in the Sacramento River, resulted in water levels rising above the design freeboard of levees protecting the Sacramento area. It is estimated that major sections of levees along the American and Sacramento rivers would likely have failed if the storms had lasted longer. These events caused undermining of the levee embankment and also washed away portions of the riverbank. Figure 4-1 below shows the erosion distress experienced on the American River as result of the 1986 event. Subsequently flood events in 1997, and late 2005 to early 2006 each caused minor surficial erosion along the American River.



Figure 4-1: 1986 Erosion Distress on the American River South Bank Near I-80

Flood events in 1986, 1997, and 2006 have also caused seepage at or near the landside levee toe as well as at distances of 2,500 to 4,000 feet landward. Seepage distress was experienced throughout the Sacramento River levee reaches. Further documentation including aerial photographs, and reports by landowners, detailed seepage boils as well as eroding and sloughing banks at locations of former channels and oxbows beginning in the late 1930's. Flood fighting

(see Figure 4-2) has occurred throughout the Sacramento River reaches in both 1986 and 1997 as a response to seepage and stability concerns.



Figure 4-2: 1997 Flood Fighting on the Sacramento River in Natomas

5.0 GEOTECHNICAL REACH DESCRIPTION

American River North Basin

On the American River north bank (Reaches A and B), there is significant riparian habitat on a typically large waterside bench. In some areas, significant landside vegetation (mostly large trees) exists near the levee toe or on the levee slope. On the landside numerous encroachments include; fences at or near the landside levee toe, parking lots built, and residences. In Reach A the levee embankment is predominantly a silty sand to silt material constructed over a thick silt to silty sand blanket which is underlain by a poorly graded gravel aquifer. While in Reach B the levee embankment is predominantly a lean clay material constructed over a silt and lean clay blanket varying in thickness which is underlain by a silty sand and gravel aquifer. In both Reaches A and B a keyed in cutoff wall was constructed under WRDA 1996/1999 to mitigate underseepage.

On the East Side Tributaries (Reaches C through H), there is sparse vegetation on the levee slopes and adjacent to the embankment. On the landside of the NEMDC (Reaches C and F) levee embankment a railroad embankment is adjacent to the levee. Along approximately half of the Reach E alignment a concrete lined ditch is located at the landside levee toe which is approximately 5 to 10 feet deep. The NEMDC (Reaches C and F) levee embankment varies from clay sand and silt to lean clay and silt material. A lean clay blanket of variable thickness is underlain by a silty sand and poorly graded sand aquifer. On Arcade Creek (Reaches D and E), the levee embankment is predominantly clayey sand, lean clay, and silt constructed over a thin lean clay and silt blanket underlain by a silty sand to poorly graded sand aquifer. The Dry/Robla Creek (Reaches G and H) levee embankment is predominantly a clayey sand and silt constructed

over a clayey sand, lean clay, and silt blanket of varying thickness. The blanket is underlain by a silty sand aquifer.

Reach I includes both leveed and non leveed portions of Magpie Creek and the Magpie Creek Diversion Canal (MCDC) between Rose Street, where the Magpie Diversion Canal flows into Robla Creek, to the west and McClellan Air Force Base to the east. The levees associated along Magpie Creek and the MCDC are composed of silty sand and is underlain by predominantly dense to very silty sand and stiff to hard sandy silt with a somewhat prevalent hardpan layer about 4 feet thick, 2 to 6 feet below grade.

American River South Basin

On the American River south bank (Reaches A, B and C), there is significant riparian habitat on a typically large waterside bench. In some areas, significant landside vegetation (mostly large trees) exists near the levee toe or on the levee slope. On the landside numerous encroachments include; fences at or near the landside levee toe, power poles, parking lots, and residences. The Reach A and B levee embankment is predominantly composed of silty sand and sandy silt while the Reach C levee is predominantly poorly graded sand. The levees are underlain by a thick silt to silty sand blanket which is underlain by a sand and gravel aquifer. In both Reaches A and B a keyed in cutoff wall, and in Reach C a hanging cutoff wall, were constructed under WRDA 1996/1999 to mitigate underseepage.

On the Sacramento River, south of the American River confluence, (Reaches D through G), there is significant waterside and landside vegetation on both the slopes and at the toe of the levee. In all of the reaches, significant urban development has occurred up to the levee toe and/or the landside levee slope. Old Town Sacramento surrounds Reach D, Reaches E and F are residential areas of the Little Pocket and Pocket neighborhoods, and Reach G is the town of Freeport and an adjacent railroad alignment. On the landside numerous encroachments include; fences at or near the landside levee toe and across the levee crest, residences, commercial structures, stairways cut into the landside levee slope, and pools. The levee embankment is predominantly poorly graded sand and silty sand constructed over a lean clay and sandy silt blanket of varying thickness which is underlain by a poorly graded sand aquifer. A through-seepage cutoff wall was constructed which extends to a depth of approximately 18 to 40 feet in portions of Reach D and G and all of E through F. Sections of deep cutoff wall to approximately 110 feet were constructed in portions of Reaches E and F.

Natomas Basin

In the Natomas Basin, on the Sacramento River east bank (Reaches A, B, and C), there is significant landside and waterside vegetation. On the waterside of the levee crest (levee slope and toe) there exists almost continuous residential structures and related features such as, driveways, out structures, and landscaping. On the landside numerous encroachments are mostly due to agricultural uses (ditches, utilities, and structures) in Reaches B and C and residential in Reach A. The levee embankment was constructed of hydraulically placed sandy core between clay trainer dikes. The trainer dikes were constructed of material obtained by excavating the natural impervious blanket. The levee was constructed over a lean clay and silt blanket underlain

by a silty sand and poorly graded sand aquifer. In Reach A and portions of Reach B a shallow through seepage cutoff wall was constructed and in the rest of Reach B and C a landside stability berm. As part of the NPACR and adjacent levee is proposed for construction on the landside of the existing levee with either/or both a deep cutoff wall or landside seepage berm to mitigate levee underseepage.

On the Natomas Cross Canal south bank (Reach D), there is limited vegetation on both the landside and waterside of the levee, which was constructed on the canal bank. The levee embankment is predominantly a silt and clayey sand material constructed on a thin silt blanket which is underlain by poorly graded sand and silty sand aquifer. For a portion of the reach a landside stability berm was constructed to half the existing levee height. As part of the NPACR a cutoff wall is proposed for construction to mitigate levee underseepage.

On the PGCC and NEMDC west bank (Reaches E through H), there is vegetation on the waterside and landside of the levee. Encroachments are sparse for Reaches E through G (mostly agricultural uses) but in Reach H commercial development exists at the landside toe. The levee embankment is predominantly a clayey sand sandy lean clay constructed on a thin lean clay blanket which is underlain by silty sand to poorly graded sand aquifer. As part of the NPACR a cutoff wall or landside seepage berm is proposed for construction to mitigate levee underseepage.

On the American River north bank (Reach I) there is significant vegetation on both the waterside and landside of the levee; predominantly comprised of trees. Encroachments include primarily commercial buildings near the landside levee toe, and parking lots adjacent to the levee embankment. The levee embankment is predominantly silty sand constructed on a sandy silt blanket which is underlain by silty sand and poorly graded sand aquifer. As part of the NPACR a cutoff wall is proposed for construction to mitigate levee underseepage.

6.0 LEVEE FAILURE MODES AND ANALYSES CRITERIA

For the purposes of problem identification and alternatives analysis, several different failure modes have been evaluated for the without project condition. The failure modes included seepage (under and through), slope stability, erosion, overtopping and seismic.

Steady state seepage analysis considered a maximum allowable vertical exit gradient at the toe of the levee to be less than 0.5 for the water at the design elevation and 0.80 for the water at the top of levee elevation. For landside seepage berms a maximum gradient of 0.8 is allowable at the berm toe. The minimum required factor of safety for the design water surface elevation for the landside steady state slope stability analysis was 1.40 and 1.20 for the top of levee water surface elevation.

The main purpose of seismic vulnerability analyses was to identify the potential seismic performance of a levee. Although seismic remediation generally will not be implemented based on these analysis results, a levee's seismic degradation potential should be considered during selection of a static remediation, or in developing an emergency action plan to be implemented

following an earthquake. For the most critical category of levee (e.g., urban levees that are frequently hydraulically loaded) the following displacements are acceptable:

- Any deformation inducing crest displacement of 1 foot or less, unless larger lateral movements comprise the ability of foundation cut-offs or toe drains, etc. to provide for safe retention of high water.
- If more than 1 foot of seismic displacement is predicted, deformation is still acceptable if the levee continues to ensure water retention with 3 feet of freeboard for a 200-year flood event.
- If other safety criteria are met (e.g., cracking that can be repaired in a few days).

The typical USACE levee section, established by EM 1110-2-1913, is nationally considered to have a minimum 10-foot crest with 2:1 (horizontal: vertical) waterside and landside slopes. According to the Sacramento District 1969 “Design Manual for Levee Construction” levees should be constructed with 3:1 waterside and 2:1 landside slopes with either a 20 or 12-foot levee crest width for main stream or tributary levees respectively. The Sacramento District Geotechnical Engineering Branch, SOP-003 *Geotechnical Levee Practice*, suggests a 20-foot crest width with 3:1 waterside and landside slopes except existing levees with good past performance exists where existing 2:1 slopes are acceptable.

Vegetation, encroachment, and access policy includes EM 1110-2-1913, SOP 03, and ETL 1110-2-571 “Guidelines for Landscaping and Vegetation Management at Levees, Floodwalls, Embankments Dams, and Appurtenant Structures”. The vegetation-free zone, as established by ETL 1110-2-571, is a three-dimensional corridor surrounding all levees, floodwalls, and critical appurtenant structures in a flood damage reduction system. The vegetation-free zone applies to all vegetation except grass. The minimum height of the corridor is 8 feet, measured vertically from any point on the ground. The minimum width of the corridor is the width of the flood-control structure (Levee toes or floodwall stem), plus 15 feet on each side, measured from the outer edge of the outermost critical structure.

7.0 TYPICAL LEVEE IMPROVEMENT MEASURES

Where levee height, geometry, erosion, access, vegetation, seepage, and slope stability deficiencies were identified (criteria not met) improvement measures consisting of cutoff walls, seepage berms, relief wells, stability berms, geotextile reinforcement, flattened embankment slopes, flood walls, retaining walls, sliver fills, and various other measures could be implemented.

Seepage cutoff walls are vertical walls of low hydraulic conductivity material constructed through the embankment and foundation to cut off potential through seepage and underseepage. In order to be effective for underseepage mitigation, cutoff walls usually tie into an impervious sublayer. The conventional method using a long stick and boom excavator has a maximum depth of 70 to 80 feet. Deeper cutoff walls, up to about 150 feet could be excavated using cable excavation method with crane rigs. Mix-in-place methods of cutoff wall construction include deep mixing method, jet grouting, and cutter soil mixing. Deep Mixing Method uses specialized construction equipment to mix the soil with bentonite and cement in situ and is capable of depths

more than 100 feet. Pressure relief wells relieve excess pore pressures that can build up beneath a surficial blanket layer to reduce exit gradient. Relief wells collect seepage and bring it to the surface where it can be discharged freely on the ground surface or collected and drained away from the levee toe. Seepage berms are earth structures built at the landside toe that provide additional weight to prevent blanket layer heave, reduce exit gradients, and can allow safe exit of underseepage. The minimum seepage berm width is typically four times the levee height and the maximum width is generally 300 to 400 feet.

Slope flattening is a mechanical method to repair a slope that may not have stable slopes. Both the waterside and landside slopes can be graded using construction equipment. In most cases, this process requires the removal of all vegetation and encroachments from the levee slope being flattened. Slopes are typically flattened to 3H:1V to 5H:1V. Stability berms are constructed of a random fill material placed over blanket and chimney drainage features to capture seepage through the levee. A thin filter sand layer is placed between the drainage layer and the levee embankment and native soils. Drained stability berms have the benefit of also reducing susceptibility to through seepage. Geotextile is a type of synthetic material that is primarily used for soil reinforcement within an embankment. Geotextile is a woven pervious sheet of fabric constructed of synthetic plastic fibers. Geotextiles only provide reinforcement in tension, thus they are primarily buried within a soil at the tension surface to strengthen the soil.

Floodwalls are an efficient, space-conserving method for containing unusually high water surface elevations. They are primarily constructed from pre-fabricated materials, although they may be cast or constructed in place, and are constructed almost completely upright. Floodwalls are typically located along a levee hinge point to allow vehicular access along the crown. To address deficiencies found in the required levee freeboard various methods of raising the existing levee crown elevation could be implemented. A crown only levee raise assumes that the levee crown is currently wide enough to support the placement of additional embankment material while maintaining the minimum allowable crown width upon the completion of the raise. A full levee raise includes an embankment raise from the waterside crown hinge point upward at a 3H:1V slope, establishing a new crown width, and then down the landside at a new 3H:1V slope).

To protect against waterside erosion in areas where a waterside berm exists, a launchable rock trench may be constructed. This is accomplished by placing rip-rap a certain distance on the waterside slope and excavating a trench at the waterside toe, or where the waterside slope meets the berm. Rip-rap is then placed in the trench and then covered with random fill. As the waterside berm is erodes, it will eventually reach the launchable rock trench. At this point, the undermining action of the erosion event and soils surrounding the trench will allow for the rip-rap contained in the trench to “launch” into the void created adjacent to the trench. In areas that have no or minimal waterside berm, on bank rip-rap is placed on the waterside levee slope to protect against erosion. This entails filling the eroded portion of the bank and installing stone protection along the levee slope from the base of the erosion area to the top of the erosion area. Vegetation would be limited to grass. If there is a natural bank distinct from the levee that requires erosion protection, it would be treated with stone protection.

The purpose of the toe access easement is to allow for necessary maintenance, inspection, and floodfight access. For the ARCF project, the Project Delivery Team (PDT) has determined that a minimum toe access easement of 10 feet is required in association with other levee improvements. The actual toe access may vary depending on site specific constraints. Where vegetation management standards are not met, that levee section must be brought into compliance or a variance may be applied to a levee system or portion of that system to provide for the same levee functionality as intended in ETL 1110-2-571. In consideration for a vegetation variance request (VVR), the VVR will preserve, protect, and enhance the natural resources of the levee system or segment.

8.0 CROSS-SECTION SELECTION

Cross-sections for geotechnical analysis were selected to represent critical surface and subsurface conditions of each reach. The topography of each reach is inherently variable. The existence of access ramps on both landside and waterside of the levee, railroads running perpendicular and parallel to the levee, and/or pump stations or other structures built up adjacent to the levee section create difficulties to discern the typical versus critical cross-section. The sections were selected based on subsurface data, laboratory test results, geomorphology, surface conditions, field reconnaissance, historical performance, and levee geometry. The ground surface elevations used in the cross-sections were based on a LiDAR (light detection and ranging) survey completed in November 2008 for the DWR, ULE project. The natural soil layers were delineated based on boring logs and laboratory test results. Elevation references in this report are in feet and are based on the North American Vertical Datum of 1988 (NAVD88) unless otherwise noted. All horizontal references in this report are in feet and are based on the California State Plane, Zone II, North American Datum of 1983 (NAD83). Plate 1 shows the relationship between LM, RM, and stationing that were used to describe the location of the cross-sections. Table 8-1 and Plate 2 presents the cross-sections where geotechnical analyses were performed, and used in the economic analyses.

Table 8-1: Index Point Locations

Basin	Reach	Channel	Bank	Unit	Levee Mile
ARN	A	American River	North	9	1.32
ARN	E	Arcade Creek	North	7	0.90
ARS	B	American River	South	4	3.90
ARS	E	Sacramento River	East	1	1.12
NAT	D	NCC	South	2	1.17

9.0 HYDRAULIC LOADING CONDITIONS

Water surface profiles for the ARCF GRR study area were obtained from the Hydraulics and Hydrology Branch, Sacramento District. The profiles provide water surface elevations in NAVD 88 by river mile for various flood frequencies. Deterministic seepage and stability analyses were performed for various flood frequencies typically incorporating the 25yr, 50yr, 100yr, 200yr, 500yr, and top of levee. The probabilistic analyses were performed for a range of stages not correlated to flood frequency, but which represented stages from no head (landside toe of levee) to maximum head (top of levee).

During the preparation of this report, the hydraulic model was in the process being revised and updated. Due to the detailed review process required of the hydraulic model update, the decision was made to perform the deterministic analyses using draft hydraulic model water surface profiles for various flood frequencies. Water surface profiles for deterministic seepage and slope stability analyses for the Natomas Basin were based on the previous hydraulic model dated December 2010, for the American River North Basin (ARN) Reaches A and B and American River South Basin (ARS) Reaches A through G were based on a draft update dated July 2011, and for ARN Reaches C through H were based on a draft update dated March 2012. The various different models have a variability of 1 to 3 feet of stage for the same flood frequency.

Table 9-1 below summarizes the water surface elevations deterministically analyzed at each index point, by basin.

Table 9-1: Analyses Water Surface Elevations

Index Point	Event	Stage	Head	Index Point	Event	Stage	Head
ARN Reach A U9 LM 1.32	Crest	52.95	9.69	ARS Reach E U1 LM 1.12	Crest	40.72	19.94
	500yr	55.83	12.57		500yr	35.82	15.04
	200yr	51.1	7.84		200yr	34.31	13.53
	100yr	46.27	3.01		100yr	32.51	11.73
	50yr	46.16	2.9		50yr	31.83	11.05
	25yr	46.08	2.82		25yr	31.22	10.44
ARN Reach E U7 LM 0.90	Crest	43.94	12.25	NAT Reach D U4 LM 1.17	Crest	44.85	15.3
	500yr	46.12	14.43		500yr	45.16	N/A
	200yr	41.34	9.65		200yr	44.23	14.68
	100yr	39.08	7.39		100yr	42.84	13.29
	50yr	37.68	5.99		50yr	41.88	12.33
	25yr	35.34	3.65		25yr	41.11	11.56
ARS Reach B U4 LM 3.90	Crest	48.83	16.03				
	500yr	47.76	14.96				
	200yr	41.31	8.51				
	100yr	38.13	5.33				
	50yr	37.73	4.93				
	25yr	37.41	4.61				

10.0 SEEPAGE AND SLOPE STABILITY ANALYSES

Deterministic steady state seepage analysis was performed using the finite element program SEEP2D within GMS 6.5 (Groundwater Modeling System). Results from the seepage analysis were used to calculate average vertical exit gradients at the landside levee toe and/or at a more critical location near the levee toe if applicable, for example at the invert of the empty drainage ditch. The pore pressures and/or phreatic surfaces were exported to UTEXAS4.0 for use in slope stability analysis.

Boundary conditions along the waterside ground surface from the waterside model extents to the levee slope were assigned as fixed total head conditions corresponding to the analyzed water elevation. On the landside, exit face boundary conditions are applied from the landside crest hinge point to landside extents of the model. All other boundaries not explicitly assigned a condition are assumed by the program to be no flow which include both vertical faces of the model and the bottom nodes. The landside model extents were extended 2,000 feet from the levee centerline and for the waterside model extents to the channel centerline.

Embankment stability against shear failure was analyzed using the UTEXAS4.0 software package for steady state conditions. Analyses to find factors of safety against sliding were conducted using a floating grid automatic circular failure surface search routine to identify the critical failure surfaces with Spencer Procedure within the embankment and/or foundation. The Spencer Procedure satisfies both force and moment equilibrium for each slice. A minimum weight restriction was applied to the slices within the failure surface to eliminate surficial failure surfaces. Where tensile stresses exist on the failure surface, a water filled crack depth was introduced to eliminate the tensile stresses, but not compressive stresses.

Material properties including hydraulic conductivity for seepage analysis and drained (effective) shear strength and unit weight for slope stability analysis were developed based on a review of field and laboratory data that was then generalized into appropriate parameters by material type. The stratigraphy of the existing levee cross-section was divided into unique layers typically consisting of levee embankment fill, foundation or blanket layer, pervious aquifer layers separated by an aquitard, and a deeper fine grained layer. Analysis material parameters were assigned considering saturated conditions.

The results of the without project seepage and slope stability analyses indicate that the levees in ARN Reaches C through F and ARS Reaches D through G did not meet minimum criteria. The analyses showed that the levees did not meet criteria at varying flood frequencies typically between the 25 and 200 year events. In general, the analyses identified underseepage deficiencies and/or underseepage related slope stability deficiencies. Therefore, the with project analyses typically included deep cutoff walls which resulted in the with project levee analyses satisfying criteria.

10.1 ARN REACH A – AMERICAN RIVER NORTH – U9 LM 1.32

The without project conditions analyses includes the WRDA 1996/1999 cutoff wall and met criteria for both seepage gradients and slope stability factors of safety. As no seepage and stability deficiencies exist, no further improvements are recommended. Figure 10-1 displays steady state seepage and landside slope stability results for analyzed flood frequencies.

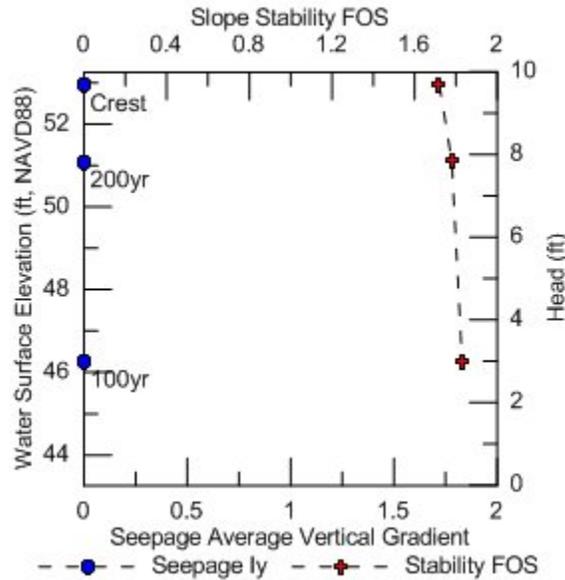


Figure 10-1: ARN Reach A U9 LM 1.32 Without Project Analyses Results

10.2 ARN REACH E – ARCADE CREEK NORTH – U7 LM 0.90

The without project conditions seepage analysis of the Arcade Creek north levee have shown the potential for seepage gradients to exceed criteria beginning at the 100 year flood frequency event due to a thin clay blanket underlain by poorly graded sand layer. Related to the underseepage deficiency, slope stability factors of safety do not meet criteria beginning at the 50 year flood frequency event. The 100 year flood frequency event corresponds to a water surface elevation of 39.08 feet and 7.39 feet of head and the 50 year flood frequency event corresponds to a water surface elevation of 37.68 feet and 5.99 feet of head. Additionally, an open drainage ditch to the landside levee toe from NEMDC to Norwood Avenue amplifies the slope stability deficiency. With project conditions analyses addressed the underseepage and slope stability deficiencies by incorporating a cutoff wall be keyed-in to a low permeability confining layer at elevation -6.0 feet. With the improvement measures described above seepage and stability analyses met criteria. Figure 10-2 displays the without project conditions analyses results and Figure 10-3 displays the with project analyses results for analyzed flood frequencies.

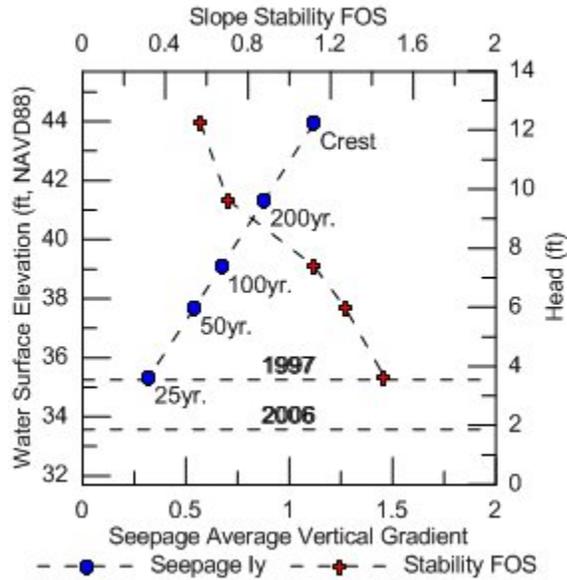


Figure 10-2: ARN Reach D U7 LM 0.90 Without Project Analyses Results

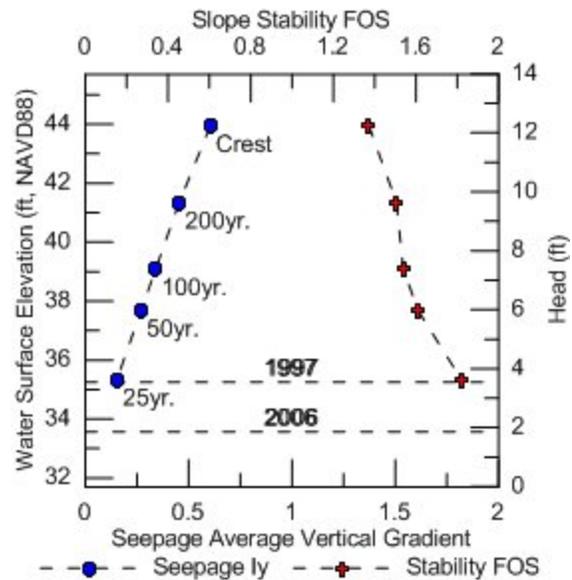


Figure 10-3: ARN Reach D U7 LM 0.90 With Project Analyses Results

10.3 ARS REACH B – AMERICAN RIVER SOUTH – U4 LM 3.90

The without project conditions analyses includes the WRDA 1996/1999 cutoff wall and met criteria for both seepage gradients and slope stability factors of safety. As no seepage and stability deficiencies exist, no further improvements are recommended. Figure 10-4 displays steady state seepage and landside slope stability results for analyzed flood frequencies.

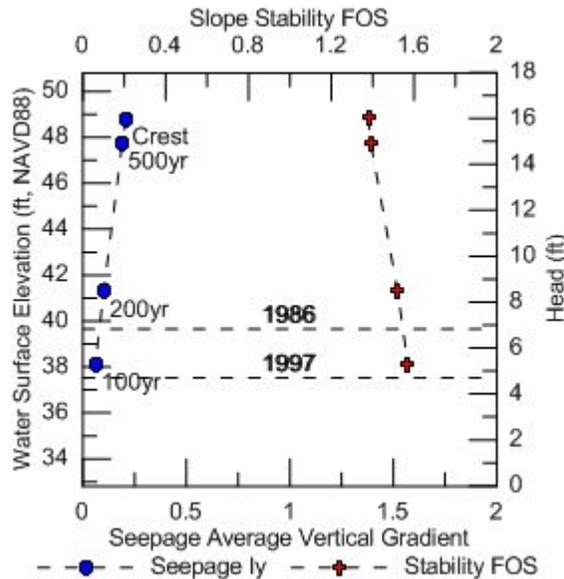


Figure 10-4: ARS Reach B U4 LM 3.90 Without Project Analyses Results

10.4 ARS REACH E – SACRAMENTO RIVER EAST – U1 LM 1.12

The without project conditions analyses on the Sacramento River, at this location, included a through seepage cutoff wall constructed by the SUALRP in the early 1990's. Additionally, a deep cutoff wall was constructed in this area between LM 0.90 and 1.36 by SAFCA in 2006. This section was analyzed both with and without the deep underseepage cutoff wall. With the deep cutoff wall, seepage and slope stability analyses results met criteria. Explorations performed since the SAFCA cutoff wall was constructed indicate that nearby subsurface conditions outside the cutoff wall extents are similar. Therefore, results are presented for without the cutoff wall as well for use in probabilistic analyses presented in Section 18.

Seepage and slope stability analyses results did not meet criteria beginning at the 25 year flood frequency event, which corresponded to a water surface elevation of 31.22 feet and 10.44 feet of head. A review of past performance indicated that seepage was observed at numerous locations both before and after the construction of the shallow cutoff wall. Past flood event stages were compared to analyzed flood frequencies and the 1997 event (maximum recorded stage) was comparable to the 50 year flood frequency. The past performance is in agreement with the seepage and stability analyses results. With project conditions analyses addressed the underseepage deficiencies by incorporating a keyed-in cutoff wall to tip elevation -75.0 feet. With project seepage and stability analyses at the 25 year to 100 year flood frequency events indicated marginal underseepage gradients (0.4 to 0.5) and slope stability factors of safety (1.40 to 1.45). Figure 10-5 displays the without project conditions analyses results and Figure 10-6 displays the with project analyses results for analyzed flood frequencies.

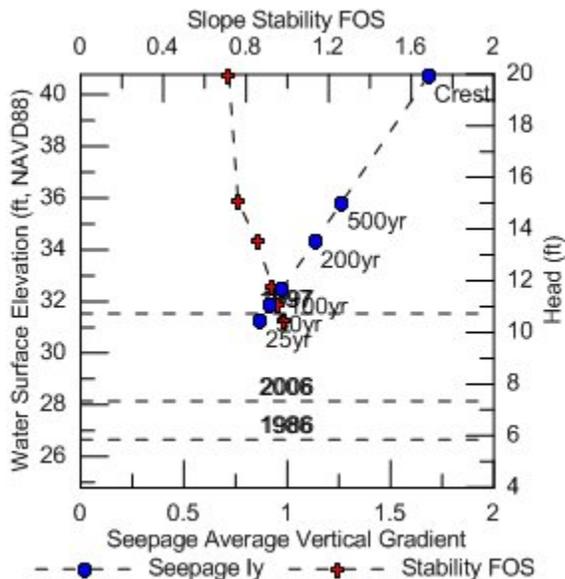


Figure 10-5: ARS Reach E U1 LM 1.12 Without Project Analyses Results

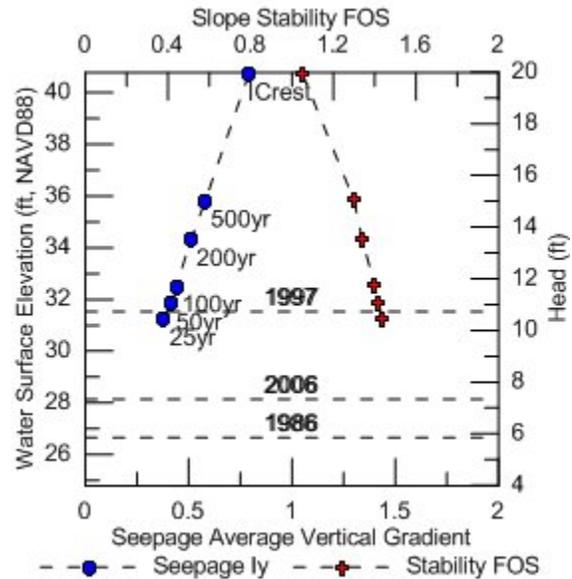


Figure 10-6: ARS Reach E U1 LM 1.12 With Project Analyses Results

10.5 NAT REACH D – NCC SOUTH – U1 LM 1.17

The without project conditions analyses includes the NPACR cutoff wall which satisfies criteria for both seepage gradients and slope stability factors of safety. The ARCF GRR included a levee raise, the NPACR analyses section met top of levee analyses criteria with the raise. Figure 10-7 displays steady state seepage and landside slope stability results for analyzed flood frequencies.

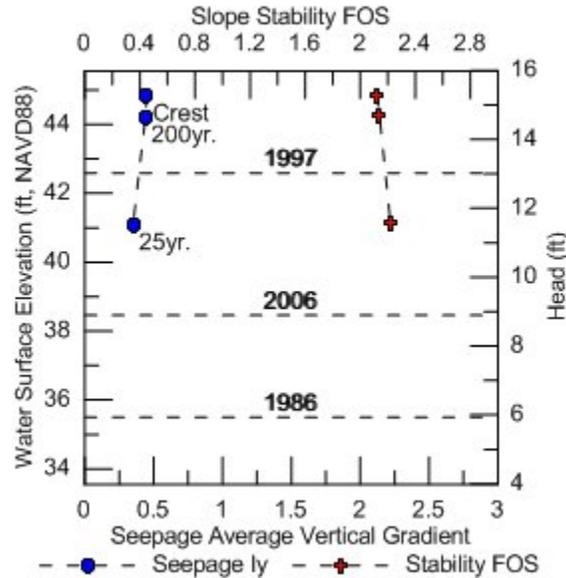


Figure 10-7: NAT Reach D U1 LM 1.17 Without Project Analyses Results

11.0 EROSION ASSESSMENT

The American River levees were originally intended to convey a release from Folsom Dam of 115,000 cfs. During several events since the construction of Folsom Dam, flows have exceeded design capacity and caused significant erosion distress. Additionally, the objective release from Folsom Dam is currently under review as part of the Folsom Dam Reoperations Study and the Joint Federal Project is currently constructing improvements to the dam for a release of 160,000 cfs.

Insufficient geotechnical data were available to adequately support existing and proposed channel stability analyses and potential design recommendations. Specifically the geotechnical and geologic study focused on characterization of soil properties through exploration and testing, geologic mapping, and 3-dimensional modeling of the subsurface stratigraphy. Additional geotechnical data was generated to characterize the material comprising the existing channel bed between the Right and Left bank levees of Lower American River (LAR) between River Mile (RM) 5.0 and 11.0. Specifically, the geotechnical study was directed to investigation the location and properties of a potentially erosionally resistant unit, better represent and organize the existing geotechnical data, and improve upon existing geologic mapping.

Additional subsurface explorations consisted of borings drilled through the waterside berm and channel, CPTs drilled through the waterside berm, and geotechnical laboratory testing.

Relatively undisturbed samples were collected of the potentially erosionally resistant unit for erosion rate testing to be used in hydraulic modeling and analyses. JET testing was performed by the Engineer Research and Development Center, Geotechnical and Structures Laboratory and the EFA testing was performed by the Texas A&M Department of Civil Engineering. Results of the EFA and JET test results are presented and discussed in the subsurface investigation memorandums and geotechnical data report. ERDC concluded that the erodibility of each sample was related to the geologic unit and that most of the specimens within the Fair Oaks formation (below the erosionally resistant surface) could be categorized as Moderate Resistant to Very Resistant. Similarly, in general, the layer above erosionally resistant surface could be categorized as Very Erodible to Erodible.

Fugro Consultants performed the geologic portion of the erosion study, key findings from their report, *Lower American River Stratigraphic and Geomorphic Mapping Report* (2012), are reproduced below. Two levels of investigation were performed: (1) detailed mapping and analysis of the geologic deposits between the levees from RM 5.0 to 11.0, and (2) development of reconnaissance mapping along the channel corridor between RM 0.0 to 5.5 and RM 11.0 to 22.4. Detailed geologic mapping, as well as petrographic and pedogenic analyses, completed during this study demonstrated the presence of two potentially erosion-resistant units. These were: (1) a moderately cohesive silty and sandy interbed of relatively limited lateral and longitudinal extent within a thicker package of loose Holocene sediments (the “upper” unit); and (2) much thicker, more widespread relatively erosion-resistant deposits associated with the Pleistocene-aged Fair Oaks formation of Shlemon (1967) (the “lower unit”).

A 3-dimensional (3D) stratigraphic model of the LAR study reach was developed by incorporating both existing and newly collected geotechnical and geologic data. The model described the stratigraphy and subsurface conditions of the study reach and help evaluate the stratigraphic susceptibility of this reach to erosion near the levee banks.

A review of the applied velocity and shear stresses at different simulated flows was performed on the left and right banks of the study area at several locations. In general velocity exceeds permissible values for fine sands prior to reaching the 30,000 cfs flow and exceed permissible velocities for most soils by 50,000 cfs on the right bank and between 80,000 and 115,000 on the left bank. Therefore, it is reasonable to assume that, depending on flood event duration, channel and bank geometry, levee geometry, soil type, and vegetation cover, that erosion distress to the American River Levees has the potential to occur beginning at flows exceeding 30,000 cfs.

12.0 SEISMIC ASSESSMENT

To evaluate the potential to liquefaction resistance of soils, liquefaction triggering analysis was performed based on the procedure from the summary report of the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, published as part of the *Journal of Geotechnical and Geoenvironmental Engineer*, dated October 2001 (Youd, Idriss, Andrus, & Arango, October 2001).

The result of the liquefaction triggering analysis and liquefaction-induced post-earthquake deformation based on limit equilibrium analysis indicates that liquefaction potential is highly

likely at each critical location for all the reaches for Natomas Basin, Reach A of American River North Basin, and Reaches C to G of American River South Basin. Moreover, at these locations, the analysis indicates that the post-earthquake deformation as the result of liquefaction of the material beneath the embankment is a global or structural failure mode that is very likely to compromise the ability to provide flood protection at these critical locations.

13.0 VEGETATION VARIANCE REQUEST SUPPORTING ANALYSES

The majority of the Sacramento River levee within the study area, require seepage, slope stability, height, and erosion improvements in order to meet USACE criteria. Construction of the levee improvement measures will require complete vegetation removal on the levee from approximately 15 feet landward of the landside toe to approximately 1/3rd the height of the levee on the waterside slope. On the waterside, where construction does not remove vegetation, on the lower 1/3rd of the slope to 15 feet waterward of the waterside levee toe, the vegetation will be left in place and a Vegetation Variance Request (VVR) will be sought by the Sacramento District. To show that the safety, structural integrity, and functionality of the levee would be retained, an evaluation of underseepage and waterside embankment slope stability was completed given that a tree fell resulting in scouring of the root ball area.

The analyses section/index point at LM 5.92 was chosen for the VVR analyses because it was considered to be representative of the most critical channel and levee geometry and the without project analyses showed the section does not meet underseepage and slope stability criteria. The cross-section geometry of the index point incorporated tree fall and scour by using a maximum depth of scour for cottonwoods as approximately 11.0ft; the associated soil removed was projected at a 2:1 slope from the base of the scour toward both the landside, and waterside slopes. The base scour width was equal to the maximum potential diameter at breast height (dbh) of Cottonwoods (12.0ft) projected horizontally at a depth of 11.0ft below the existing ground profile. The results show that the tree fall and scour did not significantly affect levee performance and that the levee meets USACE seepage and slope stability criteria considering the seepage and stability improvement measures are in place (“with project” conditions). Therefore, it is a reasonable conclusion that with a VVR to allow vegetation to remain, the safety, structural integrity, and functionality of the Sacramento River levee would be retained.

14.0 GEOTECHNICAL RECOMMENDATIONS FOR LEVEE IMPROVEMENT

The levees protecting the ARCF GRR study area are susceptible to through seepage, underseepage, slope stability, and erosion. In some locations, on the levees along the American River and Natomas Basin, substantial projects have been authorized to address these deficiencies. However, seepage, slope stability, and/or erosion deficiencies still remain on portions of the Sacramento River, on the east bank of NEMDC, on both banks of Arcade Creek, and on both banks of the American River. To address seepage and seepage related slope stability deficiencies the predominant recommendation is cutoff walls. Due to several factors including constraints on expanding the levee footprint due to urban development; seepage berms, relief, wells, and the vast majority of other seepage improvement measures were considered not feasible. Based on hydraulic modeling, some reaches may require raises to prevent overtopping at certain flood frequencies. Plate 3 shows the locations of those improvements.

On the American River erosion continues to be a potential failure mode that requires additional improvements to convey design flows. To accomplish this, two erosion protection measures have been proposed that could be implemented in combination along the levee alignment depending on factors such as, bank/bench geometry, existing habitat, and existing land use among other considerations. The two measures are a launchable rock trench and on bank rip-rap.

On the Sacramento River east levee (Reaches ARS D through G), the need for further seepage and slope stability improvements has been identified through geotechnical analyses. A combination of conventional open trench and DSM cutoff wall construction methods is anticipated. Levee raises are proposed in some segments of the Sacramento River levee which would be accomplished with a levee embankment raise and retaining wall at the landside levee toe or a flood wall constructed at the levee crest. The levee at Pioneer Reservoir was improved by the Sacramento District with relief wells and a landside seepage berm to meet criteria at the 100 year flood event. At this location additional thickness should be added to the seepage berm to increase the level of protection. To address a slope stability deficiency in Reach G, a full levee degrade and placement of geotextile within the reconstructed levee embankment is recommended. The Sacramento River levee requires erosion protection, which could be addressed with the measures presented for the American River. The extents of levee seepage and slope stability improvements for the Sacramento River are shown in Table 14-1.

Table 14-1: Sacramento River Reaches D through G Recommended Improvements

Reach	Beginning LM	Ending LM	Tip Elevation (ft)	Depth (ft)	Type	Description
D	0.00	0.18	NA	NA	NA	USACE 2000 CW
	0.18	1.25	NA	NA	NA	High Ground
	1.25	1.97	NA	NA	NA	Pioneer Reservoir
	1.97	3.2	-80	120	DSM	Hanging/Keyed In
	3.2	3.63	-45	80	Open Trench	Keyed In
	3.63	0.59	NA	NA	NA	High Ground
E	0.59	0.9	-95	135	DSM	Partial
	0.9	1.36	NA	NA	NA	SAFCA 2006 CW
	1.36	1.71	-75	115	DSM	Keyed In
	1.71	2.39	-65	105	DSM	Keyed In
E/F	2.39	3.39	-55	95	DSM	Partial
F	3.39	3.67	-75	115	DSM	Hanging
	3.67	3.99	NA	NA	NA	USACE 2006 CW
	3.99	4.57	-75	110	DSM	Keyed In
	4.57	5.01	-45	80	Open Trench	Keyed In
	5.01	6.03	-110	145	DSM	Partial
	6.03	6.11	NA	NA	NA	Sump 132 CW
	6.11	6.28	-85	120	DSM	Keyed In
	6.28	7.36	-65	100	DSM	Keyed In
F/G	7.36	8.32	-60	95	DSM	Hanging
G	8.32	10.64	NA	NA	NA	Geotextile

The NEMDC, Arcade Creek north, Dry Creek, and Robla Creek levees sections were improved in the 1990’s to early 2000’s by SAFCA and, although they did not include internal seepage improvements, the levees meet geotechnical analyses criteria for seepage and slope stability, except in limited segments. The exception being a portion of NEMDC from its confluence with the American River, both banks of Arcade Creek, and a section of NEMDC where the historic Magpie Creek intersects the levee foundation. To address underseepage and underseepage induced slope stability deficiencies the proposed improvement measure is a cutoff wall constructed with the conventional open trench method. The Arcade Creek south bank proposed cutoff wall is proposed in conjunction with a full levee height degrade and incorporation of geotextile placed within the reconstructed levee embankment. The Arcade Creek north bank cutoff wall is proposed in conjunction with replacing the existing landside ditch with buried culverts from the confluence of NEMDC upstream to Rio Linda Blvd. Depths and locations of the proposed cutoff wall are shown in Table 14-2 below

Table 14-2: East Side Tributaries Reaches C through F Proposed Cutoff Wall Extents

Reach	Beginning		Ending		Tip Elevation (ft)	Depth (ft)
	Unit	LM	Unit	LM		
C	2	0.00	2	0.48	-45	80
D	1	0.00	1	2.08	-6	45
E	7	0.00	7	1.92	-6	45
F	2	2.01	2	3.24	-45	80

For the levees surrounding the Natomas Basin, the geotechnical recommendations for seepage and slope stability were addressed in the NPACR. The recommended alternative from that report included a combination of seepage berms and cutoff walls combined with either an adjacent levee or fix-in-place improvement to the existing levee. To address levee overtopping concerns, a floodwall at the waterside hinge point or a levee embankment raise is proposed where necessary as part of the ARCF GRR.

15.0 MAGPIE CREEK

In the early 1990’s, the Sacramento District and SAFCA began studying the Magpie Creek and MCDC flood control project after the realization that the system was overtopped during frequent events. In the late 1990’s and early 2000’s, both the Sacramento District and SAFCA developed various improvement alternatives for the project. While the alternatives varied slightly, they included similar measures such as, levee raises (either embankment or floodwall), new levee construction, channel improvements (deepening or widening), and construction of detention basins. Levee improvements consisting of either levee embankment raises or flood wall raises were proposed for the left bank of Magpie Creek and the MCDC, while new levee construction was proposed for the right banks. Based on the available existing feasibility and design documents prepared in the 1990’s and early 2000’s by the Sacramento District and SAFCA, the levee, channel, and detention basin improvement measures appeared to be geotechnically adequate and require no further geotechnical analyses as part of the ARCF GRR.

16.0 SACRAMENTO WEIR AND BYPASS

The existing Sacramento Weir and Bypass, which allows high flows in the Sacramento River to be diverted into the Yolo Bypass, could be expanded to roughly twice their current width to accommodate increased bypass flows. The existing north levee of the Sacramento Bypass would be degraded and a new levee constructed approximately 1,500 feet to the north. The existing Sacramento Weir would be expanded to match the wider bypass.

The new north levee of the Sacramento Bypass would be constructed as per the standard levee section for new construction which includes; Type I Levee Fill, 3H:1V waterside and landside slopes, and a minimum crest width of 20 feet. As both the existing north and south levees have experienced undersepage and slope stability related distress, the new north levee would include a 300-foot wide drained landside seepage berm (5 feet thick at the landside levee toe tapering to 3 feet thick at the berm toe and constructed of random fill with a 1.5-foot thick drainage and filter layer at the base) with a system of relief wells located at least 15 feet landward of the berm toe and spaced at 200-foot intervals. A seepage cutoff wall with tip elevation of 5 feet should be constructed beneath the extension to the Sacramento Weir and the existing portion of the weir.

17.0 DIVERSION STRUCTURE

In lieu of substantial improvements to the levees on the Sacramento River downstream of the American River confluence a structure has been proposed to divert flows from the Sacramento River to the bypass system. The I Street Diversion Structure would restrict flows going down the Sacramento River past the City of Sacramento as well as the City of West Sacramento and force a portion of the flows (from the Sacramento and American Rivers) to travel upstream and through the Sacramento Weir and Bypass out to the Yolo Bypass. The effect of the structure would be to reduce the water surface elevation of the Sacramento River downstream of the structure to the point at which seepage, stability, height and erosion improvements would not be necessary to safely convey the design flood event.

Geotechnical Engineering Branch performed a review on October 18th, 2012 of the foundation design of the draft rough order magnitude cost estimate design submitted by New Orleans District (MVN) in September 2012. Additionally, the Sacramento District held a workshop on November 28-29th, 2012 which was tasked with using engineering judgment to evaluate the proposed design and make a recommendation on the engineering viability of the structure. The workshops in general found the structure to be constructible but identified the major design and construction considerations. Plate 4 shows a stratigraphic cross-section of the proposed structure.

While the Diversion Structure was intended to reduce the significant improvements required to the Sacramento River levees south of the American River confluence, it does not serve the same purpose for levee upstream of the structure. Levees within the study area in the NAT, ARN, and ARS basins upstream of the structure would continue to require the identified seepage, stability, overtopping, erosion, etc improvements. In addition to the improvements required for the ARCF GRR study are levees for NAT, ARN, and ARS basins that would be required to support the function of the Diversion Structure, seepage and stability improvements for the Sacramento River west bank levee and Yolo Bypass east bank levee in the West Sacramento Project GRR

study area. These improvements are shown in Tables 17-1 and 17-2 below. Plate 5 shows the locations of proposed seepage and slope stability levee improvements associated with the Diversion Structure.

Table 17-1: Sacramento River West Levee Recommended Improvements

Maintaining Agency	Unit	Beginning LM	Ending LM	Tip Elevation (ft)	Depth (ft)	Type
RD 537	1	4.17	4.76	0	40	Open Trench
MA4	2	0.0	0.74	0	40	Open Trench
		1.29	2.03	15	25	Open Trench
		2.03	2.31	-50	90	DSM
		2.31	2.87	-80	120	DSM
		2.87	3.40	-80	120	DSM

Table 17-2: Yolo Bypass East Levee Recommended Improvements

Maintaining Agency	Unit	Beginning LM	Ending LM	Tip Elevation (ft)	Depth (ft)	Type
RD 537	2	0.81	1.17	-70	105	DSM
RD 900	2	1.62	2.09	-10	45	Open Trench
		3.05	3.34	-60	95	DSM
		3.34	4.67	-110	140	DSM
		4.67	5.73	-30	60	Open Trench
DWSC: YBEL to Closure Structure				-60	95	DSM

The diversion of flow from the I Street Diversion Structure requires increased capacity in the Sacramento Bypass to convey those flows to the Yolo Bypass. The increase in capacity would be obtained through widening of the Sacramento Weir and construction of a new northern levee on the Sacramento Bypass. In order to convey an approximately additional 60,000 cfs in the Yolo Bypass the flow could be routed to the Deep Water Ship Channel (DWSC) by degrading the existing navigation levee and constructing a weir and spillway, similar to the Sacramento Weir expansion. The additional flood flows would stages during flood events on the DWSC (western levee) and Yolo Bypass levee thereby necessitating seepage and stability improvement measures from the proposed DWSC Weir to the confluence of the DWSC. If the DWSC were to be utilized for increased flow conveyance through the Yolo Bypass, a landside seepage berm would be required for the extents shown in Table 17-3.

Table 17-3: Yolo Bypass, Miner Slough, and Cache Slough Levee Improvements

Maintaining Agency	Unit	Beginning LM	Ending LM
RD 999	1	0.00	15.43
	2	0.00	2.32
RD 501	3	7.82	0.00
	2	3.57	0.00

If the additional 60,000 cfs routed to the Yolo Bypass by the Diversion Structure was not addressed through increased capacity in the DWSC, the levees along the Yolo Bypass could be improved to convey the additional flow. Based on current knowledge of past performance, recent period inspection reports, and personal experience with the Yolo Bypass and adjacent levees, Geotechnical Engineering Branch, Sacramento District held meetings in September, 2012 to formulate the required levee improvements. These improvements would mostly consist of landside canal relocation, levee reconstruction, stability berms, seepage berms, slope armoring, and foundation improvements, as shown in Table 17-4.

Table 17-4: Yolo Bypass Recommended System Improvements

MA	Unit	Beginning LM	Ending LM	Issue/potential problem	Likelihood of problem	Minimal Fix to meet Geotechnical Criteria
RD 2035	2	0.00	7.57	Seepage & Stability	High	Fill landside canal, erosion protection, & combination seepage/stability berm
	3	2.51	0.00	Stability	High	Adjacent levee
ST 12	1	5.09	0.00			
	2	0.00	7.37			
ST 11	4	0.00	3.61	Stability & Erosion	High	Fill landside canal & erosion protection
ST 7	1	8.99	0.00	Seepage & Stability	Medium	Fill landside canal
	2	7.30	0.00	Seepage & Stability	Low	Landside stability berm
RD 2068	1	0.00	5.5	Stability & Erosion	High	Fill landside canal & erosion protection
RD 2098	1	0.00	3.58	Stability & Erosion	High	Rebuild levee, erosion protection, & foundation improvements 30ft bgs
	1A	0.00	0.55			
	2	7.30	5.29	Stability & Erosion	High	Construct landside stability berm & erosion protection
RD 2060	3	1.35	5.03	Stability & Erosion	High	Rebuild levee, erosion protection, & foundation improvements 30ft bgs
	1	4.10	0.00	Stability & Erosion	High	Rebuild levee, erosion protection, & foundation improvements 30ft bgs
RD 536	1	1.35	5.66	Stability & Erosion	High	Construct landside stability berm & erosion protection
RD 536	2	0.00	4.97	Stability	Medium	Landside stability berm & erosion protection
RD 827	2	0.00	2.84	Stability	High	Rebuild levee, erosion protection, & foundation improvements 20ft bgs
RD 785	2	0.00	3.30			
RD 537	1	0.65	3.78	Seepage	High	Combination seepage/stability berm

18.0 PROBABILISTIC ANALYSES

The First-Order-Second-Moment (FOSM) method, as recommended in ETL 1110-2-556, "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies" dated 28 May 1999, was followed during the probabilistic evaluation of each index point. In this approach, the uncertainty in performance is taken to be a function of the uncertainty in model parameters. The standard deviations of a performance function were estimated based on the expected values (means) and the standard deviation of the random variable means. The performance functions considered were underseepage, through-seepage, and slope stability.

Potential sources of levee distress or failure considered in the analyses were underseepage through the levee foundation, through-seepage through the levee embankment, and instability of the landside levee slope under steady state conditions. The levees were evaluated against the above mentioned performance modes at five different water surface elevations (loading conditions), which included; levee crest, levee crest minus three feet, half levee height, toe plus three feet, and landside levee toe where the probability of failure was considered to be zero. Using this method of selecting loading conditions the curves should represent probability of poor performance at multiple flood frequencies.

The probability of poor performance was evaluated by assessing the foundation and embankment materials and assigning values for the probability moments of the random variables considered in the analyses. Random variables for underseepage were considered for the ratio of the horizontal permeability of the aquifer to the vertical permeability of the upper less permeable blanket, blanket thickness, and aquifer thickness. Random variables for through-seepage were considered for critical tractive stress, porosity, and intrinsic permeability of the levee embankment material. Random variables for slope stability were considered for effective friction angle, effective cohesion, and total unit weight of the levee embankment, and effective friction angle and cohesion of the foundation material.

A judgment based conditional probability function for each analyzed cross-section was based on existing conditions of the levee such as encroachments on the levee slopes, vegetation on the levee slopes and in the vicinity of the levee toes, existing cracks and holes due to animal burrows, erosion of the waterside levee slopes and riverbank, and considering the past history of sand boils or slope failures. Generally, past experience with poor performance at utility crossing and rodent activity indicates the risk of failure is somewhat significant in the analyzed areas.

The conditional probability of failure as a function of floodwater elevation has been developed by combining the probability of failure functions for all considered failure modes; underseepage, through-seepage, slope instability, and judgment.

The without project levee performance curves indicate that the levees in ARN Reaches C through F and ARS Reaches D through G would perform unsatisfactorily when minimally loaded. In general, the analyses identified underseepage deficiencies and/or underseepage related slope stability deficiencies. Therefore, the with project levee performance curves considered the proposed improvement measures for each failure mode, as necessary, which resulted in significant reduction in probabilities of poor performance.

18.1 ARN REACH A – AMERICAN RIVER NORTH – U9 LM 1.32

Borings chosen to be used in probabilistic analyses resulted in a mean blanket thickness value of 15.0 ft with a coefficient of variation of 96, and a mean aquifer thickness of 24.0 ft with a coefficient of variation of 42. The blanket was comprised of predominantly silty sands. The aquifer was made up of poorly graded sand to silty sand, and silty gravel.

The levee embankment contains an existing cutoff wall which mitigates underseepage, through seepage, and slope stability concerns. The without project judgment based probability portion of the curve was comprised mainly of erosion, and encroachments, accounting for 50.0% and 4.0% respectively at the crest. Past performance has indicated significant amounts of erosion of the riverbank, waterside levee slope and foundation. Overall judgment based contributions account for a Pr(f) of 55.3% of the without project combined curve at the levee crest. Figure 18-1 presents the without project conditions combined curve.

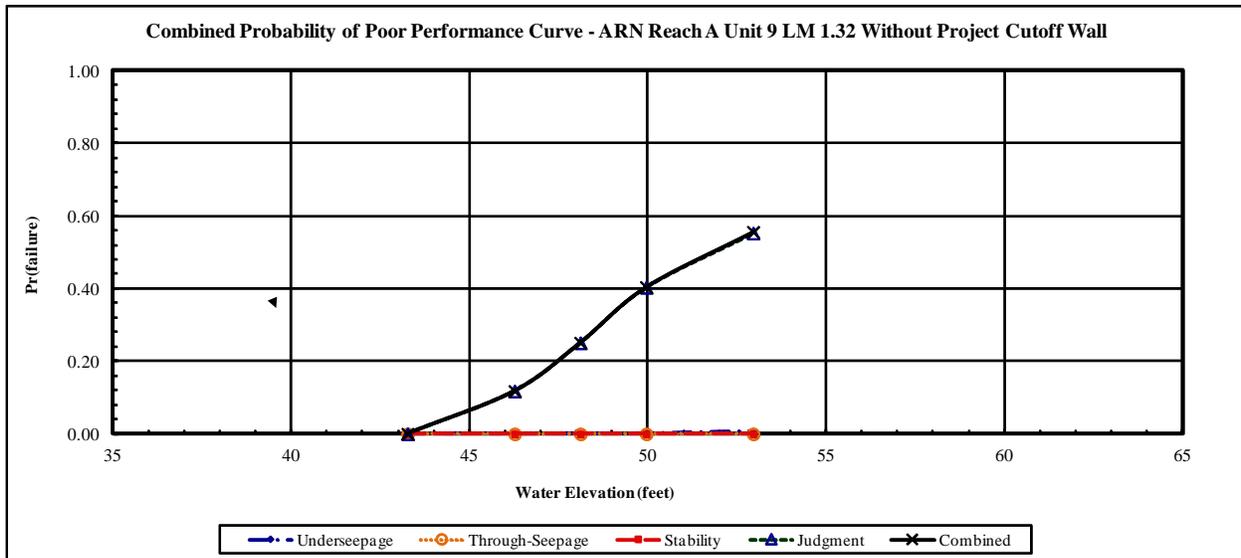


Figure 18-1: Combined Probability of Poor Performance for Without Project Conditions

The remaining probability of failure was primarily attributed to the judgment based failure modes, erosion, which is proposed to be mitigated through the placement riprap erosion protection. With project improvement measures reduce erosion to a Pr(f) of 5.0% at the levee crest. Figure 18-2 presents the with project conditions combined curve.

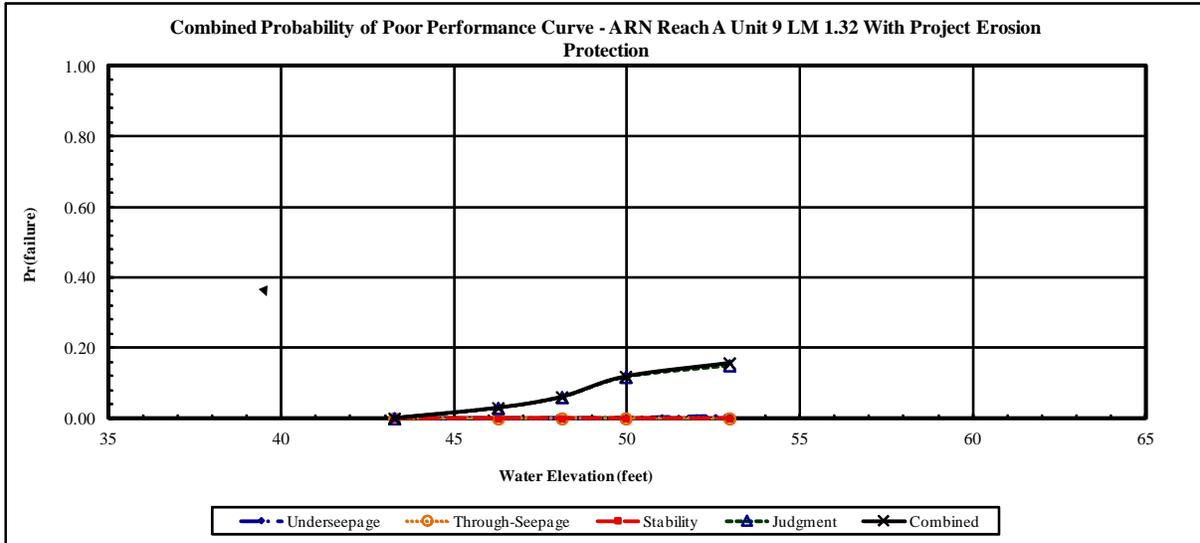


Figure 18-2: Combined Probability of Poor Performance for With Project Conditions

18.2 ARN REACH E – ARCADE CREEK NORTH – U7 LM 0.90

Borings chosen to be used in probabilistic analyses resulted in a mean blanket thickness value of 6.0 ft with a coefficient of variation of 50, and a mean aquifer thickness of 25.0 ft with a coefficient of variation of 36. The blanket was comprised of predominantly lean clay. The aquifer was made up of silty sands, poorly graded sands, and silts.

The without project underseepage analysis resulted in a Pr(f) of 72.4% at the levee crest and landside slope stability analysis resulted in a Pr(f) of 68.1% at the crest. The without project judgment based probability portion of the curve was comprised mainly of utilities and animal burrows both of which account for a Pr(f) of 6.0% at the crest. Overall judgment based contributions account for a Pr(f) of 17.7% of the without project combined curve at the levee crest. Figure 18-3 presents the without project conditions combined curve.

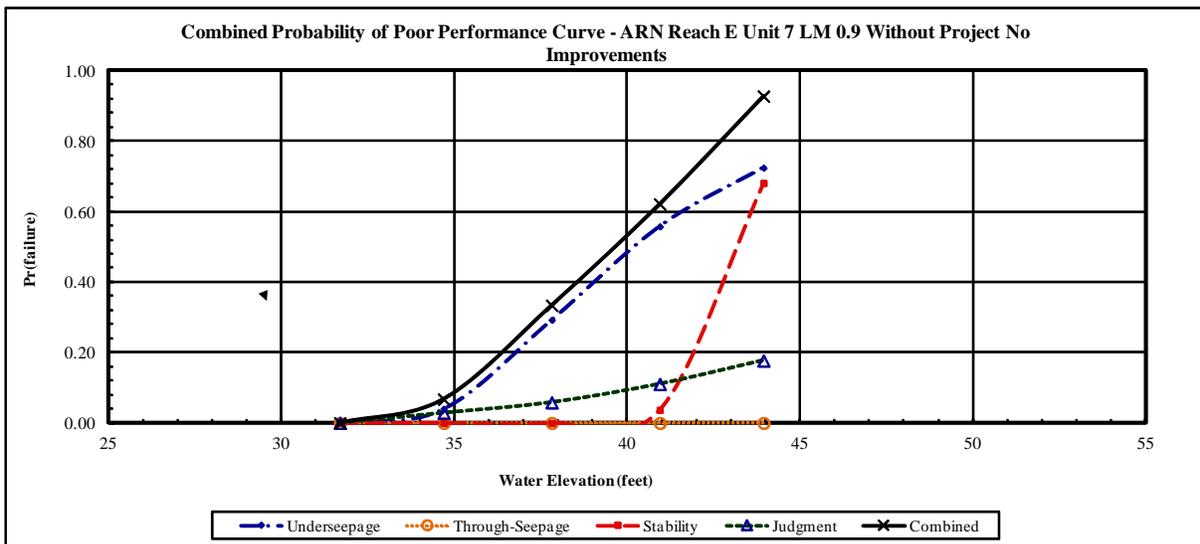


Figure 18-3: Combined Probability of Poor Performance for Without Project Conditions

With project conditions analyses were completed with the incorporation of an underseepage cutoff wall. This improvement mitigates underseepage and landside slope stability concerns. With project conditions did not provide a reduction to judgment based probabilities of failure. As such the Pr(f) for both utilities and animal burrows remained at 6.0% at the levee crest. Figure 18-4 presents the with project conditions combined curve.

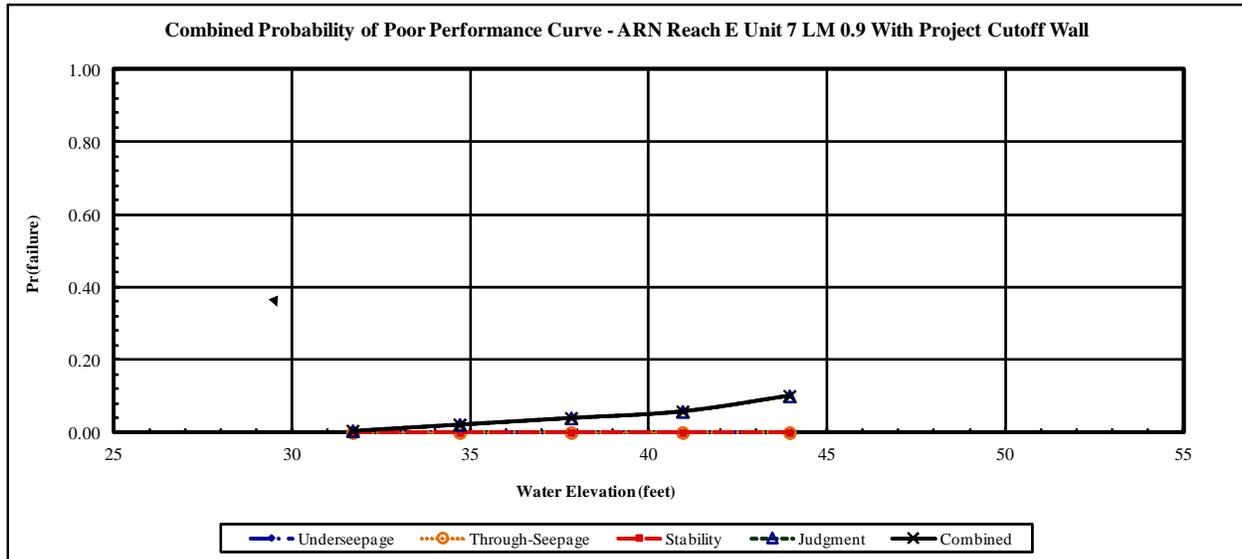


Figure 18-4: Combined Probability of Poor Performance for With Project Conditions

18.3 ARS REACH B – AMERICAN RIVER SOUTH – U4 LM 3.90

Borings chosen to be used in probabilistic analyses resulted in a mean blanket thickness value of 10.0 ft with a coefficient of variation of 30, and a mean aquifer thickness of 41.0 ft with a coefficient of variation of 24. The blanket was comprised of predominantly silts and lean clays. The aquifer was made up of silty sands and poorly graded sands.

The levee embankment contains an existing cutoff wall which mitigates both underseepage, through seepage, and slope stability concerns. The without project judgment based probability portion of the curve was comprised mainly of erosion, and encroachments, which accounted for 60.0% and 4.0% respectively at the crest. Past performance has indicated significant amounts of erosion of the riverbank, waterside levee slope and foundation. The combined without project levee performance curve resulted in a Pr(f) of 65.0% at the levee crest. Figure 18-5 presents the without project conditions combined curve.

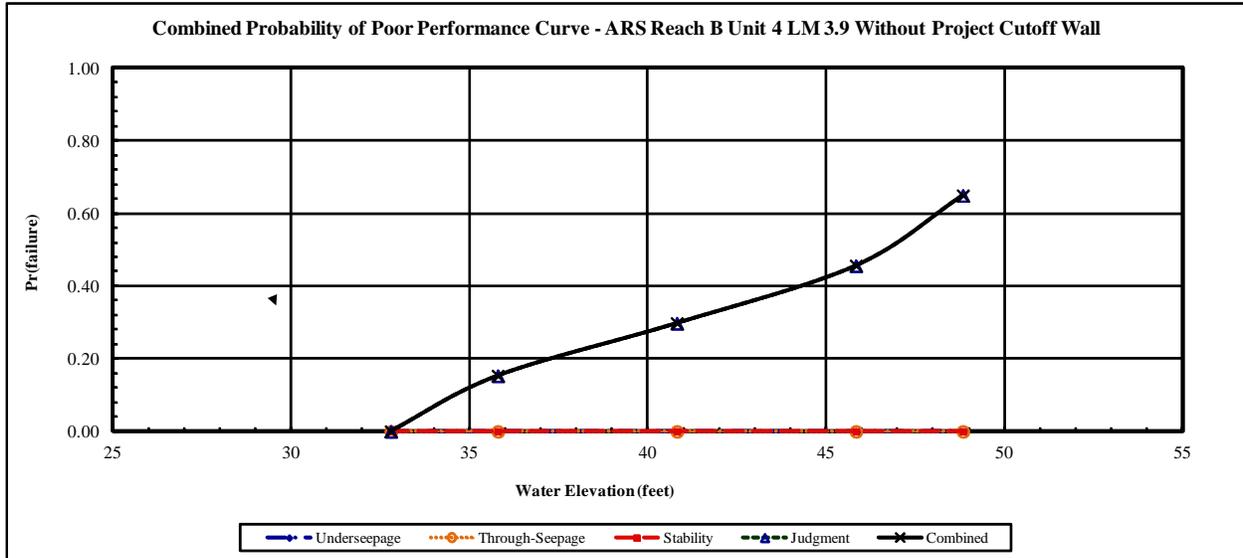


Figure 18-5: Combined Probability of Poor Performance for Without Project Conditions

The high probability of failure was primarily attributed to the erosion portion of the judgment based failure modes, which is proposed to be mitigated through the placement of riprap erosion protection. With project improvement measures reduce erosion to a Pr(f) of 5.0%. Figure 18-6 presents the with project conditions combined curve.

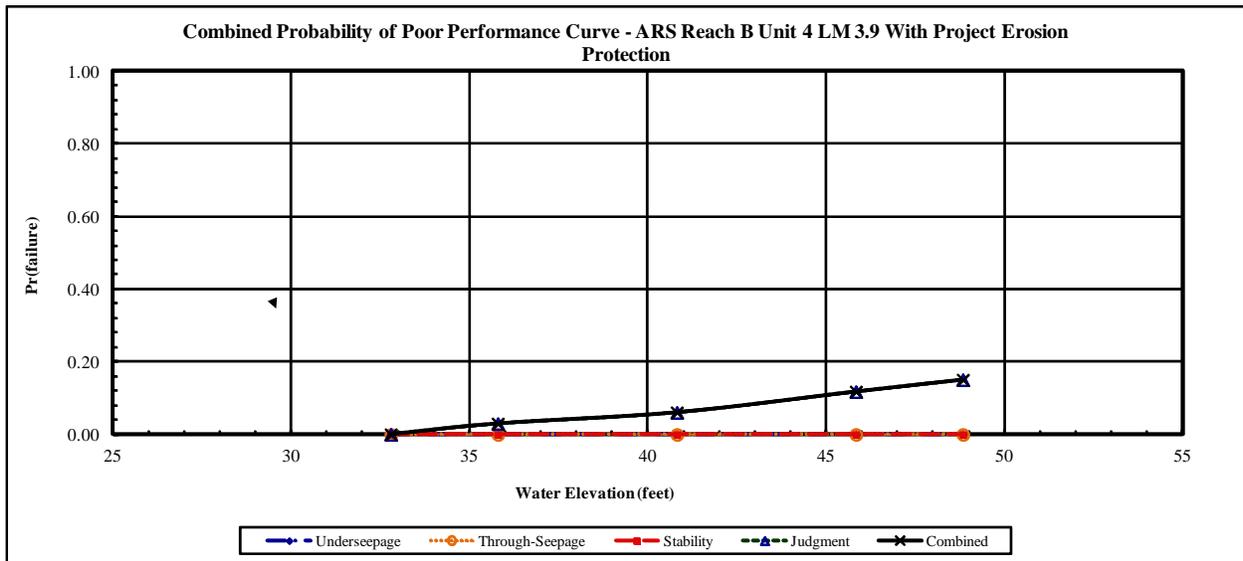


Figure 18-6: Combined Probability of Poor Performance for With Project Conditions

18.4 ARS REACH E – SACRAMENTO RIVER EAST –U1 LM 1.12

Borings chosen to be used in probabilistic analyses resulted in a mean blanket thickness value of 14.0 ft with a coefficient of variation of 86, and a mean aquifer thickness of 79.0 ft with a coefficient of variation of 25. The blanket was comprised of predominantly lean clays and silts. The aquifer was made up of mainly silty sands, and poorly graded sands.

The without project underseepage analysis resulted in a Pr(f) of 34.4% at the levee crest and landside slope stability analysis resulted in a Pr(f) of 84.8% at the crest. The levee embankment contains an existing shallow cutoff wall which mitigates through seepage concerns. The without project judgment based probability portion of the curve was comprised mainly of erosion, and encroachments, which accounted for 8.0% and 6.0% respectively at the crest. Overall judgment contributions account for a Pr(f) of 22.7% of the without project combined curve at the levee crest. Figure 18-7 presents the without project conditions combined curve.

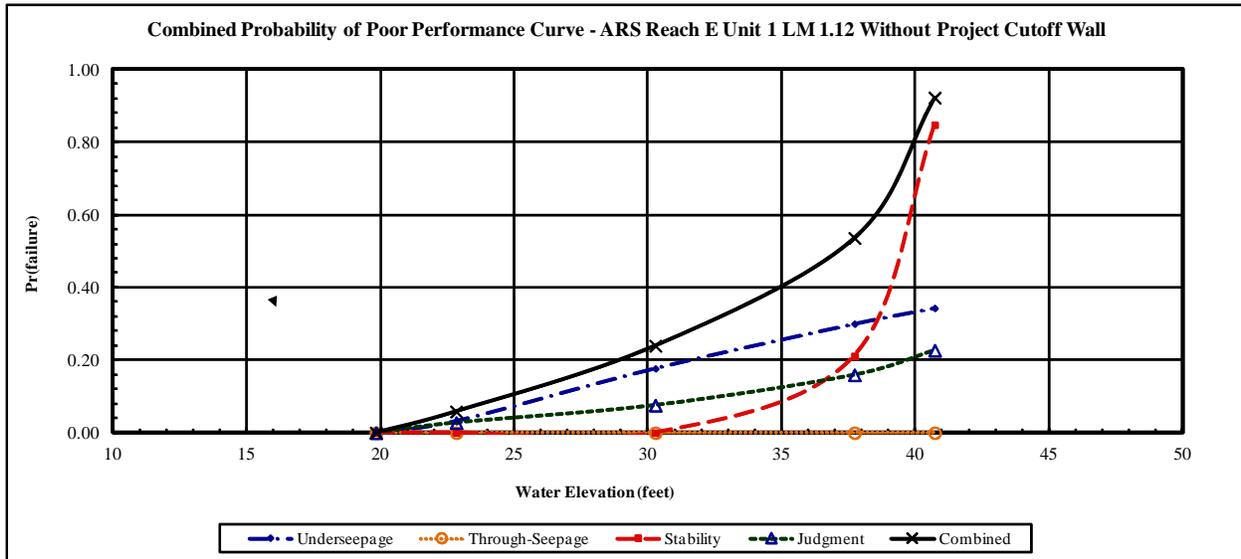


Figure 18-7: Combined Probability of Poor Performance for Without Project Conditions

With project conditions analyses were completed with the incorporation of an underseepage cutoff wall and waterside erosion protection. These improvements mitigated underseepage, landside slope stability, and erosion concerns. With project conditions analysis reduce erosion, encroachments, and utilities to a Pr(f) of 2.0%, 3.5%, and 1.0% as during construction, a portion of existing encroachments and utilities will be removed or relocated to allow for proper inspection and maintenance. Figure 18-8 presents the with project conditions combined curve.

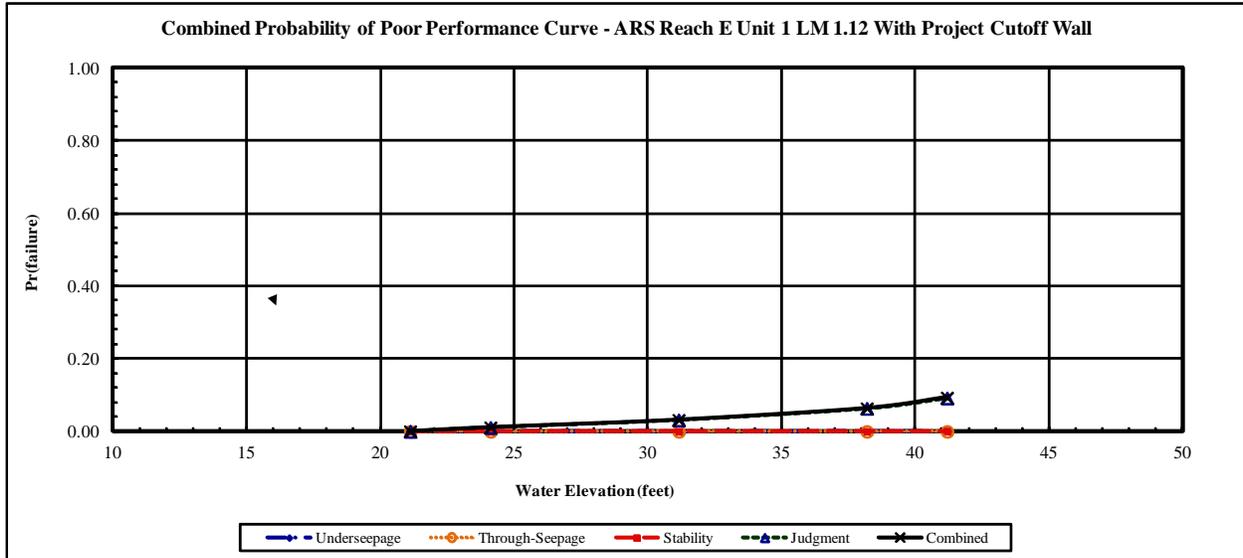


Figure 18-8: Combined Probability of Poor Performance for With Project Conditions

18.5 NAT REACH D – NCC SOUTH –U1 LM 1.30

The NPACR recommended a cutoff wall constructed through the existing levee section for Reach D of Natomas which mitigated underseepage and landside slope stability concerns. The change in Pr(f) for seepage was 4.2% from the design water surface elevation to the top of levee water surface elevation, given the inclusion of the cutoff wall. The judgment based probabilities also added a Pr(f) of 9.3%, comprised mainly of animal burrows and encroachments, which contributed a Pr(f) of 7.0% and 5.0% respectively. Figure 18-9 presents the with project levee performance curve from the NPACR which included a levee raise component. It is also the with project conditions levee performance curve for the ARCF GRR.

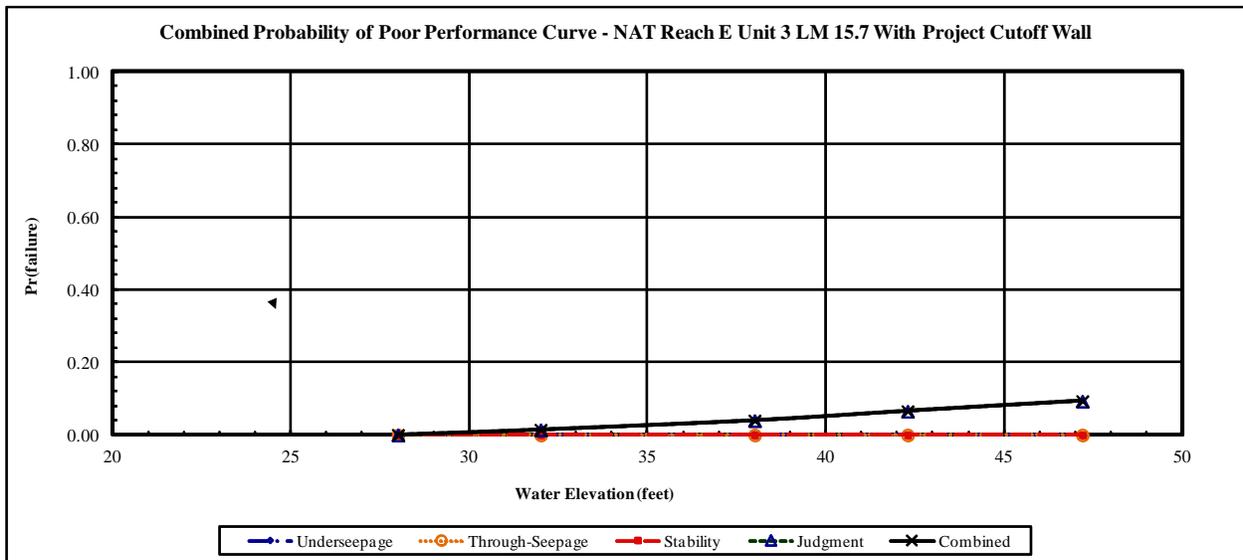


Figure 18-9: Combined Probability of Poor Performance for With Project Conditions

19.0 MATERIAL REQUIREMENTS AND BORROW SITES

It is anticipated that significant quantities of material will be required for construction of the proposed project. Several different improvement measures such as seepage berms, cutoff walls, embankment construction/reconstruction, and erosion protection are proposed. The SOP-03 established the requirements of engineered fill to be used for the construction of the levee embankments.

The material is expected to be sourced from several sites including; newly identified borrow sites within approximately 25 miles of the study area, existing borrow sites identified for the Natomas Basin by SAFCA, the DWSC dredge disposal area, the existing levees, and existing commercial sources. A desktop regional borrow study was performed to identify potential borrow sites, within 25 miles of the study area, where enough soil could be sourced to satisfy the project needs. Plates 6 and 7 show the high confidence and low confidence areas of potential borrow sites. Test pits and laboratory testing on materials collected from test pits were provided by SAFCA as part of the NLIP for borrow sites established for the Natomas Basin. Additionally, the Sacramento District has studied the DWSC spoil areas as a borrow source several time in the past, and a discussion of that borrow source is included below. Typically projects constructed by the Sacramento District utilize commercial borrow sites near the project area.

20.0 CONCLUSIONS

This report presented the results of geotechnical analyses and feasibility level geotechnical design recommendations associated with the various alternatives under consideration to address technical deficiencies in the flood risk management system protecting the study area. The alternatives consisted of a combination of structural measures to mitigate deficiencies with levee height, geometry, erosion, access, vegetation, seepage, and slope stability. They also included, the widening and construction of new levees on the Sacramento Bypass and Weir and/or a Diversion Structure on the Sacramento River.

The results of the without project seepage and slope stability analyses indicated that the levees in ARN Reaches C through F and ARS Reaches D through G did not meet minimum criteria. The analyses showed that the levees did not meet criteria at varying flood frequencies typically between the 25 and 200 year events. The with project analyses typically included deep cutoff walls which resulted in the with project levee analyses satisfying criteria.

Two potentially erosion-resistant units were identified in the stratigraphy of the American River: (1) a moderately cohesive silty and sandy interbed of relatively limited lateral and longitudinal extent within a thicker package of loose Holocene sediments (the “upper” unit); and (2) much thicker, more widespread relatively erosion-resistant deposits associated with the Pleistocene-aged Fair Oaks formation (the “lower unit”). Erosion rate testing confirmed that erodibility was related to the geologic unit and that most of the specimens within the Fair Oaks formation could be categorized as Moderate Resistant to Very Resistant and similarly, the layers above erosionally resistant surface could be categorized as Very Erodible to Erodible.

The results of the liquefaction triggering analysis and liquefaction-induced post-earthquake deformation based on limit equilibrium analysis indicated that liquefaction potential is highly likely at each critical location for all the reaches for Natomas Basin, Reach A of American River North Basin, and Reaches C to G of American River South Basin. Moreover, at these locations, the analysis indicates that the post-earthquake deformation as the result of liquefaction of the material beneath the embankment is a global or structural failure mode that is very likely to compromise the ability to provide flood protection at these critical locations.

Based on analyses at LM 5.92 that incorporated tree fall and scour it was shown that the tree fall and scour did not significantly affect levee performance and that the levee meets USACE seepage and slope stability criteria. Therefore, it was a reasonable conclusion that with a VVR to allow vegetation to remain, the safety, structural integrity, and functionality of the Sacramento River levee would be retained.

The without project levee performance curves indicate that the levees in ARN Reaches C through F and ARS Reaches D through G would perform unsatisfactorily when minimally loaded. In general, the analyses identified underseepage deficiencies and/or underseepage related slope stability deficiencies. Therefore, the with project levee performance curves typically included deep cutoff walls which resulted in significant reduction in probabilities of poor performance.

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